



Handbook of seismic risk analysis and management of civil infrastructure systems

Edited by S. Tesfamariam and K. Goda

Handbook of seismic risk analysis and management of civil infrastructure systems

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Edited by
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Contents

<i>Contributor contact details</i>	<i>xvii</i>
<i>Preface</i>	<i>xxv</i>
Part I Ground motions and seismic hazard assessment	1
1 Probabilistic seismic hazard analysis of civil infrastructure	3
G. M. ATKINSON, Western University, Canada and K. GODA, University of Bristol, UK	
1.1 Introduction: past developments and current trends in assessing seismic risks	3
1.2 Simulation-based probabilistic seismic hazard analysis (PSHA)	7
1.3 Extension of probabilistic seismic hazard analysis (PSHA) to advanced earthquake engineering analyses	17
1.4 Conclusions and future trends	23
1.5 References	25
2 Uncertainties in ground motion prediction in probabilistic seismic hazard analysis (PSHA) of civil infrastructure	29
P. J. STAFFORD, Imperial College London, UK	
2.1 Introduction	29
2.2 Explanation of ground-motion prediction equations (GMPEs)	30
2.3 Development of ground-motion prediction equations (GMPEs)	37
2.4 Sensitivity of model components	45
2.5 Future trends	51
2.6 Conclusions	54
2.7 References	54

3	Spatial correlation of ground motions in estimating seismic hazards to civil infrastructure	57
	V. SOKOLOV and F. WENZEL, Karlsruhe Institute of Technology (KIT), Germany	
3.1	Introduction	57
3.2	Spatial correlation of ground motions: evaluation and analysis	58
3.3	Ground-motion correlation and seismic loss assessment	69
3.4	Future trends	74
3.5	References	75
4	Ground motion selection for seismic risk analysis of civil infrastructure	79
	B. A. BRADLEY, University of Canterbury, New Zealand	
4.1	Introduction	79
4.2	Ground motion selection in seismic performance assessment	80
4.3	Case study: bridge foundation soil system	83
4.4	The generalized conditional intensity measure (GCIM) approach	84
4.5	Ground motion selection using generalized conditional intensity measure (GCIM)	87
4.6	Application of the ground motion selection methodology	93
4.7	Checking for bias in seismic response analysis due to ground motion selection	104
4.8	Seismic demand curve computation	105
4.9	Software implementations	108
4.10	Conclusions and future trends	108
4.11	References	110
5	Assessing and managing the risk of earthquake-induced liquefaction to civil infrastructure	113
	J. F. MIAN, Arup, UK, S. KONTOE, Imperial College London, UK and M. FREE, Arup, UK	
5.1	Introduction	113
5.2	Hazard identification	117
5.3	Hazard quantification	121
5.4	Response of infrastructure to liquefaction hazards	125
5.5	Tolerable risks and performance levels	133
5.6	Conclusions	134
5.7	References	135

Part II	Seismic risk analysis methodologies	139
6	Seismic risk analysis and management of civil infrastructure systems: an overview	141
	S. TESFAMARIAM, The University of British Columbia, Canada and K. GODA, University of Bristol, UK	
6.1	Introduction	141
6.2	Uncertainty in risk analysis	143
6.3	Risk analysis	150
6.4	Risk management	155
6.5	Conclusions	169
6.6	References	170
7	Seismic risk analysis using Bayesian belief networks	175
	S. TESFAMARIAM, The University of British Columbia, Canada and Z. LIU, Toyota Technological Institute, Japan	
7.1	Introduction	175
7.2	Bayesian belief networks (BBN)	176
7.3	Application of Bayesian belief networks (BBN) to seismic risk assessment: site-specific hazard assessment	181
7.4	Regional damage estimation	187
7.5	Vulnerability and damage assessment of individual buildings	195
7.6	Conclusions and future trends	201
7.7	References and further reading	202
8	Structural vulnerability analysis of civil infrastructure facing seismic hazards	209
	J. AGARWAL, University of Bristol, UK	
8.1	Introduction	209
8.2	Vulnerability, hazard and risk	210
8.3	Identification of vulnerability	212
8.4	Analysis of risk	218
8.5	Vulnerability of infrastructure networks	223
8.6	Advantages of vulnerability analysis	226
8.7	Conclusions	227
8.8	References	227
9	Earthquake risk management of civil infrastructure: integrating soft and hard risks	229
	D. BLOCKLEY, University of Bristol, UK	
9.1	Introduction: the inevitability of risk	229
9.2	Managing technical risks to structures	230

9.3	Reliability theory for the analysis of uncertainty and risk	232
9.4	Seismic vulnerability	235
9.5	Uncertainty: fuzziness, incompleteness and randomness (FIR)	238
9.6	Systems thinking	238
9.7	Process models and project progress maps (PPM)	243
9.8	Measuring evidence of performance	245
9.9	A structural example: procuring a new building	247
9.10	Conclusions	250
9.11	References	253
10	A capability approach for seismic risk analysis and management	255
	P. GARDONI and C. MURPHY, University of Illinois at Urbana-Champaign, USA	
10.1	Introduction	255
10.2	Desiderata for a framework for seismic risk analysis and management	258
10.3	A capability approach for seismic risk analysis and management	259
10.4	Benefits of a capability approach	265
10.5	Conclusions	266
10.6	Acknowledgments	266
10.7	References	266
11	Resilience-based design (RBD) modelling of civil infrastructure to assess seismic hazards	268
	G. P. CIMELLARO, Politecnico di Torino, Italy	
11.1	Introduction	268
11.2	Development of performance-based design (PBD)	270
11.3	Towards resilience-based design (RBD)	271
11.4	Case studies	292
11.5	Conclusions	299
11.6	Future trends	300
11.7	Acknowledgements	300
11.8	References and further reading	300
Part III	Assessing seismic risks to buildings	305
12	Assessing seismic risks for new and existing buildings using performance-based earthquake engineering (PBEE) methodology	307
	T. Y. YANG, The University of British Columbia, Canada	

12.1	Introduction	307
12.2	Performance-based earthquake engineering (PBEE) framework	308
12.3	Application: seismic performance assessment of high-rise buildings	321
12.4	Conclusions	331
12.5	Acknowledgments	332
12.6	References	332
13	Assessing the seismic vulnerability of masonry buildings D. D'AYALA, University College London, UK	334
13.1	Introduction	334
13.2	Vulnerability approaches: empirical and analytical	336
13.3	Collapse-mechanism approach to seismic vulnerability of masonry buildings	342
13.4	Stochastic and epistemic uncertainty quantification	354
13.5	Conclusions	360
13.6	References and further reading	362
14	Vulnerability assessment of reinforced concrete structures for fire and earthquake risk H. MOSTAFAEI, National Research Council, Canada	366
14.1	Introduction	366
14.2	Structural response to fire	368
14.3	Seismic response of structures	369
14.4	Fire performance of a reinforced concrete building following an earthquake	370
14.5	Residual seismic resistance of fire-damaged building columns	379
14.6	Lateral load resistance of a fire-damaged column using a hybrid method	383
14.7	Conclusions and future trends	383
14.8	References	385
15	Seismic risk models for aging and deteriorating buildings and civil infrastructure M. SÁNCHEZ-SILVA and J. RIASCOS-OCHOA, Universidad de Los Andes, Colombia	387
15.1	Introduction	387
15.2	Structural degradation	389
15.3	Shock-based damage accumulation models	391
15.4	Approximation to graceful deterioration	399

x	Contents	
15.5	Combined progressive and shock-based deterioration	403
15.6	Conclusions	406
15.7	References and further reading	406
16	Stochastic modeling of deterioration in buildings and civil infrastructure	410
	R. KUMAR, Texas A&M University, USA and P. GARDONI, University of Illinois at Urbana-Champaign, USA	
16.1	Introduction	410
16.2	A general deterioration process	411
16.3	Modeling of a general deterioration process using the stochastic semi-analytical approach (SSA)	414
16.4	Stochastic modeling of deterioration in reinforced concrete (RC) bridges	418
16.5	Conclusions	430
16.6	References	432
Part IV	Assessing seismic risks to bridges and other components of civil infrastructure networks	435
17	Risk assessment and management of civil infrastructure networks: a systems approach	437
	M. SÁNCHEZ-SILVA and C. GÓMEZ, Universidad de Los Andes, Colombia	
17.1	Introduction	437
17.2	Systems and networks	438
17.3	Hierarchical representation of networks	442
17.4	Risk assessment of infrastructure networks	446
17.5	Optimal resource allocation in infrastructure networks	454
17.6	Conclusions	459
17.7	References and further reading	460
18	Seismic vulnerability analysis of a complex interconnected civil infrastructure	465
	P. FRANCHIN and F. CAVALIERI, University of Rome 'La Sapienza', Italy	
18.1	Introduction and definitions	465
18.2	Time, space and stakeholder dimensions of the problem	466
18.3	Model, analysis type and interactions	468
18.4	Object-oriented model (OOM) of the infrastructure and hazards	470
18.5	Description of the main classes	472

18.6	Performance metrics	488
18.7	Probabilistic assessment of the model	493
18.8	Example of an application of seismic vulnerability analysis	499
18.9	Future trends	509
18.10	Acknowledgements	509
18.11	References	510
19	Seismic reliability of deteriorating reinforced concrete (RC) bridges P. GARDONI, University of Illinois at Urbana-Champaign, USA and D. TREJO, Oregon State University, USA	514
19.1	Introduction	514
19.2	Mechanisms of deterioration	515
19.3	Effects of deterioration on the reliability of bridges	519
19.4	Conclusions	531
19.5	References	532
20	Using a performance-based earthquake engineering (PBEE) approach to estimate structural performance targets for bridges Z. GÜLERCE, Middle East Technical University, Turkey	536
20.1	Introduction	536
20.2	Performance-based seismic evaluation framework (PEER approach)	537
20.3	Probabilistic seismic demand analysis (PSDA)	542
20.4	Vector-valued probabilistic seismic hazard assessment (VPSHA)	544
20.5	Performance-based seismic evaluation of ordinary highway bridges	546
20.6	Future trends	556
20.7	Acknowledgments	558
20.8	References	558
21	Incremental dynamic analysis (IDA) applied to seismic risk assessment of bridges P. TEHRANI and D. MITCHELL, McGill University, Canada	561
21.1	Introduction	561
21.2	Incremental dynamic analysis (IDA)	563
21.3	Structural modelling for incremental dynamic analysis (IDA)	564
21.4	Sources of uncertainty	567
21.5	Record selection for incremental dynamic analysis (IDA)	568

xii	Contents	
21.6	Development of fragility curves using incremental dynamic analysis (IDA) results	579
21.7	Case study for a continuous 4-span bridge	580
21.8	Conclusions and future trends	591
21.9	Sources of further information	592
21.10	References	592
22	Effect of soil–structure interaction and spatial variability of ground motion on seismic risk assessment of bridges	597
	A. SEXTOS, Aristotle University Thessaloniki, Greece	
22.1	Introduction	597
22.2	Soil–foundation–pier–superstructure interaction	599
22.3	Embankment–backfill–abutment–superstructure interaction	607
22.4	Realistic earthquake excitation scenarios for interactive soil–bridge systems	610
22.5	Conclusions	619
22.6	References	620
23	Seismic risk management for water pipeline networks	626
	T. KOIKE, Kyoto University, Japan	
23.1	Introduction	626
23.2	Seismic failure of a lifeline system	627
23.3	Seismic risk assessment	641
23.4	Seismic risk mitigation	650
23.5	Future trends	656
23.6	References	657
24	Seismic risk assessment of water supply systems	659
	Y. WANG, City University of Hong Kong, Hong Kong	
24.1	Introduction	659
24.2	General framework for evaluating seismic risk	660
24.3	System characteristics	660
24.4	Seismic hazards	664
24.5	Component responses	669
24.6	System responses	672
24.7	Economic and social consequences	676
24.8	Future trends	677
24.9	Sources of further information and advice	678
24.10	Acknowledgments	678
24.11	References	678

25	Seismic risk assessment for oil and gas pipelines	682
	D. G. HONEGGER, D. G. Honegger Consulting, USA and D. WIJEWICKREME, The University of British Columbia, Canada	
25.1	Introduction	682
25.2	Purpose of performing a risk assessment	683
25.3	Key steps in performing risk assessments for oil and gas pipelines	685
25.4	Types of seismic hazard	688
25.5	Determining hazard likelihood	691
25.6	Determining severity of hazard	692
25.7	Pipeline response to earthquake hazards	696
25.8	Consequences of pipeline damage	699
25.9	Mitigation approaches to reduce risk to pipelines	702
25.10	Challenges and issues	705
25.11	Future trends	708
25.12	Conclusions	712
25.13	References	712
 26	 Seismic risk analysis of wind turbine support structures	 716
	M. MARDFEKRI, Texas A&M University, USA and P. GARDONI, University of Illinois at Urbana-Champaign, USA	
26.1	Introduction	716
26.2	Probabilistic demand models	718
26.3	Demand models for the support structure of offshore wind turbines	719
26.4	Example of fragility estimates for an offshore wind turbine support structure	733
26.5	Conclusions	735
26.6	Future trends	737
26.7	Acknowledgments	737
26.8	References	737
 Part V	 Assessing financial and other losses from earthquake damage	 739
 27	 Seismic risk and possible maximum loss (PML) analysis of reinforced concrete structures	 741
	H. YOSHIKAWA, Tokyo City University, Japan	
27.1	Introduction	741

xiv	Contents	
27.2	Analytical procedure for assessing seismic risk	743
27.3	Case studies of seismic risk analysis for reinforced concrete structures	747
27.4	Conclusions and future trends	757
27.5	References	758
28	Seismic risk management of insurance losses using extreme value theory and copula K. GODA, University of Bristol, UK and J. REN, Western University, Canada	760
28.1	Introduction	760
28.2	Statistical modelling of extreme data	763
28.3	Insurer's earthquake risk exposure modelling	767
28.4	Earthquake insurance portfolio analysis	774
28.5	Conclusions and future trends	783
28.6	References	784
29	Probabilistic assessment of earthquake insurance rates for buildings M. S. YUCEMEN, Middle East Technical University, Turkey	787
29.1	Introduction	787
29.2	Probabilistic model for the assessment of earthquake insurance rates	789
29.3	Application: assessment of earthquake insurance rates for different seismic zones in Turkey	795
29.4	Implementation of earthquake insurance: Turkish Catastrophe Insurance Pool (TCIP)	803
29.5	Conclusions and future trends	808
29.6	Acknowledgments	811
29.7	References	811
30	Assessing global earthquake risks: the Global Earthquake Model (GEM) initiative H. CROWLEY, R. PINHO, M. PAGANI and N. KELLER, GEM Foundation, Italy	815
30.1	Introduction	815
30.2	Current status of Global Earthquake Model (GEM)	822
30.3	OpenQuake	833
30.4	Outlook for Global Earthquake Model (GEM)	837
30.5	References	837

31	Strategies for rapid global earthquake impact estimation: the Prompt Assessment of Global Earthquakes for Response (PAGER) system K. S. JAISWAL and D. J. WALD, National Earthquake Information Center, USA	839
31.1	Introduction	839
31.2	State-of-the-art of rapid earthquake loss estimation systems	840
31.3	Prompt Assessment of Global Earthquakes for Response (PAGER) system development	842
31.4	Earthquake loss models within the Prompt Assessment of Global Earthquakes for Response (PAGER) system	844
31.5	Earthquake impact scale	855
31.6	Loss estimation for recent earthquakes	857
31.7	Prompt Assessment of Global Earthquakes for Response (PAGER) products and ongoing developments	859
31.8	Conclusions	862
31.9	Acknowledgments	863
31.10	References	863
	<i>Index</i>	867

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What makes earthquake risks so catastrophic and dreadful? Possible reasons are attributed to the sudden, unpredictable, involuntary, and large-scale nature of the risks. As a consequence of a devastating earthquake, a tremendous surge of seismic damage and loss may overwhelm the risk-bearing capacities of households, companies, municipalities, and insurers, and may eventually cause serious long-term effects on communities and the regional/national economy. Recent earthquake catastrophes, including the 2004 Indian Ocean, 2008 Wenchuan, 2010 Haiti, and 2011 Christchurch earthquakes, have demonstrated that urban cities and communities in seismic regions are vulnerable to extreme seismic events. In the 2011 Tohoku, Japan earthquake, the number of fatalities was in excess of 19 000, and the total economic loss has reached 300–400 billion U.S. dollars. Earthquakes impact entire physical infrastructure, including buildings and lifelines (water and sewage networks, bridges, power supply, etc.). Furthermore, various stakeholders (e.g. global companies with supply-chain networks around the world and insurance/reinsurance companies with a widespread portfolio of businesses and investments) are affected by such catastrophes due to the complex and inter-connected global economy. In a nutshell, we live in a highly uncertain and vulnerable world due to large-scale natural disasters, and global earthquake risks must be mitigated to achieve more resilient and sustainable society.

The current framework for seismic risk analysis addresses quantification of magnitude and recurrence of earthquake, vulnerability of infrastructure systems, and consequences due to damage. Developing a comprehensive model for all types of structures and lifelines is a challenging task. A system-based model can be used, where a system is defined as an ‘assemblage of components acting as a whole’; each system in turn encapsulates different subcomponents, each of which can be described as a subsystem. In structural safety evaluation, system response to earthquake loading is of paramount importance and is often affected by complex interdependency of subsystems in terms of structural resilience and functionality. A notable

example is the cascading failures of infrastructure systems during a natural disaster; the functionality of a critical facility, such as a hospital, is influenced by the structural resilience as well as the surrounding lifeline networks (e.g. water, electricity, and staff availability). Different approaches and methods are available to address a wide variety of engineering problems, and suitable models are required to analyse the problems. It is also important to recognise that all models are necessarily incomplete and somewhat in error; and the system being modelled may have inherent variability or un-measurability in its behaviour. Despite these limitations, the systems approach has a great utility and future potential for assessing seismic performance of civil infrastructure and for more effective seismic risk management and decision making.

Primary role and purpose of risk analysis and management are to quantify uncertainty and facilitate sound and efficient decision making. Risk management is a process of weighting alternatives (options) and selecting the most appropriate action by integrating the results of risk assessment with engineering data as well as social/economic/political factors to reach an acceptable decision. Generally, the risk assessment/analysis process involves objectivity, whereas risk management involves preferences and attitudes which have both objective and subjective elements. In seismic risk analysis and loss estimation, uncertainties are prevalent in hazard assessment, ground condition, ground motion prediction equations, building stock and infrastructure exposure, and vulnerability of infrastructure. Typology and definition of uncertainty within the engineering community are broad. In risk analysis, uncertainty can be categorised into aleatory and epistemic uncertainty. Aleatory uncertainty (variability) is due to natural heterogeneity or stochasticity of a physical process and it cannot be reduced, while epistemic uncertainty is due to ignorance or subjectivity, which can be reduced with availability of more information. Therefore, consideration of uncertainty in any risk analysis is unavoidable, and modelling and treatment of epistemic uncertainties are critically important.

The book, which is in five parts, is intended to address the complex and multi-stakeholder nature of risk analysis and management.

- Part I describes the state of the art in ground motions and seismic hazard assessment. Chapters in Part I cover the current and future development of probabilistic seismic hazard analysis, spatial correlation and uncertainties in ground motion prediction, ground motion record selection for improved seismic performance evaluation of engineering structures, and liquefaction assessment.
- Part II covers a broad topic in seismic risk assessment and management. The topics include seismic risk analysis and management with incorporation of uncertainties, integrating soft and hard risks, application of a

Bayesian belief network, structural vulnerability, novel capability-based design approach for disaster management, and proposal for a new resilience-based design.

- Part III deals with various risk assessments of buildings subject to different hazards (seismic, fire, deterioration, etc.). This entails the state of the art in performance-based earthquake engineering (PBEE) methodology, seismic vulnerability of masonry, seismic and fire vulnerability of reinforced concrete structures, and risk assessment of buildings subject to stochastic deterioration.
- Part IV deals with seismic risk assessments of bridges and other lifeline infrastructure networks. Lifeline structures (e.g. pipeline network and road network) are linear systems, and require consideration of a system-based approach that accounts for the complexity and interdependence. Chapters in Part IV cover a PBEE framework of bridge risk assessment, subject to deterioration and consideration soil–structure interaction, and a novel application of incremental dynamic analysis for bridge seismic risk assessment. Moreover, Part IV presents seismic risk assessment and management of water and oil pipeline networks, and wind turbine support structures.
- Part V deals with assessing financial and other losses from earthquake damage. This entails models for quantitative seismic risk and possible maximum loss (PML) analysis, seismic risk management, and probabilistic assessment of insurance losses from earthquakes. Furthermore, a rapid earthquake loss estimation models for civil infrastructure, the Prompt Assessment of Global Earthquake for Response (PAGER) system, is discussed, and finally, a new Global Earthquake Model (GEM) initiative that deals with assessing global earthquake risks is presented.

This book has highlighted the complex nature of seismic risk analysis and management, and the need for a multi-disciplinary approach. The authors that contributed to each chapter are experts in each topic, and have provided the state of the art and future direction. It has brought experts from a wide range of fields, such as engineering seismology, earthquake engineering, risk management, social sciences, and mathematics/statistics. It is envisioned that collection of such chapters in one book can be used as a good reference for decision makers and other stakeholders. It can also inspire the younger generation of researchers, practitioners and educators in this field. Hopefully, by targeting such an audience, this book has a far-reaching and long-term impact.

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Probabilistic seismic hazard analysis of civil infrastructure

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Abstract: This chapter summarises the past and current developments of probabilistic seismic hazard analysis, which forms the fundamental basis for many modern earthquake design code provisions and practices. A focus is given to an approach based on Monte Carlo simulation, because it offers flexibility and rigorousness in the assessment. The method is explained by following a step-by-step procedure and by using a recent case study for western Canada. Two illustrations, related to liquefaction triggering analysis and seismic vulnerability analysis, are discussed to demonstrate how probabilistic seismic hazard analysis facilitates advanced earthquake engineering applications.

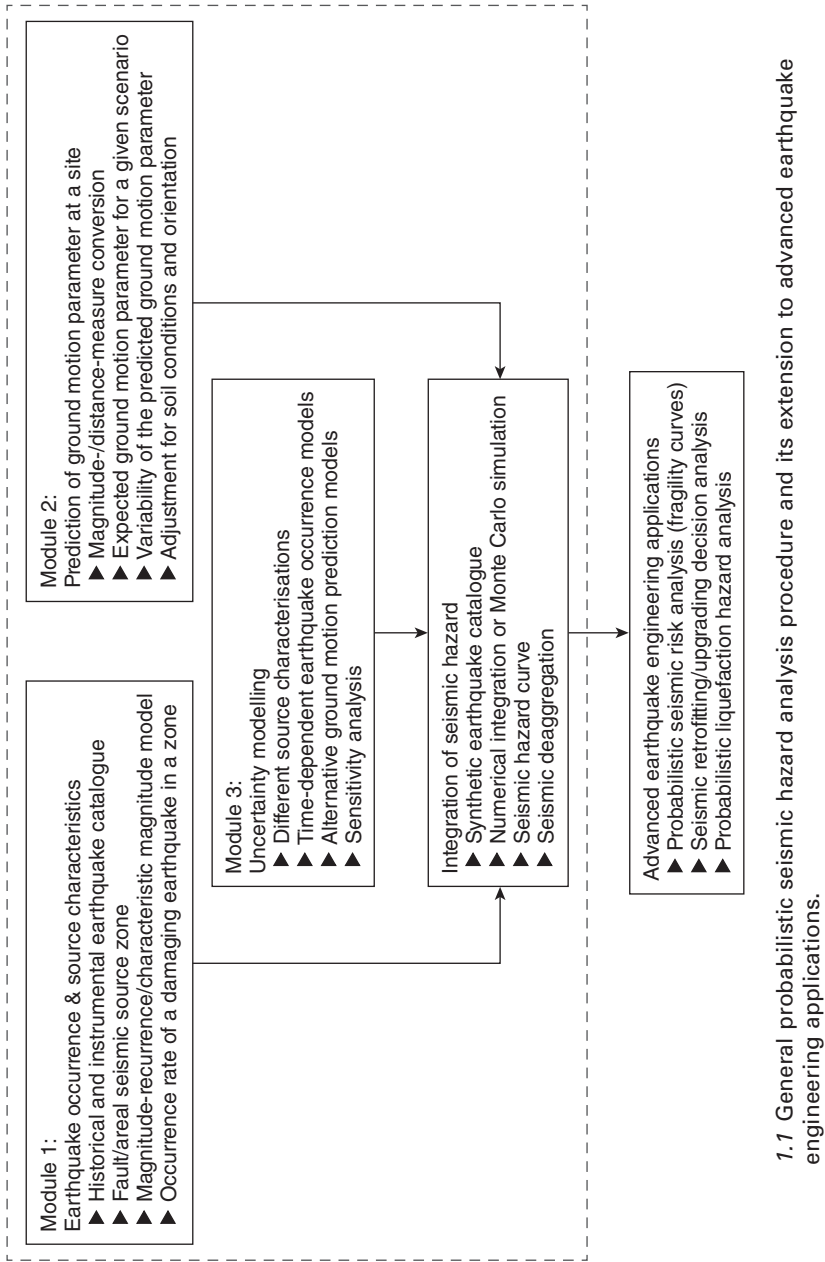
Key words: probabilistic seismic hazard analysis, probabilistic seismic risk analysis, performance-based earthquake engineering, uncertainty, liquefaction.

1.1 Introduction: past developments and current trends in assessing seismic risks

Probabilistic seismic hazard analysis (PSHA) is concerned with the evaluation of the likelihood of strong ground motion intensities (Cornell, 1968; McGuire, 2004, 2008), which may cause destruction of buildings and infrastructure and disruption of economic and social activities. Popular measures for quantifying ground motion intensity include peak ground acceleration (PGA), peak ground velocity (PGV), and spectral accelerations (SAs) at different vibration periods. One of the most important features of PSHA is a comprehensive accounting of uncertainties related to earthquake occurrence, source rupture, wave propagation, and site effects by integrating hazard contributions over all scenarios. Over the past decades, the PSHA methodology has evolved significantly by incorporating more detailed and extended components, such as seismic hazard deaggregation (McGuire, 1995), nonlinear site response analysis (Bazzurro and Cornell, 2004), vector-value PSHA (Baker and Cornell, 2005), near-fault effects (Tothong *et al.*, 2007), and aftershock hazard (Yeo and Cornell, 2009). The outputs from site-specific PSHA are seismic hazard curves (i.e. plot of expected ground

motion levels as a function of probability) and uniform hazard spectra (i.e. ordinates of expected ground motion levels for a given probability as a function of vibration period). These are essential tools for developing national seismic hazard maps and modern seismic design provisions in national building codes. For instance, the current seismic design codes in the United States and Canada specify expected ground motion intensities at 2% probability of exceedance in 50 years (i.e. 2475 years return period level). The adopted probability level is related to regional seismicity and historical progress of seismic design requirements (including intended risk targets) and should be determined by balancing with other social requirements, such as health and welfare (Rosenblueth, 1986).

The conventional PSHA is composed of three major modules: (1) earthquake occurrence in time and space and earthquake source characteristics, (2) ground motion prediction (including path and site effects), and (3) integration of hazard contributions and treatment of uncertainties. An illustration of a typical PSHA procedure (and its extension to conduct more advanced analyses, as described below) is shown in Fig. 1.1. The spatiotemporal earthquake occurrence is often characterised by fault/areal seismic sources. The delineated sources reflect past seismic activities in a region (historical and instrumental earthquake catalogue), and their activity rates can be determined based on a slip rate for a fault segment, and/or characterised by a magnitude–recurrence relationship for an areal zone. As the first module of PSHA, a synthetic regional seismic catalogue can be generated using Monte Carlo simulation (Musson, 2000; Hong *et al.*, 2006). The second module evaluates seismic intensity measures at a site of interest for all significant earthquake scenarios (generated by the first module), typically by using a ground motion prediction equation (GMPE). Internally consistent magnitude and distance measures should be considered in evaluating the ground motions expected at the site (Scherbaum *et al.*, 2004; Bommer *et al.*, 2005). Random scatter of motions about the median GMPE must be incorporated in the assessment. Alternatively, more extensive approaches based on stochastic simulation methods can be adopted (Wen and Wu, 2001). Although the implementation of modules 1 and 2 captures some uncertainties, a more comprehensive consideration of possible alternatives is needed. Such alternatives include: different source characterisations/zones (e.g. kernel smoothing and tessellation; Woo, 1996; Beauval *et al.*, 2006; Hong *et al.*, 2006), time-dependent seismic hazard (e.g. renewal model; Goda and Hong, 2006), uncertainties associated with magnitude–recurrence relationships (e.g. parametric uncertainties and bias due to completeness and magnitude-scale inhomogeneity; Weichert, 1980; Atkinson and McCartney, 2005), and choice of multiple regional GMPEs (e.g. selection of suitable equations and their weighting in a logic tree; Bommer *et al.*, 2005). The above-mentioned issues have been already addressed and



incorporated in modern PSHA studies through the use of a logic tree method (Petersen *et al.*, 2008; Atkinson and Goda, 2011). Nevertheless, difficulties arise, because not all the models that analysts wish to apply are based on consistent data/assumptions and, more importantly, a full range of alternatives and uncertainties is not conceivable. This is an on-going issue in PSHA, and needs to be improved in the near future. Furthermore, it is important to carry out detailed and comprehensive sensitivity analyses as a part of PSHA. The final step of PSHA is the integration of hazard contributions due to all possible scenarios and models/assumptions. This is conventionally done using numerical integration, while Monte Carlo simulation can be used as an alternative (Musson, 2000; Hong *et al.*, 2006). Because the latter is versatile in dealing with various probabilistic models and is easily extendable to advanced earthquake engineering analyses, this chapter is focused on the Monte Carlo approach.

PSHA is an essential part of probabilistic seismic risk analysis (PSRA) in the performance-based earthquake engineering (PBEE) framework (Cornell *et al.*, 2002; McGuire, 2004; Goulet *et al.*, 2007; Ruiz-Garcia and Miranda, 2007). The objective of advanced engineering analyses is to quantify the extent of inelastic seismic demand (e.g. maximum inter-story drift) and consequence (e.g. building damage cost and indirect cost related to business down time) caused by extreme ground motions probabilistically. Such analyses are facilitated by the use of a so-called fragility curve, which is the conditional probability distribution of a seismic performance measure as a function of seismic intensity level; in risk analysis, fragility functions are integrated over all possible earthquake scenarios. The output is usually expressed as a seismic risk curve, and can be used for facilitating informed decision-making related to seismic risk mitigation (Goda and Hong, 2006; Goulet *et al.*, 2007). The concerted efforts of developing seismic hazard and fragility models for various seismic regions and structural types are one of the significant advancements in recent years.

PSHA can be combined with geotechnical techniques, such as the stress-based liquefaction potential analysis of Seed and Idriss (1971). In the Seed and Idriss stress-based method, the required input information is the expected PGA and moment magnitude (both of which are available from PSHA). With such information, the likelihood of liquefaction occurrence/triggering can be evaluated in probabilistic terms (based on contributions of various earthquake scenarios to the overall seismic hazard). By integrating PSHA with liquefaction assessment, a new engineering tool for probabilistic liquefaction hazard analysis (PLHA), which is useful for rational decision making in geotechnical applications, can be developed (Kramer and Mayfield, 2007; Juang *et al.*, 2008; Goda *et al.*, 2011). Furthermore, additional model components, such as a prediction model for ground settlement

due to earthquakes (Cetin *et al.*, 2009), can be incorporated into the assessment to quantify the extent of geotechnical consequences probabilistically (similar extension from PSHA to PSRA). A key fact is that PSHA produces the fundamental input information (i.e. seismic hazard curve or earthquake scenarios) to such advanced analyses.

This chapter gives an overview of the simulation-based PSHA methodology by summarising a step-by-step analysis procedure and current developments/issues in this area. The use of the simulation-based approach is highlighted, because it is versatile and extendable to facilitate other advanced earthquake engineering applications. This is particularly important in the context of the PBEE methodology. Then, two example applications/illustrations will be discussed: one is related to liquefaction hazard analysis, while the other is related to seismic risk analysis. The chapter concludes by mentioning future trends and suggested improvements in PSHA.

1.2 Simulation-based probabilistic seismic hazard analysis (PSHA)

1.2.1 Methodology

The main objective of this section is to introduce a basic PSHA methodology to a broader audience who is not familiar with PSHA. In particular, an emphasis is given to a Monte Carlo simulation approach. To facilitate this, a case study for western Canada is used as an illustration in the following.

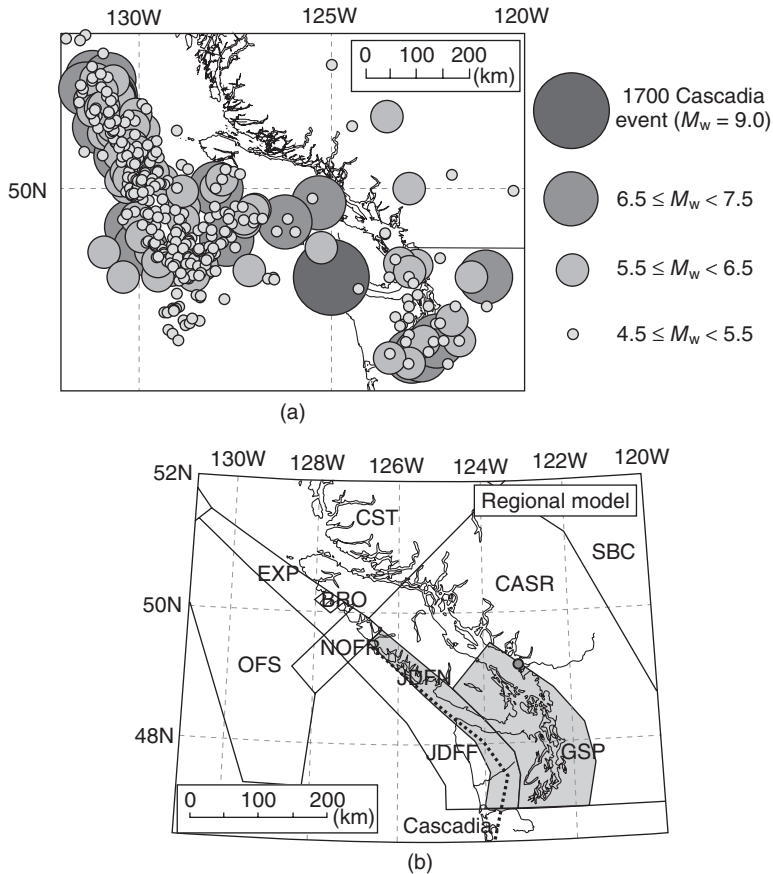
PSHA evaluates the expected seismic intensities corresponding to different probability levels. A rational approach to achieve this is to take all possible scenarios into account, and sum the effects in the assessment by reflecting their relative contributions. Based on Total Probability Theorem (and some assumptions/simplifications), the overall integration/summation process can be decomposed into several modules. Mathematically, the annual rate that a specific ground motion level is exceeded $v_{GM}(\geq gm)$ is given by:

$$v_{GM}(\geq gm) = \sum_{\text{model}, i=1}^p w_i \sum_{\text{zone}, j=1}^q \left[\lambda_{M,ij} \iint_{\Omega_{m,r,ij}} P(GM \geq gm|m, r) f_{M,R,ij}(m, r) dm dr \right], \quad [1.1]$$

where the subscript indices i and j are for the model/assumption (1 to p) and for the source zone (1 to q), respectively; w_i is the weight of a model/assumption i (note: w_i sums to 1); $\lambda_{M,ij}$ is the annual occurrence rate of potentially damaging earthquakes (e.g. events with magnitudes greater than 4.5); $P(GM \geq gm|m, r)$ is the probability of ground motion parameter

exceeding a level gm for a given scenario specified by magnitude m and distance r ; $f_{M,R,ij}$ and $\Omega_{m,r,ij}$ are the probability density function of an earthquake scenario with m and r and the scenario domain (for integration), respectively. In Equation [1.1], the first summation is related to alternative models and assumptions, while the second summation is related to multiple source zones in space (such that all major earthquake sources are included in the calculation). It is noteworthy that the use of discrete models in Equation [1.1] is for convenience only (e.g. logic tree approach); they can be replaced by the continuous probability distributions for more general cases.

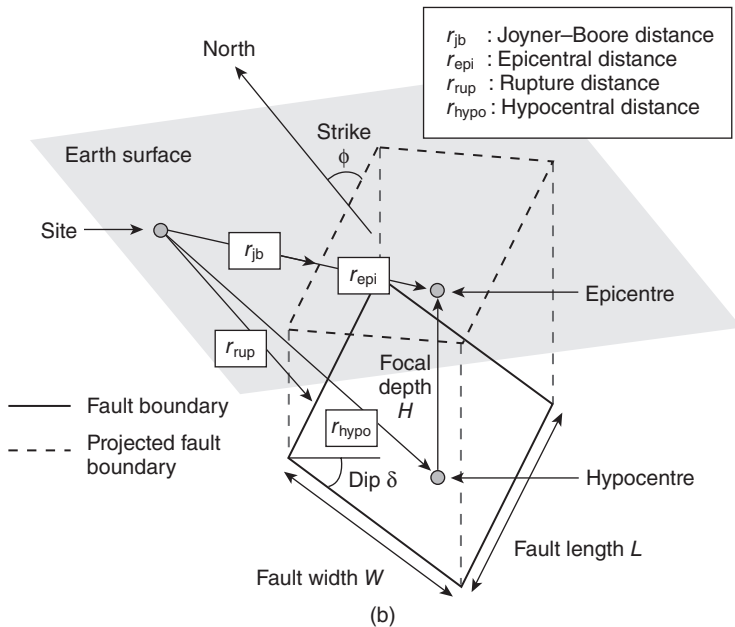
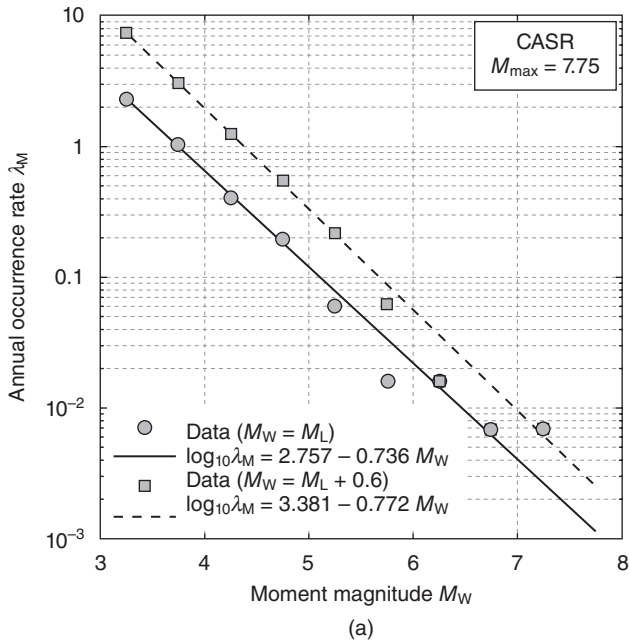
In module 1 (Fig. 1.1), seismic sources are determined based on historical and recent regional seismic activities (i.e. earthquake catalogue) and geological and paleoseismic knowledge. An example of seismic activities and delineated source zones for western Canada (Adams and Halchuk, 2003) is presented in Fig. 1.2. It is noted that the Cascadia subduction zone is represented by a fault source, while other activities are modelled by areal sources. The magnitude of the Cascadia subduction event can be as large as $M_w 9.0$, and the fault rupture zone stretches over 1000 km off south-western British Columbia (B.C.), Washington, Oregon, and northern California; this event can pose significant threat due to both intense ground shaking and tsunamis to cities and towns along the coast. For each identified seismic source, temporal occurrence and potential earthquake size are then characterised. A popular approach for well-defined fault systems is the use of slip rate and characteristic earthquake by utilising geodetic and geological information. For the Cascadia zone, tsunami records in Japan and turbidite histories off-shore of B.C. and Washington provide estimates of the recurrence period and earthquake size (Satake *et al.*, 2003; Goldfinger *et al.*, 2008). For well-studied faults, time-dependent earthquake occurrence models, such as lognormal or inverse-Gaussian renewal model, may be more suitable than a time-independent Poisson model (Goda and Hong, 2006). By contrast, the temporal and size characteristics for areal source zones are best modelled by the Gutenberg–Richter relationship, which describes annual frequency of earthquakes with magnitudes greater than a specified value. An illustration of the Gutenberg–Richter model is shown in Fig. 1.3a; in the figure, two sets of recurrence statistics and fitted relationships are shown, corresponding to two possible cases with different moment-to-local magnitude ($M_w - M_L$) conversions (see Atkinson and McCartney, 2005, for details). Using Monte Carlo simulation, synthetic scenario earthquakes are generated by drawing random samples from the assumed PSHA model components. In the first step, a synthetic earthquake catalogue is constructed, which typically includes occurrence time, location, focal depth, and earthquake size. The fault plane parameters that are needed for each earthquake to fully evaluate recent GMPEs include fault length L , fault



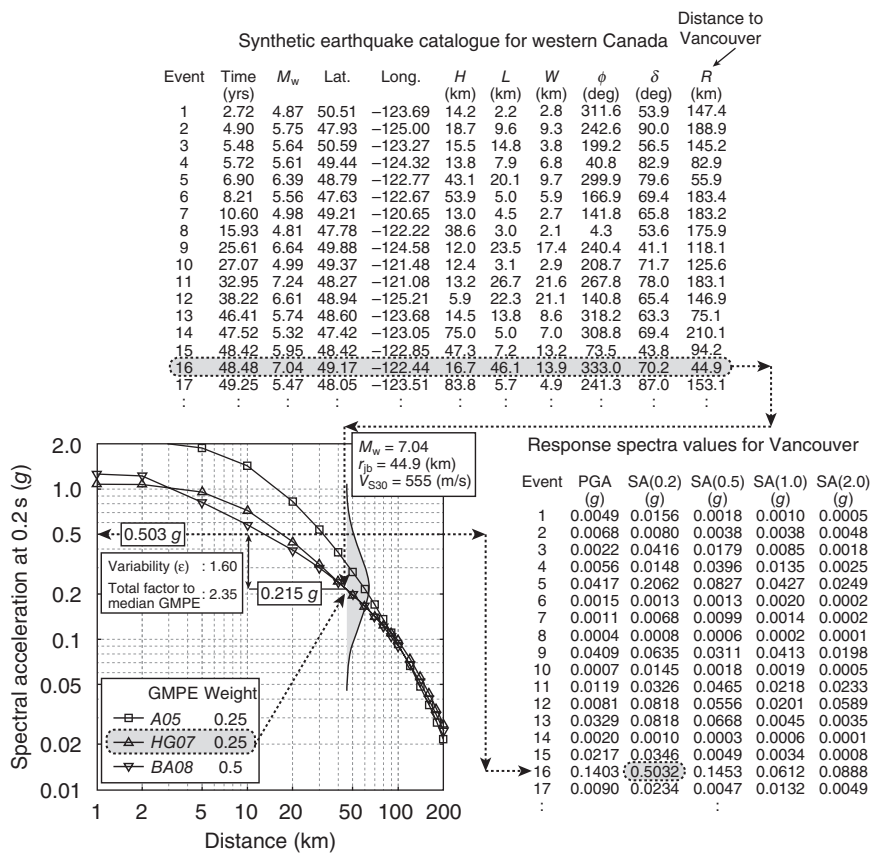
1.2 (a) Seismic activities in western Canada, and (b) delineated regional seismic source zones that were adopted in the national seismic hazard model (BRO: Brooks Peninsula; CASR: Cascade Mountains; CST: Coastal; EXP: Explorer Plate Bending; GSP: Georgia Strait/Puget Sound; JDFN: Juan de Fuca Plate Bending Onshore; JOFF: Juan de Fuca Plate Bending Offshore; OFS: Offshore; SBC: Southeast Canada Background) (Adams and Halchuk, 2003).

width W , strike ϕ , and dip δ for a given earthquake magnitude, earthquake type, and depth (see Fig. 1.3b for a typical finite-fault plane model). Appropriate model parameters can be generated using Monte Carlo simulation (Scherbaum *et al.*, 2004).

Once a synthetic earthquake catalogue has been generated, ground motion parameters can be evaluated for each seismic event contained in the catalogue. An illustration of a sample synthetic earthquake catalogue and evaluation of a GMPE for a given scenario is shown in Fig. 1.4. In this



1.3 (a) Gutenberg-Richter model for the seismic source zone CASR, and (b) finite-fault model of an extended earthquake source.



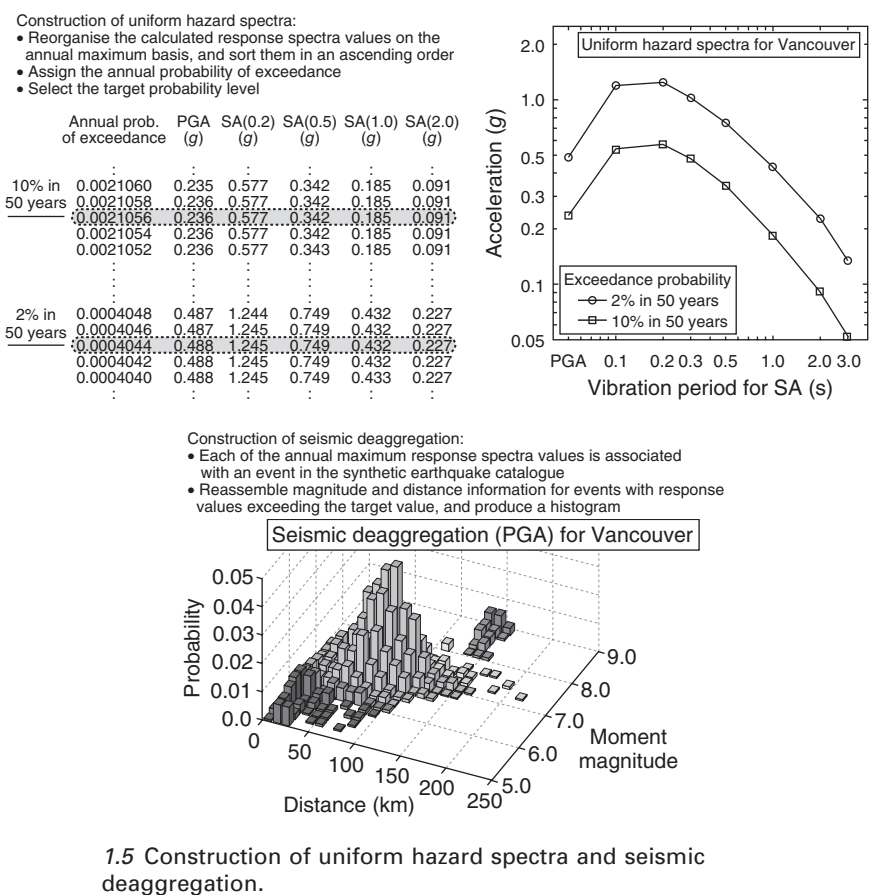
1.4 Evaluation process of predicted ground motion parameters using a synthetic earthquake catalogue and multiple GMPEs.

step, one typically uses multiple GMPEs deemed applicable to the region/site of interest. The use of alternative GMPEs is a popular approach to considering epistemic uncertainties in the correct median motions for given event scenarios (Atkinson and Goda, 2011), though it is not necessarily the best approach, as will be discussed later. By applying GMPEs to all events in the synthetic catalogue, a long (hypothetical) history of ground motion intensity estimates is obtained. Finally, the calculated values are reorganised to develop a list of the annual maximum ground motion parameters in an ascending order (note: for each event, occurrence time should be kept in the synthetic catalogue, so that the annual maximum values can be identified). The seismic hazard curve is simply a plot of the sorted annual maximum ground motion parameter as a function of exceedance probability; mean

fractiles of the ground motion parameters at various probability levels can be easily extracted to construct uniform hazard spectra.

A significant advantage of using Monte Carlo simulation rather than numerical integration to perform the hazard analysis is that it facilitates the production of seismic deaggregation results at any probability level (i.e. to show hazard contributions by magnitude and distance), because all information that is needed for seismic deaggregation is kept throughout the analysis. The process of constructing uniform hazard spectra and deaggregation results is illustrated in Fig. 1.5.

A further advantage of the simulation-based approach over conventional numerical integration is transparency of the procedure. The calculation steps are straightforward and intuitive: simulating earthquake time and location, generating earthquake magnitude and fault dimensions, and then estimating ground motion parameters for given earthquake characteristics.



1.5 Construction of uniform hazard spectra and seismic deaggregation.

Simple statistical data analysis is followed to develop seismic hazard curves and uniform hazard spectra. Through multiple repetitions of this process, additional uncertainties regarding different source models, uncertain parameters of the Gutenberg–Richter relationships, and choice of GMPES can be easily incorporated. In numerical integration, by contrast, inclusion of various uncertainties and probabilistic models can be problematic, because the number of combinations in a logic tree expands exponentially and discretisation of non-Gaussian probability distributions can be troublesome.

1.2.2 Simulation procedure

To demonstrate how the above-mentioned simulation-based PSHA works, let us consider a simple case for western Canada. The location of interest is Vancouver and the site condition of interest is firm soil (NEHRP site class C and reference ground condition for national seismic hazard maps in Canada). The seismic source zone CASR, shown in Fig. 1.2b, is considered (see Fig. 1.3a for the Gutenberg–Richter relationship of CASR). The ground motion parameter of interest is SA at 0.2 s.

The first step is to generate the occurrence times of events in CASR. This can be done by calculating the annual occurrence rate λ_M for CASR based on the Gutenberg–Richter relationship with magnitude bounds:

$$\lambda_M = N_0 \frac{\exp(-\beta M_{\min}) - \exp(-\beta M_{\max})}{1 - \exp(-\beta M_{\max})}, \quad [1.2]$$

where $M_{\min}$ is the minimum magnitude (and is taken to be 4.75); and N_0 and β are the Gutenberg–Richter parameters and are equal to 571.5 and 1.695 for CASR, respectively ($M_w = M_L$ case; see Fig. 1.3a). The calculated value of λ_M is 0.1813. This means that there is one event (of $M > M_{\min}$) within CASR (over entire area) in every 5.52 years. Taking the calculated occurrence rate as a Poissonian rate, an inter-arrival time to the next event can be simulated as:

$$t = -\ln(1-u)/\lambda_M, \quad [1.3]$$

where u is a sample from the standard uniform distribution (i.e. a random number between 0 and 1). For example, considering $u = 0.3128$, the inter-arrival time is obtained as 2.07 years. By considering that the occurrence time of the previous event in CASR is 46.41 years (13th event in Fig. 1.4), the occurrence time of the next event in CASR is 48.48 years (16th event in Fig. 1.4). This process can be repeated to generate the occurrence time for the subsequent events in CASR (and afterwards sequentially).

The next step is to generate a random location of an epicentre within CASR. Assuming uniform likelihood of events within a source zone, a

rejection method can be used by simply generating x and y locations of an epicentre (strictly, spherical coordinates, latitude and longitude, should be converted to plane coordinates, when random samples for a location is generated), and by accepting (or rejecting) a candidate location, depending on whether it is inside (or outside) the source zone. For instance, by sampling an epicentre location for the 16th event (which is highlighted in Fig. 1.4), [latitude, longitude] = [49.17, -122.44] is obtained. At the same time, the focal depth can be generated. For example, by considering the focal depth as a normal variate (Scherbaum *et al.*, 2004), a depth for the 16th event is obtained as 16.7 km. The epicentral and hypocentral distances are calculated as $r_{\text{epi}} = 55.64$ km and $r_{\text{hypo}} = 58.11$ km, respectively.

Subsequently, the earthquake magnitude can be simulated by using the Gutenberg–Richter relationship as:

$$m = -\ln\{\exp(-\beta M_{\min}) - (1-u) \times [\exp(-\beta M_{\min}) - \exp(-\beta M_{\max})]\} / \beta. \quad [1.4]$$

By generating a uniform random sample u , the moment magnitude of the 16th event in Fig. 1.4 is obtained as 7.04.

The above steps are the standard procedure. It is noted that the generated source location corresponds to the location of the epicentre (thus epicentral distance and hypocentral distance can be calculated). By contrast, in modern GMPEs, distance measures based on extended fault plane models are often required (e.g. shortest horizontal distance to projected rupture plane, Joyner–Boore distance r_{jb} , and shortest rupture plane distance r_{rup} ; see Fig. 1.3b). A direct method to compute an extended-source distance measure is to generate fault plane parameters, such as fault length L , fault width W , strike ϕ , and dip δ using their probabilistic information (Scherbaum *et al.*, 2004), and then to calculate the distance exactly.

Alternatively, one can adopt a probabilistic conversion equation from a point-source distance measure to an extended-source distance measure (Goda *et al.*, 2010). For example, by taking the direct method, a simulated fault plane for the 16th event (Fig. 1.4) can be characterised by $L = 46.1$ km, $W = 13.9$ km, $\phi = 333.0$, and $\delta = 70.2$. Based on this information, the shortest distances to surface projection of fault plane is obtained as $r_{\text{jb}} = 44.9$ km.

Finally, a value of SA at 0.2 s for a given earthquake scenario [M_w , r_{jb} , V_{S30}] can be calculated by randomly selecting a ground motion model based on assigned weights (this case, a ground motion model HG07 was chosen; see Atkinson and Goda (2011) for details) and by sampling random variability of the selected equation (this case, $\varepsilon = 1.60$). V_{S30} is the average shear wave velocity in the top 30 m from ground surface and one of the most popular surrogate measures for representing a local soil condition. The predicted SA at 0.2 s is computed as 0.5032 g .

The above process needs to be repeated many times so that sufficient samples are generated to determine fractile values at probability levels of

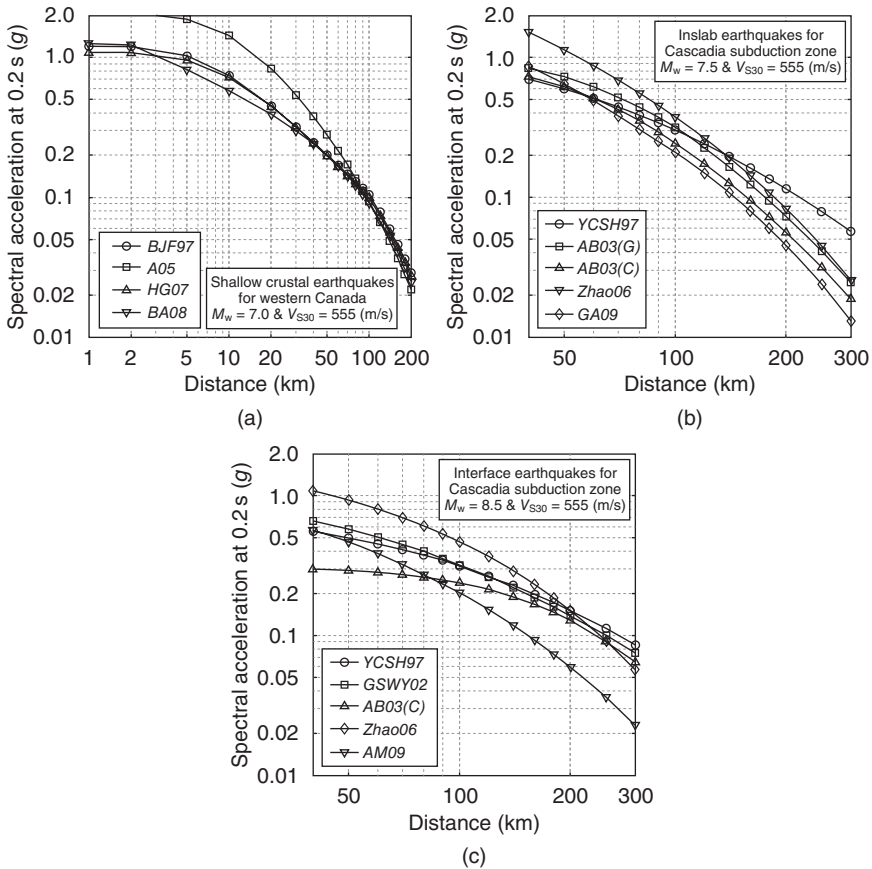
interest. The procedure can be easily adjusted to include multiple source zones. It is noteworthy that in repeating the subsequent simulation cycles, different/alternative parameters/models can be considered as a part of epistemic uncertainty.

1.2.3 Current issues in modern PSHA

PSHA results are sensitive to model components/assumptions. Therefore, any seismic hazard assessment should be scrutinised by testing various cases regarding key model components. For many applications, the choice and weighting of the GMPEs, including the assigned aleatory uncertainty, are the most critical of the input parameters. This should not be surprising, as the GMPEs control the ground motions at the site for every earthquake considered; however, the GMPEs and their uncertainty often do not receive sufficient scrutiny in PSHA. Moreover, as discussed in Section 1.4, common practices regarding the use of multiple GMPEs to represent epistemic uncertainty, along with the use of regression statistics to define aleatory uncertainty, should be improved. Another significant impact is the spatio-temporal characterisation of seismic activities and seismotectonic features, which controls the rates of seismicity in and around the site (Beauval *et al.*, 2006; Atkinson and Goda, 2011). These characterisations may be particularly uncertain in low-to-moderate seismicity regions. Other less critical issues requiring careful consideration include ensuring internally-consistent definitions for various variables in PSHA, such as magnitude, distance, and orientation of ground motion parameters.

To illustrate some of the above-mentioned issues, GMPEs used for assessing seismic hazard in western Canada are compared in Fig. 1.6 (see Atkinson and Goda, 2011, for more details). Figure 1.6 shows significant differences of the predicted SA at 0.2 s for the interface and inslab events; attenuation characteristics over distance differ significantly and variability is large at short-distance range (where empirical data are scarce). These differences are caused by several factors, such as different ground motion datasets, adopted functional forms, and employed approaches (e.g. regression analysis of empirical data versus simulation of ground motions). Uncertainties are particularly large when ground motions are required for types of seismic events that have not yet been recorded (e.g. Cascadia subduction events) and when empirical data coverage is poor in the magnitude–distance ranges of engineering interest (e.g. as is often the case in low-seismicity regions, where instrumentation tends to provide only sparse coverage).

It should be emphasised that the controlling factors for PSHA results depend significantly on location, site condition, and probability level of interest. Thus it is important to conduct a full range of sensitivity analyses as a part of PSHA. A sensitivity analysis may be even more important than



1.6 Comparison of several GMPEs for western Canada: (a) crustal events, (b) inslab events, and (c) interface events (see Atkinson and Goda (2011) for details).

an uncertainty analysis, in that it provides insight into the critical assumptions that drive the results. Note that the effects of changing seismicity models and GMPEs on seismic hazard estimates are inter-connected; for instance, a change of spatial distribution of seismic activities affects scenario characteristics and their contributions to the overall hazard, and thus the impact of alternative GMPEs may depend on the seismicity model characteristics.

There are several pending issues to be addressed/incorporated in the PSHA methodology (this is not a comprehensive list): (i) characterisation of regional site amplification factors using a proxy measure beyond V_{S30} (Hunter *et al.*, 2010; Cadet *et al.*, 2011), (ii) improvement of GMPEs (including better characterisation of magnitude scaling for both small and large events, and more comprehensive treatment of regional site effects), and (iii)

quantification of epistemic uncertainty. To improve the prediction accuracy of site effects, the fundamental frequency of a site can be employed, which renders supplementary information in addition to near surface average shear wave velocity. Such an additional measure tends to reduce variability associated with GMPEs (Cadet *et al.*, 2011), thus resulting in reduction of overall uncertainty in PSHA. A challenging issue is the appropriate treatment of epistemic uncertainty. Epistemic uncertainty, by definition, stems from incomplete knowledge/understanding of the subject matter. Therefore, it is not trivial to confidently capture epistemic uncertainty, and events or their consequences may not be foreseen (e.g. as in the tragic 11 March 2011 M_w 9.0 Tohoku earthquake). Modelling of epistemic uncertainty requires a synthesis of theories and available data from various fields, and imagination to blend model components to evaluate possible scenarios.

1.3 Extension of probabilistic seismic hazard analysis (PSHA) to advanced earthquake engineering analyses

This section presents two illustrative applications regarding extension of PSHA to other earthquake engineering analyses. Specifically, PLHA and PSRA are discussed in Sections 1.3.1 and 1.3.2, respectively.

1.3.1 PLHA

Ground failure due to large earthquakes has disastrous consequences on structures and infrastructure, particularly building foundations and buried pipes. One of the major ground failure phenomena is soil liquefaction, causing lateral spreading of slope, damage to pile-foundation, and failure of retaining wall. Seismic loss due to liquefaction can be extremely high (e.g. sliding of residential houses on slope and tilting of buildings result in complete demolition of structures). A current trend in the popular simplified method, which was originally proposed by Seed and Idriss (1971) as a deterministic procedure, is to carry out probabilistic assessment of liquefaction triggering potential at a site of interest by taking uncertainties of multiple earthquake scenarios and ground motions into account. This is particularly motivated by the adoption of uniform hazard spectra as a seismic design tool. Conventionally, two key input parameters for liquefaction triggering analysis are PGA and moment magnitude. From PSHA, PGA values are obtained as uniform hazard spectra ordinates. The PGA value at a selected probability level is not governed by a single scenario, rather by multiple scenarios. Therefore, a one-to-one relationship between PGA from PSHA and moment magnitude is not possible to establish. Several methods have been developed to deal with this problem by

retrieving the relationship between PGA and moment magnitude (more specifically, their joint distribution) through seismic deaggregation results (Kramer and Mayfield, 2007; Juang *et al.*, 2008). Alternatively, using Monte Carlo simulation, a simpler method can be devised by conducting probabilistic liquefaction triggering assessment for individual earthquakes included in a synthetic catalogue (Goda *et al.*, 2011). A brief summary of the latter approach based on the Standard Penetration Test (SPT) data is given in the next paragraph, and an example case study of PLHA is discussed subsequently.

The Seed–Idriss simplified method based on the SPT data is capable of predicting the occurrence of liquefaction triggering by comparing cyclic resistance ratio with cyclic stress ratio at a specific depth of a soil column. One of the recent probabilistic models for liquefaction triggering is the one developed by Cetin *et al.* (2004), which takes key uncertainties associated with the input data/parameters and the adopted model into account. Cetin *et al.*'s model computes the probability of liquefaction initiation P_L at a depth of interest as:

$$P_L = \Phi \left(- \frac{N_{1,60}(1 + 0.004FC) - 13.32 \ln(CSR_{eq}) - 29.53 \ln(M_w) - 3.70 \ln(\sigma'_v/P_a) + 0.05FC + 16.85}{2.7} \right), \quad [1.5]$$

where Φ is the standard normal function; $N_{1,60}$ is the corrected SPT blow count (but not adjusted for fines content); FC is the fines content (in percentage); CSR_{eq} is the cyclic stress ratio (but not adjusted for moment magnitude); σ'_v is the vertical effective stress; and P_a is the atmospheric pressure ($= 100$ kPa). The correction of the *in-situ* N values to $N_{1,60}$ can be done by using a set of correction factors, as suggested by Youd *et al.* (2001) and Cetin *et al.* (2004). The value of CSR_{eq} for a given depth d is calculated by:

$$CSR_{eq} = 0.65 \frac{PGA}{g} \frac{\sigma_v}{\sigma'_v} r_d, \quad [1.6]$$

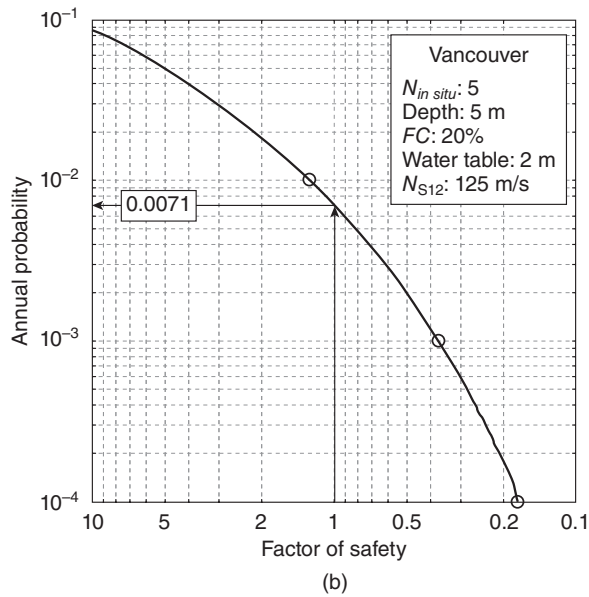
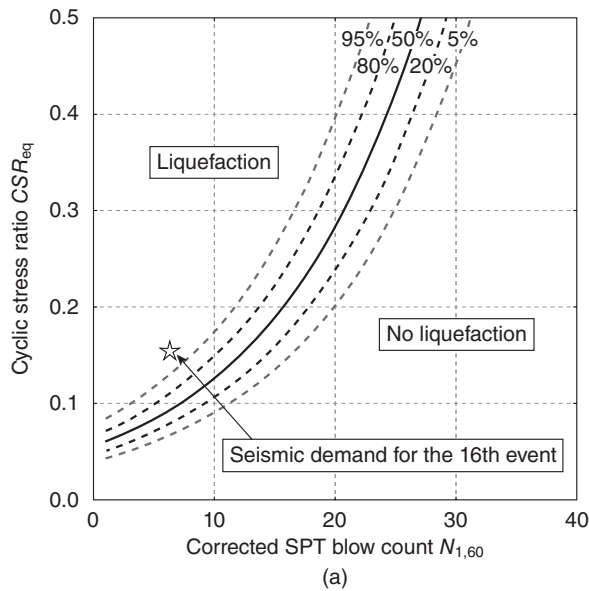
where PGA is the geometric mean of PGAs of two horizontal components at ground surface (Youd *et al.*, 2001); σ_v is the total vertical stress; and r_d is the nonlinear stress reduction factor. Cetin *et al.* (2004) conducted extensive site response analyses to derive a new probabilistic relation for r_d (note: the calculation of r_d requires the average shear wave velocity in the top 12 m V_{S12} in addition to PGA and M_w ; see Cetin *et al.*, 2004, for details). Using Equations [1.5] and [1.6], a value of P_L at a depth d can be evaluated for a given scenario (i.e. a pair of PGA and M_w). It is noted that the cyclic resistance ratio (CRR_{eq}) for a given P_L can be obtained as:

$$CRR_{eq}(P_L) = \exp \left[- \frac{N_{1,60}(1 + 0.004FC) - 29.53 \ln(M_w) - 3.70 \ln(\sigma'_v/P_a) + 0.05FC + 16.85 + 2.7\Phi^{-1}(P_L)}{13.32} \right]. \quad [1.7]$$

Therefore, a conventional factor of safety, $F_s(P_L) = CRR_{eq}(P_L)/CSR_{eq}$, can be evaluated for a selected value of P_L , and a liquefaction hazard curve based on $F_s(P_L)$ can be developed. The above assessment can be extended to multiple soil layers near ground surface and thus the thickness of liquefied soil layers can be evaluated, which is useful for pile-foundation stability analysis. By repeating the above analysis for individual earthquakes included in a synthetic catalogue, liquefaction hazard potential at a site over a certain period can be assessed. Similarly, a liquefaction hazard curve based on the factor of safety shown in Equation [1.7] can be developed (see Goda *et al.*, 2011, for other applications). The probabilistic assessment of P_L is an important step towards performance-based geotechnical earthquake engineering; such information can be utilised to make decisions on ground improvement to mitigate the liquefaction-related risk of structures.

To illustrate an application of PLHA, consider a sand soil layer (with $FC = 20\%$) that is 5 m below ground surface. The water table is 2 m below ground surface. The total vertical stress σ_v and vertical effective stress σ'_v are 100 and 70 kPa, respectively. The *in-situ* SPT blow count is $N = 5$ and $V_{S12} = 125$ m/s; with the correction factors, $N_{1,60} = 6.3$. This is a typical soil layer encountered in the Vancouver area of B.C. (City of Richmond). The site is subjected to an earthquake scenario of $M_w = 7.04$ and $r_{jb} = 44.9$ km (i.e. 16th event in Fig. 1.4). Assuming that $V_{S30} = 200$ m/s, PGA is calculated as 0.223 g (note: this PGA value is greater than that shown in Fig. 1.4 because of the larger site amplification); subsequently, $r_d = 0.746$ and $CSR_{eq} = 0.154$ (from Equation [1.6]). Finally, for the considered scenario, $P_L = 0.994$ (from Equation [1.5]). The evaluation procedure is shown in Fig. 1.7a, noting that the seismic demand due to this event falls outside of the 95% confidence line.

To further demonstrate PLHA, a liquefaction hazard curve based on the factor of safety with $P_L = 0.5$ is developed by considering the same sample soil and seismic activities. In the assessment, PSHA is conducted to construct a synthetic earthquake and ground motion catalogue for soft soil conditions ($V_{S30} = 200$ m/s), and for each event, the factor of safety (as in Equation [1.7]) is calculated. Based on the long list of the factor of safety values over one million years, samples of the annual maximum factor of safety are identified and then plotted in Fig. 1.7b as a liquefaction hazard curve. The curve provides useful information on the non-exceedance of a specific value of the factor of safety. For instance, one may be interested in



1.7 (a) Comparison of the cyclic stress ratio for the 16th event in a synthetic earthquake catalogue with the cyclic resistance ratios with different confidence levels, and (b) liquefaction hazard curve based on the factor of safety for Vancouver.

knowing the annual occurrence probability of the factor of safety being less than 1.0, which can be readily obtained from the figure.

1.3.2 PSRA

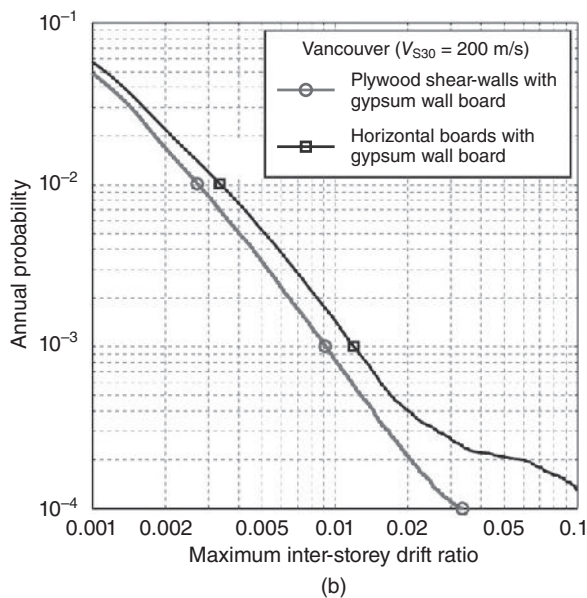
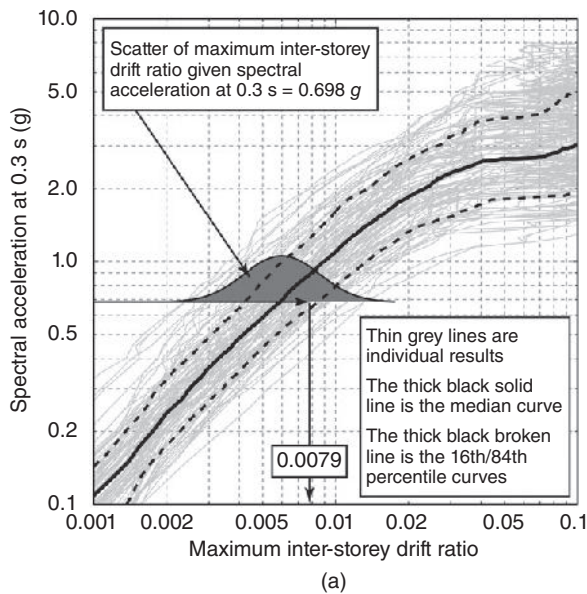
PSRA evaluates the seismic performance of a structure quantitatively and is one of the key building blocks in the PBEE methodology (Cornell *et al.*, 2002; McGuire, 2004; Ruiz-Garcia and Miranda, 2007). The output renders the statistical distribution of an engineering seismic demand/damage quantity, such as the maximum inter-story drift over building height, and can be further utilised to estimate seismic loss, and eventually to make decisions regarding seismic risk mitigation (Goda and Hong, 2006; Goulet *et al.*, 2007).

The essential component for extending PSHA to PSRA is the seismic vulnerability/fragility function, which describes the statistical characteristics of an engineering demand/damage parameter (EDP) for a given ground motion intensity level (e.g. PGA and SAs). Then, the occurrence rate of EDP v_{EDP} exceeding a specified damage level (e.g. maximum inter-story drift ratio of 0.03 for moderate damage) is given by:

$$v_{\text{EDP}}(\geq edp) = \int_{\Omega_{\text{GM}}} P(\text{EDP} \geq edp | gm) f_{\text{GM}}(gm) dgm, \quad [1.8]$$

where $P(\text{EDP} \geq edp | gm)$ is the vulnerability function; and f_{GM} and Ω_{GM} are the probability density function of a ground motion measure and its domain for integration (note: it is common to define f_{GM} based on the annual maximum quantity so that v_{EDP} is interpreted as the annual exceedance rate), respectively. A plot of v_{EDP} as a function of edp is a seismic risk curve.

Using a simulation-based PSHA approach, it is straightforward to carry out numerical evaluation of Equation [1.8]. For instance, various ground motion parameters for each scenario event in a synthetic catalogue are readily available from PSHA to assess seismic vulnerability of a structural system. Consider that the seismic performance of a conventional wood-frame house in Vancouver needs to be assessed for the 16th scenario event in Fig. 1.4 ($M_w = 7.04$ and $r_{\text{jb}} = 44.9$ km). For such purposes, an existing structural model for typical wooden houses with unblocked plywood shear-walls and gypsum wall board (GWB) interior finish (Ventura *et al.*, 2002; White and Ventura, 2006) can be adopted. For characterising the seismic performance of a house model subjected to various ground motion levels, incremental dynamic analysis can be conducted by taking into account regional features of seismic activities (consistent with PSHA) and by selecting and scaling adequate sets of ground motion records based on conditional mean spectra (Baker, 2011). Such a model has been developed by Goda and Atkinson (2011) by considering the maximum inter-storey drift ratio as EDP and SA at 0.3 s as GM . Figure 1.8a shows the incremental



1.8 (a) Incremental dynamic analysis results for a wood-frame house with plywood shear-walls and GWB interior finish, and (b) seismic risk curve in terms of maximum inter-storey drift ratio for two wood-frame houses located at soft soil sites in Vancouver.

dynamic analysis curves for the wooden house with plywood shear-walls and GWB interior finish; in the figure, individual incremental dynamic analysis results, which correspond to selected and scaled ground motion records, are shown with thin lines with grey colour in background, while the median and 16th/84th percentile curves are shown with thick black lines (solid line for the median and broken lines for the 16th/84th percentiles, respectively). By generating a sample of the maximum inter-storey drift ratio for the scenario (SA at 0.3 s is 0.698 g), the maximum inter-storey drift ratio is obtained as 0.0079 (minor damage) by taking the inverse of the empirical cumulative probability distribution function or the fitted lognormal model. A scatter of the maximum inter-storey drift ratio for the given SA at 0.3 s is highlighted in Fig. 1.8a.

By repeating the same analysis for all scenario earthquakes in a synthetic catalogue, PSRA is performed for two wood-frame houses with different shear-wall features: one is with plywood shear-walls and GWB interior finish, while the other is with horizontal boards and GWB interior finish; the former is more resistant against earthquake loading than the latter (see White and Ventura, 2006, and Goda and Atkinson, 2011). These houses are located at soft soil sites ($V_{s30} = 200$ m/s) in Vancouver. The obtained seismic risk curves in terms of maximum inter-story drift ratio are compared in Fig. 1.8b. The results indicate that the maximum inter-storey drift ratio for the house with horizontal boards is consistently greater than that for the house with plywood shear-walls (which is expected). The difference between these two curves can be interpreted as the benefit that is derived from a stronger lateral force resisting system. Moreover, it is noticed that the slope of the seismic risk curve for the house with horizontal boards tends to become flatter, as the annual occurrence probability level becomes smaller (near 2×10^{-4}); this is because the structural system gradually approaches its ultimate capacity (i.e. near collapse).

1.4 Conclusions and future trends

PSHA is an essential tool for current earthquake engineering practices and performance-based seismic design methodology. It addresses key uncertainties in earthquake occurrence and ground motion, and provides fundamental information on earthquake hazard (i.e. occurrence time, location, magnitude, wave propagation, site effects, and ground motion intensities). The direct outputs from PSHA, such as uniform hazard spectra and seismic hazard deaggregation, facilitate modern seismic design provisions and practices for buildings and infrastructure. Importantly, PSHA can be enhanced by extending its model components (e.g. nonlinear site response analysis, near-fault motions, and aftershock hazard), and can be incorporated into more advanced earthquake engineering analyses, such as liquefaction-

induced damage assessment of geotechnical structures and structural vulnerability assessment due to ground shaking. Moreover, these advanced analyses are an integral part of broader applications, such as decision analysis under uncertainty and cost-benefit analysis for seismic risk mitigation. From the implementation viewpoint, an approach based on Monte Carlo simulation is of advantage over conventional numerical integration, because the former greatly facilitates flexibility and transparency in the assessment. It should be noted that the approach taken (numerical integration or Monte Carlo simulation) is a matter of choice and convenience, and does not affect the outcome of PSHA.

There are several PSHA issues that warrant improvements. Although the concepts of PSHA are sound, its implementation requires considerable experience and judgement to ensure reliable results. More research is needed on how to properly characterise uncertainty and the relationship between its epistemic and aleatory components. The division into these components is somewhat artificial, and does not affect the mean hazard, but needs to be carefully considered to avoid ‘double-counting’ of uncertainty, which can artificially inflate the calculated hazard. For example, a portion of the scatter in GMPEs, as derived from regression analysis, is due to the inclusion of multiple regions in the underlying database, and should not really be treated as aleatory variability for a specific site (Atkinson, 2011). Furthermore, the use of multiple GMPEs is not really an appropriate way to characterise the uncertainty in what will be the mean ground motions at a site for a given scenario. Alternative approaches to uncertainty in GMPEs that examine the epistemic and aleatory components are needed (Atkinson, 2011).

The overall framework for PSHA can be modernised and streamlined by integrating its various components and products more seamlessly. Ultimately, the use of GMPEs to characterise ground motions will give way to direct simulation of time histories for the events contributing to hazard (i.e. simulating a time-history rather than invoking a GMPE). This will remove the need to ‘de-aggregate’ a UHS to determine ‘design events’ for time-history selection and scaling. Other methodological improvements will also develop, facilitating further new applications.

This chapter has illustrated the step-by-step procedures of a simulation-based PSHA approach by using a case study in western Canada. Two illustrative applications for incorporating PSHA into PLHA and PSRA were used to demonstrate the potential of the method for more advanced applications in earthquake disaster management. Simply, any types of analyses can be added to PSHA to conduct probabilistic assessment of buildings and infrastructure subjected to extreme natural hazards (not necessarily limited to ground shaking). For the specific examples discussed in this chapter, these additional components are: liquefaction triggering analysis based on

the Seed–Idriss simplified method for PLHA and seismic vulnerability assessment based on incremental dynamic analysis for PSRA. They can be replaced by any suitable models/methods that achieve seamless interface with PSHA (note: depending on the necessity, PSHA needs to be improved to promote the integration; e.g. PSHA can be carried out for various seismic intensity measures).

It is envisaged that a vast field of applications and extensions is possible. A partial list of future research topics to be explored includes:

- In PSHA, ground motion prediction equations can be replaced by ground motion time-series simulation (e.g. stochastic point-source and finite-fault methods). The benefit of such a direct approach is that input time-series data for nonlinear dynamic analysis can be readily available without selecting and scaling available ground motion records.
- Applications to PSRA of a building can be further extended to multiple buildings and infrastructure systems. For such analyses, consideration of spatially correlated ground motions is the key. Several initial applications have been performed (see for example Chapter 3). Nonetheless, there are many unexplored research topics in this field.
- Extension of PLHA to liquefaction-related damage consequences is an important and open research field. For these purposes, probabilistic models for predicting ground settlement, pile-foundation damage, and slope instability need to be developed (with similar rigorousness to structural vulnerability models).
- Beyond the effects of ground shaking, a multi-hazards approach can be advanced. For instance, if tsunami inundation models are available, they can be incorporated into the framework to take into account both ground shaking and tsunami impact. This will be particularly useful for mega-thrust subduction zones (e.g. Japan, Mexico, and Chile).

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Uncertainties in ground motion prediction in probabilistic seismic hazard analysis (PSHA) of civil infrastructure

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Abstract: Ground-motion prediction equations are a critical element of any probabilistic seismic hazard or risk analysis. The total uncertainty in a typical risk analysis is commonly dominated by the uncertainty associated with ground-motion prediction. Within the fields of engineering seismology and earthquake engineering a lot of attention has been paid to identifying intensity measures that are efficient for predicting response measures, and this efficiency is of great relevance for seismic risk analysis and the development of well-constrained fragility curves. However, the epistemic uncertainties associated with ground motions are still not adequately dealt with, and such uncertainties contribute to the overall predictability of an intensity measure, and hence of an engineering demand parameter. The present chapter provides an overview of the uncertainties that exist within ground-motion prediction and emphasises some of the main components that are not dealt with in a robust manner in current risk analyses. The chapter also highlights recent advancements as well as likely future trends associated with the treatment of uncertainty in ground-motion prediction, with a particular emphasis upon how such uncertainties influence the development of ground-motion models.

Key words: ground-motion prediction equation (GMPE), probabilistic seismic hazard analysis (PSHA), aleatory variability, epistemic uncertainty, parametric, ergodic assumption, input variable uncertainty.

2.1 Introduction

Engineering seismology is a discipline of fundamental importance to earthquake risk analysis. The primary focus of an engineering seismologist is to develop models that provide a characterisation of earthquake-induced ground shaking that is of use as an input for structural analysis. More specifically, an engineering seismologist would most likely view their role as involving the development and application of empirical ground-motion models, as well as the undertaking of probabilistic seismic hazard analyses. The outputs of the latter are used as inputs for site-specific probabilistic risk analyses.

The main challenge in the field of engineering seismology is to find an appropriate balance between scientific rigour and engineering pragmatism and this necessarily means that simplifying assumptions and approximations must be made. The primary purpose of the present chapter is to outline the key features of uncertainty that are associated with ground-motion models as used in seismic hazard and seismic risk analyses today. A secondary objective is to explore some of the shortcomings that are currently associated with these models, as well as to consider some of the recent advancements that have been made in the field of ground-motion modelling that look to try to overcome some of these issues.

In order to achieve these objectives, the chapter starts by explaining ground-motion equations in terms of their role in seismic hazard and risk applications, the different types of ground-motion models that are encountered in practice, and how these models are derived. With this context having been set, the latter parts of the chapter focus upon the consideration of uncertainty in ground-motion models, with a particular emphasis on how epistemic uncertainty influences empirical ground-motion models.

2.2 Explanation of ground-motion prediction equations (GMPEs)

Ground-motion prediction equations (GMPEs) are frequently also referred to as attenuation relationships, empirical ground-motion models, or just ground-motion models. Although various parties argue that the use of one of these terms is more appropriate than others, for practical purposes all of these terms can be taken to mean the same thing. While the use of the term *ground-motion prediction equations* has become more common in recent years, the use of the word prediction has implications that are not helpful for enabling people to understand their true purpose. This point will be discussed in the following section.

Regardless of how we refer to it, a GMPE is essentially a function that takes a number of input parameters that relate to the properties of an earthquake and its spatial relationship with a site, and provides two outputs that together define the distribution of ground-motion values that is to be expected for the considered scenario.

Before discussing this further it is instructive to first consider what we mean by ‘ground-motion’. An earthquake manifests via the radiation of waves that are related to the release of elastic strain energy previously stored in the vicinity of the earthquake source. These waves result in time-dependent variations in the position of points on the Earth’s surface and earthquake-induced ground-motions can therefore be described in terms of the current position of a given point at a given time, or equivalently by the motion of such a point in terms of its velocity or acceleration. In order to understand the characteristics of these motions, we deploy accelero-

graphs that can be thought of as damped single-degree-of-freedom oscillators. When an earthquake occurs, these oscillators respond to the shaking that they experience and we can record their response. The act of responding to the input excitation has the same effect as filtering the original excitation that the oscillator was subjected to. Under the assumption that we know the response characteristics of the oscillator, and that it responds in a linear manner, we can remove the response of the oscillator from the recording in order to recover the original input excitation. This excitation consists of a series of acceleration values recorded at discrete time increments.

For a seismic risk application, we are interested in determining the response of structural or geotechnical systems to earthquake-induced ground-motions and in doing so in as efficient a manner as possible. It is therefore common to characterise the ground-motion by a representative scalar quantity and to then relate this quantity to some measure of the response of the system we are analysing. So, when we refer to ‘ground-motion’ herein, we are talking about this scalar quantity that is chosen to represent the features of the acceleration time-history that we deem to be most relevant for a given application. The discussions that follow in the rest of this chapter are applicable to any scalar quantity that is used to represent the earthquake-induced ground shaking. However, it is the 5% damped spectral acceleration (or pseudo-spectral acceleration) that is far and away the most commonly used scalar quantity within seismic hazard and risk applications.

2.2.1 The role of GMPEs within seismic hazard and risk applications

Ground-motion models provide the connection between the occurrence of earthquakes and the shaking that might be experienced by a structure at a given location. While many people believe that ground-motion models attempt to *predict* values of ground motion, and that these predictions have an associated error, it is better to not view GMPEs in this way. In order for a ground-motion model to be useful within the context of seismic risk analyses, it is crucial that the model be able to provide a prediction of both the expected value of the ground-motion for a given scenario, as well as a prediction of the dispersion of ground-motion values about this expected value. These two parameters, when coupled with the assumption that ground-motion values can be described by a lognormal distribution, fully define the likelihoods that a ground-motion value takes on a particular level given the occurrence of an earthquake.

This is a very important point. Within the context of seismic hazard and risk analysis, the role of a ground-motion model is to define the distribution

of ground-motion values that should be expected for a given earthquake scenario. Commonly, this earthquake scenario is defined in terms of an earthquake of a particular type occurring at a certain location in space. Mathematically, we can state that for a given earthquake scenario, $\mathbf{X}$, the value of ground motion is a random variable, Y , and that this random variable is defined by a particular distribution, $f_Y(y|\mathbf{X}, \theta)$, and the parameters of this distribution, θ .

$$Y \sim f_Y(y|\mathbf{X}, \theta) \quad [2.1]$$

Once the role of a ground-motion model is viewed in this way, our interpretation of the uncertainty in ground-motion prediction changes from one in which a GMPE provides an estimate of the expected level of motion (which has some uncertainty) to one in which the uncertainty relates to the choice of distribution as well as to the uncertainty in the parameters of this distribution. This interpretation is significantly different from the manner in which GMPEs are normally viewed. In the conceptual framework outlined above, the commonly held idea that a new and improved model should have a lower standard deviation becomes meaningless. What should become lower is the uncertainty with which we are able to define the parameters of the distribution.

The uncertainty that is associated with the choice of the distribution used to characterise the ground-motion values is relatively low. For many years it has been assumed that ground-motions are well described by a lognormal distribution, and recent work shows that this assumption cannot be rejected using a comprehensive database (Jayaram & Baker, 2008). The implication is that two parameters are required to define the distribution of ground-motions for a given scenario: (1) the mean and (2) the standard deviation. If, as is normally done, we develop GMPEs in terms of the logarithm of a ground-motion value, then the distribution that characterises values for a particular scenario is the normal distribution, which again has just two parameters: $\theta = \{\mu_{\ln Sa}, \sigma_{\ln Sa}\}$, the mean and standard deviation of the logarithmic ground-motion. The distribution of logarithmic spectral accelerations for a given scenario can therefore be described mathematically as:

$$\ln Sa \sim N(\mu_{\ln Sa}, \sigma_{\ln Sa}^2), \quad [2.2]$$

in which both $\mu_{\ln Sa}$ and $\sigma_{\ln Sa}$ take on specific values for the scenario in question.

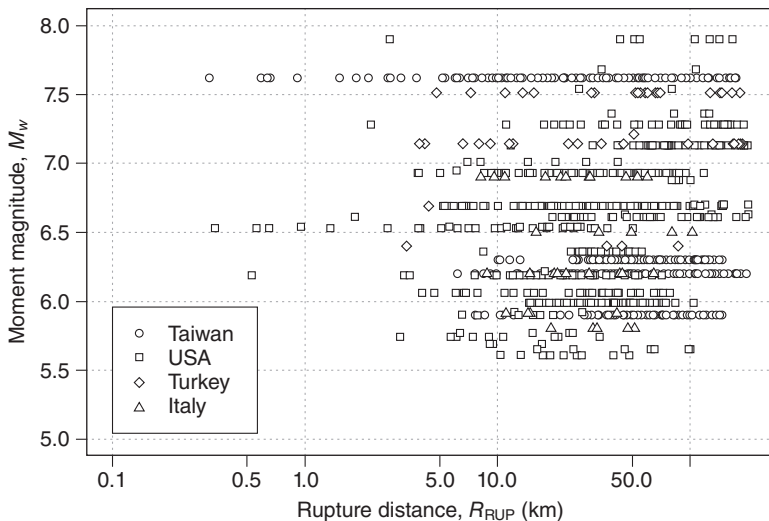
2.2.2 Discussion of types of GMPEs

A very large number of ground-motion models have been developed over the past few decades. Douglas (2011), in summarising these models, has shown that the vast majority of these have been developed for shallow

active crustal regions using empirical approaches. However, such models are not appropriate for application in all tectonic regimes and the purpose of the present section is to briefly outline some of the key features of GMPEs that are encountered in varying regimes.

Models for shallow active crustal regions

Models for spectral accelerations in seismically active regions with shallow crustal seismicity are the most commonly encountered. The main reason for this is that they are able to occur at small source-to-site distances and their recurrence intervals are relatively high in comparison to other regions. Therefore, datasets that are compiled in these regions are relatively rich. However, with the first ever strong-motion recording having been made in the 1933 Long Beach, California, earthquake, the period of instrumental observation is very small in comparison with the recurrence intervals of moderate-to-large earthquakes. In any given region we therefore have just a small sample of the population of ground motions that could be experienced. The implication is that datasets of strong ground-motions are usually very unbalanced for individual regions and comprise recordings from the earthquake scenarios that just happen to have occurred during the period of observation. For example, Fig. 2.1 shows the magnitude-distance distribution of accelerograms (for which finite rupture models existed) that were available to developers within the PEER NGA project (Power *et al.*, 2008). In Fig. 2.1, it is clear that there are scenarios for which no ground-motion recordings have been made. One can also appreciate that where the cover-



2.1 Magnitude–distance distribution of a subset of the NGA database.

age is reasonably dense, this is sometimes the result of combining data from very different regions. This is a common practice.

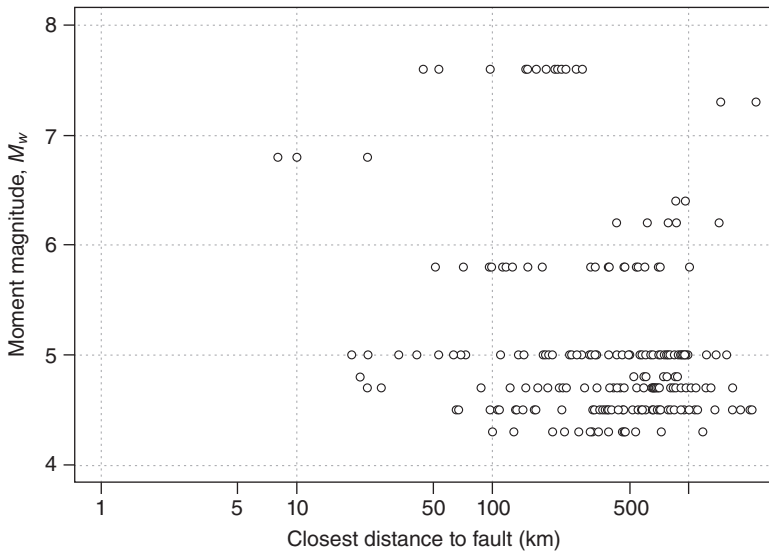
For a hazard or risk analysis one ideally desires a model that provides the distribution of ground motions that would be experienced at *a particular site* from earthquake scenarios in the vicinity of this site. Aside from scenarios corresponding to very small earthquakes, it is generally not possible to develop such a model. As a result, model developers invoke the ergodic assumption in which the lack of recordings that have been made for a particular site is compensated for by assuming that recordings made in similar regions elsewhere are representative. This ergodic assumption is often discussed in terms of replacing a lack of temporal information with spatial information. The assumption hinges upon the equivalence of motions from region to region.

Models for shallow active crustal regions reflect motions from earthquakes occurring in the seismogenic portion of the crust (typically within the uppermost 12–15 km of the crust). These models are also associated with regions in which the active deformation associated with tectonic processes causes the crust through which waves propagate to be highly deformed. This deformation results in heterogeneities within the propagation medium that cause scattering and attenuation of relatively high-frequency components of ground motion. The net result is that high-frequency spectral amplitudes observed in active regions of shallow seismicity tend to be lower than those in stable continental regions. Here, the explanation is given in terms of differences in the propagating medium, but presently this should be interpreted as relating to both the path attenuation and the near surface attenuation (often regarded distinctly as a component of site response).

Models for stable continental regions

Conceptually, the approach that one would adopt when developing ground-motion models for stable continental regions is the same as that for shallow crustal regions. However, the stability of these regions is linked to the fact that they have lower rates of seismicity and hence smaller datasets of strong-motion recordings associated with earthquake scenarios of engineering interest. In fact, while the magnitude-distance distribution shown in Fig. 2.1 features numerous regions where observations are sparse, the situation is far worse in the case of stable continental regions. This issue is illustrated in Fig. 2.2 in which the magnitude-distance distribution of recordings within the database of Atkinson & Boore (2006) is shown.

Additionally, it is clear from Figure 2.2 that there is a very clear bias towards recordings of small magnitude events at relatively large source-to-site distances. This reflects both the fact that the locations of future events



2.2 Magnitude–distance distribution of the database used for the development of the Eastern North American GMPE of Atkinson & Boore (2006).

in stable continental regions are more difficult to predict coupled with the fact that the average inter-station spacing tends to be greater for stable regions. Hence, even when a moderate-to-large magnitude event does occur, it is relatively unlikely that a set of high-quality accelerograms will be recorded at distances of primary engineering interest.

Because the strong-motion datasets are insufficient to constrain empirically derived ground-motion prediction equations in stable regions, the approach taken for active crustal regions cannot be adopted. Instead, one must look to make use of the data that exists from the relatively small magnitude events in order to constrain a physically motivated model for the scaling of spectral amplitudes. While there are detailed seismological models that could be employed for this purpose, for engineering applications the current approach is to implement simplified spectral representations of the earthquake process and wave propagation. These spectral models, more commonly referred to as stochastic-based models (Boore, 2003), are derived by first defining a model for the Fourier amplitude spectrum and then coupling this spectral representation with random vibration theory to either directly estimate the peak response of a damped oscillator, or to simulate time-series from which the peak can be computed.

An important point to note is that these spectral representations relate to far-field shear waves being radiated from a point source only. As a result, the scaling that is commonly implied by these models for large magnitude

events and short source-to-site distances is not consistent with what we would expect. As a consequence, various modifications are required in order to obtain sensible predictions from the near-field scenarios (e.g., Toro, 2002; Boore, 2009).

Given the subject of the present chapter, an important feature of stochastic-based ground-motion models is that the standard deviations that are provided for these models are not directly constrained by empirical data. Instead, the standard deviation is computed by attempting to attribute uncertainties to the various parameters of the underlying spectral model and to then propagate these through the simulation process. While significant efforts have been made in order to make this approach transparent (Toro *et al.*, 1997), the standard deviations that are currently reported for these stochastic models are not regarded as being particularly robust. This is a very important point within the context of hazard and risk assessment.

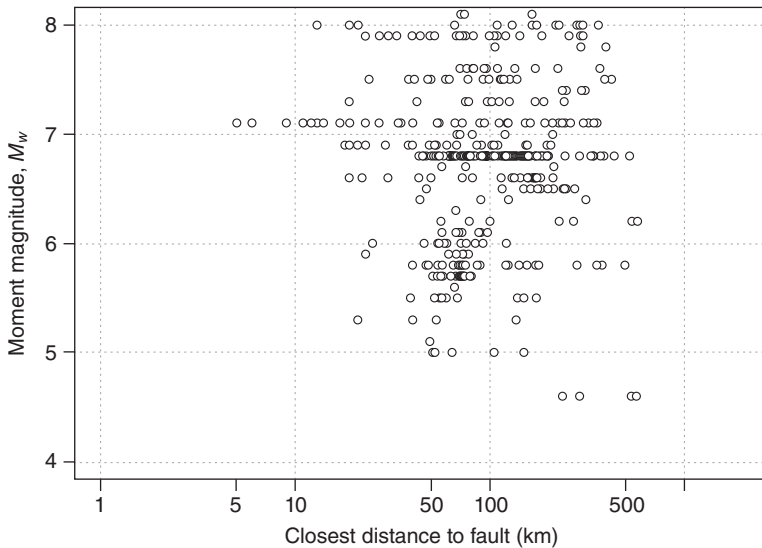
Models for subduction zones

Subduction zones are responsible for the largest earthquakes that we observe, yet despite this far less attention has been devoted to deriving empirical ground-motion models for these types of events. One reason for this is that data from these events is not as readily accessible as it is for shallow crustal events. That is not to say that there are not abundant sources of data regarding subduction zone ground motions, but rather that fewer efforts have been made to compile this data in some routine way and to make it publically available. One possible reason why this is the case is that there is arguably more reason to suspect that there could be regional differences in ground motions coming from different subduction zones. This results from the different geometries that are encountered in the various subduction zones around the world.

When efforts are made to compile datasets for subduction zone events the resulting databases tend to be rather rich in recordings from large magnitude events, particularly in comparison with the datasets compiled for stable continental regions and shallow crustal regions. Figure 2.3 shows the magnitude–distance distribution of the data used for the development of the subduction zone prediction model of Atkinson & Boore (2003). The same axis ranges are used in this figure as in Fig. 2.2 and it is clear that that datasets almost complement each other.

For the subduction zone models that do exist there are three key features that distinguish them from their stable continental and shallow crustal counterparts:

1. Subduction zone models typically include functional terms that account for the depth of the earthquake event.



2.3 Magnitude–distance distribution of the database used for the development of the Subduction zone GMPE of Atkinson & Boore (2003).

2. Subduction zone models must make the distinction between interface events and intraslab events. The interface events are those that occur due to relative slip between the subducting plate and the continental plate above while the intraslab events are associated with ruptures occurring within the subducting slab itself. The nature of these two types of events usually dictates that intraslab events are associated with tensile deformation while the interface events are associated with compressive thrusting.
3. Because subduction events occur at depth, the various functional terms that are required for accounting for near-field effects in crustal models are not required for subduction models.

Ground-motion models for subduction zones are usually based upon datasets of comparable quality as those in shallow crustal regions. In many situations this results from both types of events being recorded on the same networks, e.g., Japan and New Zealand.

2.3 Development of ground-motion prediction equations (GMPEs)

As noted briefly in the previous section, ground-motion models developed for different regions make use of different techniques that reflect the

amount of data available to the developer. In the present section the main options for ground-motion model development are briefly summarised.

2.3.1 Generic types of GMPE

There are three broad classes of ground-motion model: empirically-based models; spectral models; and seismological models. Each of these models can be used to make predictions of the median expected motions from a given scenario. However, there are very large differences in terms of how the aleatory variability and epistemic uncertainty are handled in each case. In each of the following subsections the approach to development of each type of model is outlined.

Empirical models

Empirical models are developed by first compiling large databases of accelerograms as well as metadata that relates to each of these recordings. This metadata comprises information about the magnitude of the causative earthquake, the distance from the source to the site and various other quantities that are used as independent variables within the regression analysis that is subsequently performed. This regression analysis is most commonly conducted using the mathematical representation shown in Equation (2.3). In this equation the total difference that is computed between the observed logarithmic spectral acceleration, $\ln Sa_{ij}$, and the model prediction, $\mu(\mathbf{Z}_{ij}|\theta)$, is partitioned into two components that reflect the earthquake-specific difference between the average motions for the population (for a given scenario) and the motions for that particular event, $\delta_{E,i}$, and the difference between the observed motion and the event-specific average motion, $\delta_{A,ij}$. In Equation (2.3), the matrix $\mathbf{Z}_{ij} = \{M_{w,i}, R_{RUP,ij}, V_{s30,ij}, \dots\}$ contains all of the metadata relevant to each observation and will typically include the magnitude of the event, a measure of source-to-site distance, and information about the site conditions, among potentially other things. The vector θ contains all of the regression coefficients and are published in tabular form with a particular row or column of this table relating to a given response period.

$$\ln Sa_{ij} = \mu(\mathbf{Z}_{ij}|\theta) + \delta_{E,i} + \delta_{A,ij} \quad [2.3]$$

The model that is published consists of two components, as previously mentioned. These components are the information about the median prediction and the information about the variance components. In the framework outlined in Equation (2.3), the variance components are effectively (although not exactly) the variances of the $\delta_{E,i}$ and $\delta_{A,ij}$ terms. These terms

are assumed to be independent and hence the total model variance can be computed from the sum of the individual variance components:

$$\text{var}(\ln Sa) \equiv \text{var}(\delta_{E,i}) + \text{var}(\delta_{A,ij}) \quad [2.4]$$

The reason why Equation (2.4) has not been written using an exact equality is that the inter-event and intra-event variances that are described here as $\text{var}(\delta_{E,i})$ and $\text{var}(\delta_{A,ij})$ respectively are not computed from the residuals in standard regression analyses. Rather, the variances are estimated during a maximum likelihood procedure. The numerical results are always very similar to what one obtains from directly computing the variances of the residuals, but there are important differences that are worth keeping in mind.

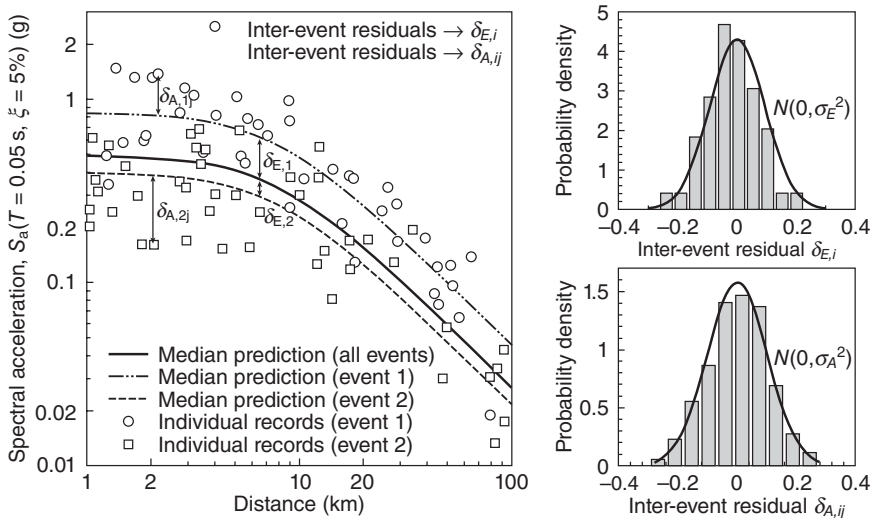
A published ground-motion model will present an equation for the total standard deviation in the form of Equation (2.5).

$$\sigma_{\ln Sa}^2 = \sigma_E^2 + \sigma_A^2 \quad [2.5]$$

This overall process of partitioning the observed variability into components can be seen in the schematic illustration shown in Fig. 2.4.

Stochastic, or spectral, models

A stochastic-based, or spectral, model is based upon the assumption that one can compute estimates of response spectral ordinates by first



2.4 Schematic illustration of the process of partitioning the total observed variability to inter-event and intra-event components.

developing a model for the Fourier amplitude spectrum of ground-motions (typically of displacement or acceleration) and then using this to generate spectral ordinates after application of a transfer function. The most commonly implemented method is that outlined by Boore (2003) in which the Fourier amplitude spectrum corresponding to the response of a damped oscillator, $A(f)$, is described by a series of terms representing the source, $E(f)$, the path, $P(f)$, site effects, $S(f)$, and the transfer function of the damped oscillator, $I(f)$:

$$A(f) = E(f)P(f)S(f)I(f) \quad [2.6]$$

The peak response of the oscillator can then be found from the moments of the spectrum shown in Equation (2.6) via random vibration theory (Boore, 2003).

This formulation of the underlying amplitude spectrum is entirely deterministic and, in the case that one employs random vibration theory, the peak response of the oscillator is also entirely deterministic. Therefore, in order to obtain an estimate of the variance of spectral amplitudes (which is critical for hazard and risk applications) one must make use of an error propagation framework and map the uncertainty that exists in the parameters defining the various terms of Equation (2.6) into the uncertainty in predicted spectral amplitudes.

As demonstrated by Boore (2003), the mathematical equations defining the peak response are highly nonlinear in terms of the underlying parameters that describe the amplitude spectrum (such as the stress parameter, $\Delta\sigma$). The implication is that one can only *estimate*¹ the variance of spectral amplitudes using a framework such as that shown in Equation (2.7) in which X_i represents a random variable for each of the parameters defining the spectrum in Equation (2.6) – which includes the stress parameter, the geometric spreading coefficients, the site kappa value, etc.

$$\sigma_{\ln Sa}^2 = \sum_{i=1}^n \sum_{j=1}^n \rho_{ij} \left(\frac{\partial \ln Sa}{\partial X_i} \right) \left(\frac{\partial \ln Sa}{\partial X_j} \right) \sigma_{X_i} \sigma_{X_j} \quad [2.7]$$

Therefore, rather than using this analytical framework for estimating the variance, model developers instead make use of Monte Carlo simulation strategies to generate realisations of spectral amplitudes for a given set of basic variables.

From the perspective of ground-motion uncertainty, there are some subtle points that must be appreciated when developing these models, and these points are closely linked to why there is not great confidence in

¹ This expression only gives an estimate rather than an exact answer as the distributions of the underlying variables may not be normal and the function for $\ln Sa$ is nonlinear in terms of these variables.

current estimates of standard deviation that have been obtained using this approach. In the vast majority of cases the distributions of the basic variables, such as the stress parameter, or site kappa, are not known. To obtain an estimate of the aleatory variability of ground-motions the σ_{x_i} terms in Equation (2.7), or, more precisely, the entire distribution of the basic variables, should reflect the aleatory variability in these parameters. In addition, the aleatory covariance among the parameters should also be known and used within this framework. However, given that this approach is implemented in data-poor regions, this information is not known *a priori*. The result is that there is significant epistemic uncertainty associated with what the parametric aleatory variability should be. Presently, the approach of model developers is to *assume* what the total uncertainty is in each model parameter and to then further assume what the breakdown is between epistemic and aleatory contributions. It is this subjective assignment of variance components that currently undermines the confidence in estimates of the standard deviations associated with these stochastic-based models.

One final point that is worth raising in terms of the estimates of uncertainty for stochastic models is related to the discussion in Section 2.2.1 concerning the fact that a requirement of a ground-motion model is that it defines the distribution of ground-motion amplitudes for a given earthquake scenario. It is clear from the discussion in the present section that the stochastic-based models are heavily biased toward providing estimates of just one of the two parameters required to define the distribution of ground-motion amplitudes, that being the mean logarithmic motion. For hazard and risk applications, where both parameters of the ground-motion distribution are very influential, the drawbacks associated with the definition of the variance are considerable.

Seismological models

While empirical and spectral models are by far the most commonly used models in hazard and risk applications, seismological simulations have also been used in the past, particularly when scenario events are considered, or when one looks to recreate the effects of a historic event. Seismological models can be broadly grouped into kinematic models and dynamic models. The former relates to those models that prescribe a slip distribution over a rupture surface and then relate these source slip amplitudes to the displacement amplitudes that would be experienced at some other point in space. On the other hand, dynamic models prescribe initial and boundary conditions for a rupture that is allowed to grow in accordance with some physical laws. These dynamic models can either follow deterministic rules or can incorporate stochastic features as well in order to replicate source complexity. Once the source excitation is defined, either kinematically or

dynamically, a detailed model of the propagation medium is required in order to define the manner in which source effects are linked to site effects. This link is achieved via Green's functions (Aki & Richards, 2002) and may be theoretical Green's functions or empirical Green's functions.

Unfortunately, the computational effort that is required to generate simulated wave fields from seismological models at frequencies of relevance for engineering applications are significantly greater than that required for either empirically based models or spectral models. More importantly, we have stated that there is not a great deal of confidence in the standard deviations that are presented by developers of spectral-based models at the present time. This statement also holds for the seismological models, but the assertion is even stronger: these models require a larger number of parameters and it is known that these parameters are strongly coupled with each other. When one performs an inversion to recover, say, a model of the slip distribution from a recorded event one is implicitly obtaining a set of consistently coupled parameter values. However, from a forward modelling perspective one must prescribe the covariance structure (or, more precisely, the joint distribution of parameters) *a priori*. At this point, very little is known about this joint distribution and given the nonlinear dynamic character of the rupture process the implications of not adequately accounting for a source effect, or for the appropriate relationships among variables can have drastic consequences (Strogatz, 1994).

Therefore, while significant advances have been made in fields related to seismological simulation of high-frequency ground motions, we are not yet at a point where these models satisfy the requirements for use within a hazard or risk analysis framework. Given that empirically derived models are by far the most commonly used within hazard and risk applications and that the spectral-based models and seismologically based models have their own (very different) issues related to the modelling of ground-motion uncertainty, the rest of this chapter will focus solely upon empirically derived ground-motion models.

2.3.2 Compilation of empirical strong-motion databases

One of the many decisions that a developer of empirical ground-motion models must make is which records are to be included within a dataset. For a long time this decision was not given a great deal of attention because most modellers simply compiled as many records as they could. However, now that databases of recordings are becoming larger, the decision related to compilation of datasets becomes more important. The reason for this is largely related to the regional composition of data as well as the heterogeneous quality of metadata. As will be discussed more formally in Section 2.4.2, the quality of the metadata directly impacts upon the model variance

that is obtained during the model development. Hence, there is a direct trade-off between the quality of the metadata used in the model development and the value of the standard deviation (and also the median predictions) obtained during the regression procedure. As noted by McGuire (2004), it has long been realised that one is better off working with a smaller number of records with high-quality metadata than to simply use as much data as possible, irrespective of its quality. That said, however, in order to constrain the scaling of ground-motion amplitudes over a broad range of scenarios it is important to have a dataset that is as balanced and well distributed as possible. Here, by balanced the meaning is that the data provides a reasonable coverage over the magnitude–distance space of greatest relevance for hazard and risk applications.

In principle, the fact that different model developers can select a subset of recordings from larger available datasets means that some element of epistemic uncertainty can be accounted for in this way. However, in reality, the datasets that are used for the development of ground-motion models are still relatively sparse and this dictates that there are numerous magnitude–distance combinations where developers do not have the freedom to pick and choose the records they use. At least, if they were to be selective in certain magnitude–distance ranges this would have a strong impact upon their ability to constrain the scaling of their model. This is even more the case when developers look to be selective about which earthquakes should be included, rather than just which recordings are included (see, for example, the numbers of events above magnitude 7.5 in Fig. 2.1).

2.3.3 Mixed-effects regression analysis

On pages 38–9 the total difference between observed ground-motion amplitudes and model predictions was partitioned into two parts, one referred to as the inter-event residual, $\delta_{E,i}$, and one referred to as the intra-event residual, $\delta_{A,ij}$. Presently, within engineering seismology, two main approaches are used in order to partition the total residual into these components: ‘one-stage’ maximum likelihood (or expectation-maximisation) approaches (Brillinger & Preisler, 1984, 1985; Abrahamson & Youngs, 1992; Joyner & Boore, 1993) or ‘two-stage’ approaches (Joyner & Boore, 1993). While both approaches appear to solve the same problem and both provide estimates of the inter-event standard deviation and the intra-event standard deviation there is one very important difference between the two approaches. The problem that is addressed is how to maximise the likelihood function defined in Equation (2.8). In this equation, the term $\mathbf{C}$ is the covariance matrix and it is this matrix that contains the information regarding how events recorded from the same earthquake, or how recordings from different earthquakes made at the same site, are correlated.

$$\ln L = -\frac{N}{2} \ln 2\pi - \frac{1}{2} \ln |\mathbf{C}| - \frac{1}{2} [\ln \mathbf{S}\mathbf{a} - \mu(\mathbf{Z}|\theta)]^T \mathbf{C}^{-1} [\ln \mathbf{S}\mathbf{a} - \mu(\mathbf{Z}|\theta)] \quad [2.8]$$

In the two-stage approach described by Joyner & Boore (1993), the ‘event terms’ $\delta_{E,i}$ are explicitly solved for during the regression analysis. That is, these terms are effectively coefficients of the model with the implication that statistical tests for significance, etc., could be performed on these terms (although this is never done). In contrast, in the one-stage approach (often, although incorrectly, referred to as a random effects approach, Abrahamson & Youngs, 1992) the variance components are estimated directly rather than the individual ‘random effects’. While in the regression algorithm of Abrahamson & Youngs (1992) the random effects (the $\delta_{E,i}$ terms) are computed, these are *a posteriori* estimated values conditioned upon the directly estimated values of the variance components.

When developing ground-motion models it is common to use the different types of residual to evaluate the performance of different parts of the model. For example, the inter-event residual is related to the relative amplitude of a particular earthquake for a given magnitude, depth and style-of-faulting, etc. Therefore, when assessing the performance of a fitted model one would commonly look at a residual plot of the inter-event residuals against magnitude in order to assess the performance of a particular functional form. If one were to observe a systematic trend in the inter-event residuals then one could modify the functional form related to magnitude scaling in order to correct for this. The result would be a reduction of the inter-event variance. However, while this is common practice, it must be kept in mind that the inter-event residuals that are computed are dependent upon the particular regression approach that is taken. This point is best illustrated by considering a poorly-recorded event. In this case, the two-stage regression analysis procedure of Joyner & Boore (1993) would require the event term for this earthquake to be directly estimated, but this term would have a large confidence interval associated with it if the parameter estimate is based on a small number of records. If this confidence interval includes zero then the event-term should be regarded as being statistically insignificant and hence it should be dropped from further consideration. The implication would be that the intra-event residuals for this event would be equal to the total residual and that the event term would be zero. This directly impacts upon the implied scaling of ground-motions with respect to magnitude if this scaling is based upon inspection of event terms. On the other hand, in the one-stage maximum likelihood approach of Abrahamson & Youngs (1992), when an event is not well recorded then the inter-event residual (the random effect) tends towards a constant fraction of the total residual that has been observed. In the extreme, when just one recording

of an event has been made, the total residual, $\delta_{T,ij}$, is partitioned such that the inter-event residual is defined as in Equation (2.9):

$$\delta_{E,i} = \frac{\sigma_E^2}{\sigma_E^2 + \sigma_A^2} \delta_{T,ij} \quad [2.9]$$

Thus far, mixed-effects regression procedures, or variants of this (including the approaches discussed herein), have been implemented widely without necessarily being given the critical evaluation they deserve. For example, more complex decompositions of the total residual into constitutive parts have been proposed in the literature (Chen & Tsai, 2002; Tsai *et al.*, 2006) and a generic framework for referring to these various components has also been proposed (Al Atik *et al.*, 2010). In the approach of Tsai *et al.* (2006), Equation (2.3) would be rewritten as in Equation (2.10) where what was the intra-event residual, $\delta_{A,ij}$, has now been decomposed into a path-specific term, $\delta_{P,ik}$, a site term, $\delta_{S,k}$, and a residual error, ϵ_{ijk} .

$$\ln Sa_{ijk} = \mu(\mathbf{Z}_{ijk}|\theta) + \delta_{E,i} + \delta_{P,ik} + \delta_{S,k} + \epsilon_{ijk} \quad [2.10]$$

In this revised formulation, the indices i , j and k correspond to an event, a recording and a site respectively. Note that the index j is required because of the discrete manner in which different travels paths are defined in practice.

When one works with a large dataset, the decomposition shown by Tsai *et al.* (2006) is preferred over the more commonly implemented approach of only decomposing the within- and between-event components. However, even this approach can be significantly enhanced if one makes use of the full power of mixed-effects modelling procedures. For example, the mathematical framework shown in Equation (2.10) implies that the source, path and site effects all influence the scaling of ground motions in the same way, i.e., they simply shift the median prediction up or down by a constant amount. However, within a mixed-effects framework there is no reason why the random effects cannot be attributed to different model coefficients, such as those contained within the vector θ of Equation (2.10). The only study that appears to have recognised this thus far is that of Chiou & Youngs (2008) in which the nonlinear site response terms include a nonlinear dependence upon a random effect associated with the source.

2.4 Sensitivity of model components

Thus far the discussions have been focused upon the standard deviation of a ground-motion model and how this represents the aleatory variability of the predicted intensity measure. In order to understand the total uncertainty associated with predicted ground-motions we must also give due attention to epistemic uncertainty. With very few exceptions, in hazard and

risk analyses that have been conducted to date epistemic uncertainty in ground-motion predictions has been accommodated by using multiple ground-motion models and assuming that the differences among these encapsulate the epistemic uncertainty. However, there are some obvious issues that are associated with the development process that have been discussed thus far that dictate that individual models also have their own epistemic uncertainty. In the following sections the topic of individual model uncertainty is discussed as well as how this can be dealt with in hazard and risk analyses.

2.4.1 Model-specific epistemic uncertainty

Inspection of Fig. 2.1 revealed that empirical datasets used to develop ground-motion models are not balanced with respect to magnitude-distance scenarios. That is, some scenarios have far more empirical constraint provided by the data than others. This clearly has an implication for the uncertainty that we should place upon the model predictions for different scenarios. However, in current practice it is generally assumed that this uncertainty will be captured via the different predictions that would be obtained through the use of different ground-motion models. Unfortunately, there is no guarantee that this would in fact be the case. For example, it may be that for scenarios for which few recordings have been made, the model developers constrain the scaling of their model using theoretical considerations. If all developers are subscribing to the same underlying theory then it could be the case that the scaling implied by the predictions of a suite of models could be very similar.

In what follows, it is assumed that differences in predictions among different models may arise from the application of different hypotheses about how ground-motions should scale with the primary predictor variables. It is further assumed that these interpretations are all consistent with the available data to at least some extent. In this case, the differences in model predictions would be accounting for epistemic uncertainty. However, there are other factors that influence the uncertainty in a model and these relate to what dataset was used for the regression, what the quality of this dataset is, how the regression was performed, what criteria was used to retain or reject possible functional expressions, etc.

The issue of the quality of the dataset is discussed in Section 2.4.2 concerning the influence of uncertainty of the independent variables. To some extent this section also considers the issue of how the regression analysis is conducted. The issue of what criteria are used to retain or reject functional expressions is referred to in Section 2.4.3 on prediction intervals and this section also deals with how a dataset is compiled in terms of the data coverage as well.

2.4.2 Influence of uncertainty in independent variables

Previously it has been asserted that one is better off working with a smaller high-quality dataset than a larger dataset in which the metadata is not as well constrained. The reason why this is the case is that any uncertainty that is associated with the independent variables gets mapped into apparent aleatory variability during the regression analysis. For example, if one derived a deterministic expression that precisely related two variables to each other via some constant, such as $E = mc^2$, then if one knew the exact value of the mass, m , in this case, then one would also know the exact value of the energy, E . However, if the mass was not known precisely, but was rather a normally distributed random variable with variance σ_m^2 , then the energy would also be a normally distributed random variable with a variance of $\sigma_E^2 = c^4 \sigma_m^2$. If one attempted to derive a relationship between observed energy quantities that were exactly known and estimated mass values using the expression $E_i \sim \alpha m_i + \varepsilon_i$, then one would observe two effects. Firstly, the regression coefficient α would most likely not be equal to its true value of c^2 and secondly one would find that the residual error denoted by ε_i would have a non-zero variance, i.e., it would be required in order to explain the observations. The point here for the context of empirical ground-motion models is that when an independent variable is not known precisely then the computed variance of the model will be an overestimate and the derived coefficients may also be biased.

While it is clear that treating independent variables as though they are exactly known, when in fact they are not, will influence the obtained model for both the median predictions and the standard deviation, this effect is very rarely accounted for. Rhoades (1997) demonstrated that the effect of magnitude uncertainties is not particularly large when a reasonable constraint is available on the catalogue magnitudes. More recently, Gehl *et al.* (2011) have also applied a similar approach in order to account for the uncertainties in site characterisation using the average shear-wave velocity. The basic idea in both of these studies is to partition either the event-specific residual (in the case of Rhoades) or the site-specific residual (in the case of Gehl *et al.*) into two components, one of which represents the difference between the true value of the independent variable (and its influence upon the predicted motions) while the other represents the true residual. For example, if one began with a regression model of the form of Equation (2.10) one could replace the terms $\delta_{E,i}$ and $\delta_{S,k}$ with the new terms given in Equations (2.11) and (2.12).

$$\delta'_{E,i} = \frac{\partial \mu(\mathbf{Z}_{ijk}|\theta)}{\partial M_w} \Delta M_w + \delta_{E,i} \quad [2.11]$$

$$\delta'_{S,k} = \frac{\partial \mu(\mathbf{Z}_{ijk}|\theta)}{\partial V_{s30}} \Delta V_{s30} + \delta_{S,k} \quad [2.12]$$

In Equations (2.11) and (2.12) the terms ΔM_w and ΔV_{s30} represent the difference between the unknown true values of magnitude M_w and average shear-wave velocity V_{s30} and their estimated values used for the regression analysis. The regression equation therefore becomes:

$$\ln Sa_{ijk} = \mu(\mathbf{Z}_{ijk}|\theta) + \delta'_{E,i} + \delta_{P,ik} + \delta'_{S,k} + \varepsilon_{ijk} \quad [2.13]$$

Now, rather than the total variance of the model being defined by Equation (2.14), the variance is given by the expression in Equation (2.15) in which the terms $\sigma_{M_w}^2$ and $\sigma_{V_{s30}}^2$ are the variances of the ΔM_w and ΔV_{s30} values.

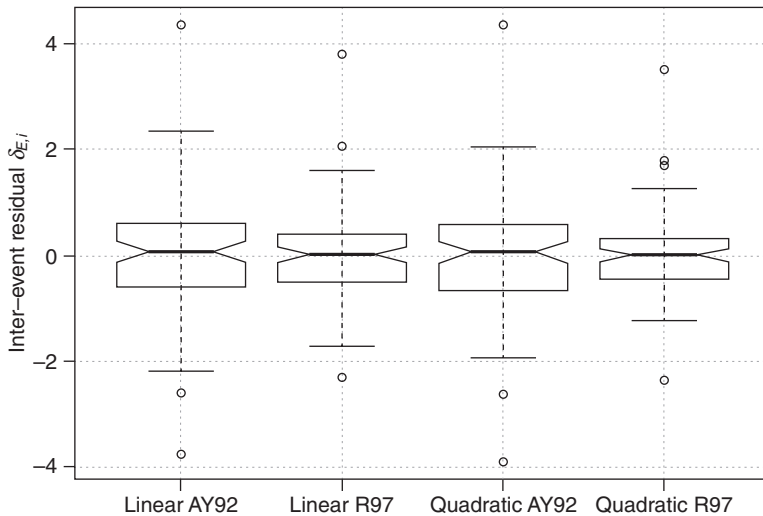
$$\sigma_{\ln Sa}^2 = \sigma_E^2 + \sigma_P^2 + \sigma_S^2 + \sigma_\varepsilon^2 \quad [2.14]$$

$$\sigma_{\ln Sa}^2 = \underbrace{\sigma_E^2 + \left[\frac{\partial \mu(\mathbf{Z}_{ijk}|\theta)}{\partial M_w} \right]^2 \sigma_{M_w}^2 + \sigma_P^2 + \sigma_S^2}_{\sigma_E^2 \text{ from Equation 2.14}} + \underbrace{\left[\frac{\partial \mu(\mathbf{Z}_{ijk}|\theta)}{\partial V_{s30}} \right]^2 \sigma_{V_{s30}}^2}_{\sigma_S^2 \text{ from Equation 2.14}} + \sigma_\varepsilon^2 \quad [2.15]$$

When the uncertainties in the independent variables are known, or at least can be estimated, then the net effect is that the total variance observed from the regression analysis can be partly explained by the propagated errors in these variables. From Equation (2.15) it is apparent that in order for the total $\sigma_{\ln Sa}^2$ to be preserved the estimated values of the inter-event variance and the site-to-site variance must be reduced. It can be demonstrated that a significant component of what is traditionally regarded as being the intra-event variance is actually attributable to the site-to-site variance and that a significant portion of this can again be attributed to the uncertainty associated with site classification (Gehl *et al.*, 2011).

Naturally, the approaches just outlined can also be extended to account for the uncertainties in any input variable. The actual reduction in the standard deviation of the regression model will depend upon the degree of uncertainty in the independent variables. However, one can obtain roughly a 10% reduction in the standard deviation due to uncertainties in each of magnitude, shear-wave velocity or distance (i.e., over 20% total reduction). While this reduction may not seem to be particularly large, a reduction of this magnitude can have a significant impact upon computed hazard and loss curves.

By way of practical example, Fig. 2.5 shows how the computed inter-event standard deviation can vary depending upon whether or not the individual magnitude uncertainties are accounted for. The summary of the residuals presented in this figure correspond to models for Arias intensity developed



2.5 Influence of accounting for magnitude uncertainty during the derivation of a prediction equation for Arias intensity. Here the reduction is expressed by way of comparing the computed inter-event standard deviations using different methods. AY92 stands for Abrahamson & Youngs (1992), R97 stands for Rhoades (1997). The boxplots are generated for inter-event residuals for both linear and quadratic models. The quadratic model residuals correspond to an extension of the method of Rhoades (1997) developed by Stafford (2006).

by Stafford (2006) that either include linear or quadratic magnitude scaling. In the case of the quadratic model while the figure attributes this to Rhoades (1997), the regression method implemented was an extension for quadratic magnitude scaling proposed by Stafford (2006). It is clear that when magnitude uncertainty is accounted for and the quadratic functional form is implemented a significant reduction in the computed inter-event standard deviation can be obtained.

2.4.3 Prediction intervals and parameter uncertainty

Once the regression approach has been selected and the dataset chosen one is able to conduct the regression analysis in order to obtain estimates of the model coefficients. While there are a number of algorithms available for this purpose, all decent algorithms will enable the analyst to simultaneously compute a covariance matrix associated with the estimated model coefficients. The trace of this matrix is generally used to compute the standard errors, and p -values of the coefficients in order to test for their statistical significance. The off-diagonal terms of the matrix are also inspected to

check that certain terms within the functional form are not strongly correlated with one another. However, recently it has been highlighted that this information can also be used in order to define a component of model-specific epistemic uncertainty.

In a routine regression analysis one would perform the regression and then look at the model coefficients and decide whether the observed trends are statistically significant or not. In the case that they are found to be not significant then the coefficient is commonly removed from the functional form. The basis for deciding whether or not a term is significant is to look at its p -value. The p -value can be interpreted as being the probability that a coefficient would take on its computed value purely by chance (as a result of the random noise in the data). That is, if a coefficient has a very low p -value then there is a very good chance that it is indicating a real dependence within the data. Ordinarily a derived coefficient is regarded as being significant if its p -value is less than 0.05 (corresponding to a 95% confidence limit). To summarise this process: a regression analysis is conducted and if a coefficient is found to have a p -value greater than 0.05 then it is often discarded, while those terms with values below 0.05 are retained. Coefficients that are near this limit, but that are retained, are clearly not as well constrained as other coefficients with much lower p -values. The implication is that the terms of the model that relate to these coefficients will be less well constrained than other parts. When one considers the model-specific epistemic uncertainty this varying degree of constraint should obviously be considered.

Often the reason why a particular coefficient is not well constrained is because the choice of the functional form is not ideal. In other cases, the lack of constraint can arise because there is little data corresponding to the effect that is trying to be modelled. In order to reflect both of these effects, Arroyo & Ordaz (2011) presented the concept of prediction intervals for ground-motion models. This concept is not at all new within regression analysis and can be found in most textbooks on the subject. However, it had (surprisingly) not previously been used in the context of ground-motion prediction. While Arroyo & Ordaz (2011) only considered the case of linear models, the standard framework can be extended to be described more generally by the following equation:

$$\hat{\sigma}_{\ln Sa}^2 = \sigma_{\ln Sa}^2 + \nabla \mu(\mathbf{X}|\theta) \text{COV}(\theta) \nabla \mu(\mathbf{X}|\theta)^T \quad [2.16]$$

In Equation (2.16), ∇ is the gradient operator, θ are the estimated model coefficients, COV represents the covariance, and the vector $\mathbf{X}$ is a set of input parameters that define a future earthquake scenario (the magnitude, distance, etc.). From Equation (2.16) it is clear that the variance for a *prediction* associated with a ground-motion model, $\hat{\sigma}_{\ln Sa}^2$, is larger than the variance obtained from the regression analysis, $\sigma_{\ln Sa}^2$. The extent to which

this is larger depends upon how well constrained the regression coefficients are as well as how strongly the functional for depends upon these coefficients. What is ordinarily found when implementing Equation (2.16) is that the influence of the uncertainty in the regression coefficients becomes larger as one considered scenarios that are further away from the centroid of the data used to develop the model. In effect, the formulation of Equation (2.16) *knows* that the model is being pushed toward its limits of applicability and increases its model-specific uncertainty accordingly.

2.5 Future trends

The approaches used to develop ground-motion models have been relatively stable for many years now. However, there have been an increasing number of contributions in the literature that demonstrate issues that are associated with these existing approaches and it is likely that ground-motion model development will take more of these issues into account in the coming years. In the present section I outline what I anticipate to be the major changes that are either already being considered or that are likely to attract further attention in the near future.

2.5.1 Removal of the ergodic assumption

In order to constrain empirical models for the wide range of magnitude and distance scenarios that are considered within probabilistic hazard and risk analyses it is generally necessary to compile datasets of ground-motions from different regions (see Fig. 2.1). However, as mentioned earlier, what we ideally desire is a prediction of the distribution of motions that should be expected when a particular event occurs in a particular location. Any differences that exist between the propagating media, the source excitations or the site responses in the regions from where the data has been obtained will manifest as increased variance in the ground-motion model. One of the key challenges that remains to be properly solved is how to remove the ergodic assumption that is currently made. Studies have already been conducted on this issue and have usually been undertaken in data-rich regions. However, the vast majority of these studies have fundamental issues that leave the solution of this problem unresolved. Unfortunately, it is not possible to enter into a full discussion of these issues here. However, from the work that has been completed to date it is clear that a significant reduction in the site-specific standard deviation can be obtained once the ergodic assumption is removed. That said, the removal of this assumption requires a certain level of understanding about the specific characteristics of the target site and region that is often not currently available. The net result is that while the aleatory variability can be reduced quite significantly, this

reduction is offset by an increase in the epistemic uncertainty associated with accurately defining the characteristics of the target region.

2.5.2 Semi-empirical approaches

The NGA project was relatively unique in that it combined traditional empirically based regression techniques with constraints from numerical simulations in order to capture aspects of ground-motion scaling that could not be extracted from the data alone. The features that have been constrained thus far include the nonlinear site response and the large magnitude scaling (which are in some ways coupled). While this may be considered an advancement in many ways the current approaches to combining these theoretical constraints with empirical approaches is not correct. The problem is that when one imposes a theoretical constraint upon the way in which a particular term of a model should scale one does not currently also allocate a component of uncertainty to this part of the model. The consequence is that a component of uncertainty that is associated with this component is forced to be allocated to another part of the model. In the case that theoretical constraints are imposed in regions where there is zero data, this will not influence the derived models coefficients. However, when there is some data then neglecting the relevant component of uncertainty influences the overall model fit. To clarify the situation here, let us consider a particular example. Imagine that numerical analyses are used to obtain a theoretical model for nonlinear site response, and that this model component can be represented by $g_{\text{site}}(V_{s30}, \dots)$. One way to incorporate this scaling into a regression analysis would be to work with the following mathematical framework:

$$y_{ij} = \ln Sa_{ij} - g_{\text{site}}(V_{s30,ij}, \dots) = \mu'(\mathbf{Z}_{ij}|\theta) + \delta_{E,i} + \delta'_{A,ij} \quad [2.17]$$

Note that in Equation (2.17) the ultimate model for median predictions is given by $\mu(\mathbf{Z}_{ij}|\theta) \equiv \mu'(\mathbf{Z}_{ij}|\theta) + g_{\text{site}}(V_{s30,ij}, \dots)$. Furthermore, the intra-event residual here also has a slightly different interpretation because in an empirical dataset the intra-event residual is actually composed of a path-specific component, a site-specific component, and a residual error term. If we were to account for the theoretical constraint appropriately, we would also account for the site-specific component of the intra-event residual and this would also be subtracted from the dependent variable. However, as we cannot know this term *a priori* we are forced to allocate this component of the residual into what should just be the path and residual error components.

This problem has not yet been recognised and addressed by ground-motion developers, but will become increasingly relevant as further

attempts are made to remove the ergodic assumption within ground-motion modelling.

2.5.3 Extended ranges of applicability

There are many situations in which one must perform a hazard or risk analysis in a region for which only weak ground-motion data is available. In such cases, a question that always arises is whether it is possible to develop a model using just this weak-motion data and to then extrapolate this model so that it makes valid predictions of motions for the moderate-to-large events that are always found to dominate the hazard computations. Generally it is not possible to infer the appropriate magnitude scaling for the larger events from the data from small earthquakes. Consequently people look to import models from other regions that have been developed using data from the moderate-to-large magnitude events that are assumed to be important. However, when one extrapolates these existing models down to magnitudes for which data is available in the target region one generally finds that the existing models over-predict the weak motions. This issue has only recently been highlighted (Bommer *et al.*, 2007), but has resulted in modifications having been made to recently developed ground-motions models that were otherwise thought to capture the pertinent features of magnitude scaling (Chiou *et al.*, 2010; Atkinson & Boore, 2011).

Models that are currently being developed as well as some that have recently been developed are now paying far greater attention to the scaling of magnitude over a much broader range. In particular, whereas emphasis has traditionally been given to how motions should scale for moderate-to-large events that have the most engineering significance, it is now recognised that it is also very important to appropriately capture the scaling for much smaller events. When consideration is given to this scaling, the resulting models become far more transportable from region to region.

However, from an uncertainty point-of-view, it is likely that a consequence of considering a larger range of magnitudes is that estimates of the aleatory variability will increase due to the metadata for these small events not being known as well as that for the larger events (Bommer *et al.*, 2007). This *apparent* heteroskedasticity with respect to magnitude can be incorporated into derived models, but it strictly should not be because it is incorporating epistemic uncertainty due to parametric uncertainty into what is treated as aleatory variability within hazard and risk computations. The consequence of this is that short return period demand and loss predictions will be overestimated by these new models unless the heteroskedasticity of the epistemic uncertainty in the metadata is accounted for.

2.6 Conclusions

Empirical ground-motion modelling is relatively mature yet there are many enhancements that can be made to the way in which models are derived. The scarcity of recordings from many earthquake scenarios of interest to hazard and risk applications has resulted in modelling assumptions being made that are strictly not appropriate. Many of the issues that have been outlined in this chapter relate to the need to try to compile a dataset that provides constraint for the wide range of scenarios considered in these applications. In a similar manner, the brief discussion of the future trends actually relate to how one can go about removing some of these assumptions as the numbers of high-quality recordings available for use in regression analyses increases in time.

The most important point to keep in mind when considering ground-motion models in the context of probabilistic hazard and risk analyses is that the role of these models is to provide the distribution of ground-motion amplitudes for given earthquake scenarios. This distribution is defined by two parameters and each of these parameters has a degree of epistemic uncertainty associated with it. In order to rigorously account for ground-motion uncertainty within hazard and risk analyses we must account for these epistemic uncertainties. Traditional approaches based upon logic tree formulations do not capture the extent of epistemic uncertainty adequately as they ignore the important contribution of model-specific epistemic uncertainty.

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Spatial correlation of ground motions in estimating seismic hazards to civil infrastructure

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Abstract: During the last decade the attention given to studies of ground-motion correlation has grown. It is related to similarity of ground-motion variability, or residuals between observations and results of modeling, for different earthquakes (between-earthquake correlation) and different locations (within-earthquake correlation). The proper treatment of ground-motion correlation is essential for the estimation of seismic hazard, damage and loss for widely located building assets (portfolios) and spatially distributed structures (lifelines). The goal of this chapter is to summarize the recent advances of ground-motion correlation studies, and to provide a list of key references.

Key words: strong ground motion, correlation, seismic hazard and loss.

3.1 Introduction

In the past decade increasing attention has been focused on studies of ground-motion correlation. There has been a focus on similarity of ground-motion variability, or differences between observations and results of modeling, for different earthquakes (between-earthquake correlation) and different locations (within-earthquake correlation). The correlation of ground-motion residuals reflects a non-random component in the residuals, which is caused by factors not accounted for by the ground-motion model and which therefore constitutes epistemic uncertainty (attributable to incomplete knowledge) that may be reduced, in principle. Parameters of the probability distribution function for the loss to a portfolio (e.g., fractiles and standard deviation) during an earthquake, which are very important for decision making and mitigation activities, are affected by ground-motion uncertainty and correlation. Therefore the consideration of ground-motion correlation is essential for the estimation of seismic hazards, damage and loss for widely-located building assets (portfolios) and spatially-distributed structures (lifelines) (see reviews in Sokolov and Wenzel, 2011a,b). This chapter summarizes the recent advances in ground-motion correlation

studies by emphasizing the influence of correlation on seismic hazard and loss estimation, and provides a list of key references where interested readers may find comprehensive information.

3.2 Spatial correlation of ground motions: evaluation and analysis

3.2.1 Basic definitions

The ground motion parameter Y at n locations during m earthquakes can be represented by

$$\log Y_{i,j} = f(e_i, p_{i,j}, s_{i,j}) + \eta_i + \varepsilon_{i,j} \quad i = 1, 2, \dots, m; \quad j = 1, 2, \dots, n; \quad [3.1]$$

where e_i denotes variables that are properties of the earthquake source; $p_{i,j}$ are the properties of propagation path; $s_{i,j}$ are the properties of site location j during earthquake i ; and f is a suitable function, which describes the dependence of mean value of ground-motion parameter $\log \bar{Y}_{i,j}$ on magnitude, distance, local site conditions, etc., i.e. $\log \bar{Y}_{i,j} = f(e_i, p_{i,j}, s_{i,j})$. The random variables η_i and $\varepsilon_{i,j}$ represent the between-earthquake (or inter-event) and within-earthquake (or intra-event) components of variability (independent and normally distributed with variances σ_η^2 and σ_ε^2), respectively. The value of η_i is common to all sites during a particular earthquake i , and the value of $\varepsilon_{i,j}$ depends on the site. Assuming the independence of the two random terms, the total variance σ_T^2 is given by $\sigma_T^2 = \sigma_\eta^2 + \sigma_\varepsilon^2$ (e.g. Brillinger and Preisler, 1984, 1985; Abrahamson and Youngs, 1992; Joyner and Boore, 1993; Boore *et al.*, 1997). Note that if the pseudospectral acceleration (PSA) is considered, the variances σ_η^2 and σ_ε^2 are determined as the functions of the natural vibration period T . A comprehensive description may be found elsewhere (e.g. Goda and Hong, 2008a; Goda and Atkinson, 2009; Jayaram and Baker, 2009); here for the sake of simplicity, we describe the period-independent correlation.

Let us consider two recording sites separated by distance Δ . The similarity of ground-motion variability for a randomly selected earthquake i at the sites j and k (or, $\eta_i + \varepsilon_{i,j}$ and $\eta_i + \varepsilon_{i,k}$ values) can be represented by the correlation coefficient $\rho_T(\Delta)$, given by (e.g. Park *et al.*, 2007; Goda and Hong, 2008a):

$$\rho_T(\Delta) = \frac{\sigma_\eta^2 + \rho_\varepsilon(\Delta)\sigma_\varepsilon^2}{\sigma_T^2} = \rho_\eta + \rho_\varepsilon(\Delta) \frac{\sigma_\varepsilon^2}{\sigma_T^2} = \rho_\eta + \rho_\varepsilon(\Delta)(1 - \rho_\eta) \quad [3.2]$$

where $\rho_\eta = \sigma_\eta^2/\sigma_T^2$ is the between-earthquake correlation (Wesson and Perkins, 2001); $\rho_\varepsilon(\Delta)$ represents the correlation coefficient between $\varepsilon_{i,j}$ and $\varepsilon_{i,k}$ (within-earthquake correlation). Alternatively, the correlation coefficient $\rho_T(\Delta)$ is expressed as

$$\rho_T(\Delta) = 1 - \frac{\sigma_d^2}{2\sigma_T^2} \quad [3.3]$$

where σ_d^2 denotes the variance of $(\eta_i + \varepsilon_{ij}) - (\eta_i + \varepsilon_{ik})$ (Boore *et al.*, 2003). The within-earthquake correlation coefficient $\rho_\varepsilon(\Delta)$, which by definition is given by

$$\rho_\varepsilon(\Delta) = \frac{COV[\varepsilon_{ij}, \varepsilon_{ik}]}{\sigma_\varepsilon^2} \quad [3.4]$$

where $COV(A, B)$ denotes the covariance of A and B , can be expressed as

$$\rho_\varepsilon(\Delta) = 1 - \frac{\sigma_d^2}{2\sigma_\varepsilon^2} \quad [3.5]$$

Note that the derivation of the correlation coefficients shown in equations (3.2) to (3.5) are based on the consideration that residuals are related to the statistics of the logarithm of a single randomly oriented horizontal component Y_{ij} . If the correlation coefficients are derived by considering the statistics of the geometric mean of two orthogonal horizontal components of the ground motion parameter, one has the following relationship:

$$\rho_\varepsilon(\Delta) = \rho_{gm}(\Delta) \frac{1 + \rho_C}{2} \quad [3.6]$$

where $\rho_{gm}(\Delta)$ is the correlation coefficient based on geometric mean of two orthogonal horizontal components; ρ_C is the correlation coefficient between ground-motion parameters of two orthogonal horizontal components. Baker and Cornell (2006) provided the following empirical expression for ρ_C , which depends on T :

$$\rho_C = 0.79 - 0.023 \ln(T) \quad [3.7]$$

3.2.2 Procedures of analysis

Let $Z(x)$ denote the total residuals between the log of observed ground motion parameter at a site x and that of the value predicted from the attenuation relationship, i.e. $Z(x) = \log Y_{ij} - \log \hat{Y}_{ij}$. Several methods, which can be adopted to estimate within-earthquake correlation of ground motion residuals, have been described in the literature. Here we briefly describe three of them.

In *Procedure 1*, Boore *et al.* (2003) performed the following analysis to investigate the within-earthquake correlation of ground motion residuals: (1) select the data for a selected earthquake i ; (2) compute separation distance Δ for all pairs of stations; (3) compute the differences $Z(x)$; (4) divide the range of Δ into bins so that the separation distance in the same bin is

within $\Delta \pm \delta/2$; (5) all pairs of sites that fall in the bin centered at Δ (i.e. the sites that are separated by a distance $\in (\Delta - \delta/2, \Delta + \delta/2)$) are used to estimate the variance of the residuals, and then compute the standard deviation σ_d of residuals within each Δ bin. Once $\sigma_d(\Delta)$ is estimated it can be used in equations (3.3) and (3.5) to calculate estimations of $\hat{\rho}_T(\Delta)$ and $\hat{\rho}_e(\Delta)$, respectively.

In *Procedure 2*, Wang and Takada (2005) considered the covariance function of the values $Z(x_1)$ and $Z(x_2)$, which is described as

$$COV[Z(x_1), Z(x_2)] = COV(\Delta) = E\{[Z(x_1) - \mu_Z][Z(x_2) - \mu_Z]\} \quad [3.8]$$

where $E[\bullet]$ is an expectation; Δ is a separation distance between the locations x_1 and x_2 ; and μ_Z is the mean value of $Z(x)$. Correlation function is obtained by normalizing covariance function with the variance σ_Z^2

$$\rho[Z(x_1), Z(x_2)] = \rho(\Delta) = \frac{COV(\Delta)}{\sigma_Z^2} \quad [3.9]$$

A discrete version of the equations follows

$$\mu_Z = \frac{1}{N_{\text{all}}} \sum_{j=1}^{N_{\text{all}}} Z(x_j) \quad [3.10a]$$

$$COV(\Delta) = \frac{1}{N(\Delta)} \sum_{j=1}^{N(\Delta)} [Z(x_j) - \mu_Z][Z(x_j + \Delta) - \mu_Z] \quad [3.10b]$$

where N_{all} is the total number of observation sites; $N(\Delta)$ is the number of pairs of sites separated by Δ . Underling assumption of this method is that $Z(x)$ is a homogeneous 2D stochastic field; therefore, the homogeneity of the residual data should be examined for every considered case (Wang and Takada, 2005). Homogeneity implies similarity of statistical properties (mean value and standard deviation) of the residuals for various distances and magnitudes.

In *Procedure 3*, Baker and Jayaram (2008), Jayaram and Baker (2009), and Esposito and Iervolino (2011) evaluated the within-earthquake correlation in the form of semi-variogram $\gamma(\Delta)$ (a measure of average dissimilarity between the data, e.g. Goovaerts, 1997) as follows:

$$\gamma(\Delta) = \frac{1}{2} [E\{Z_u - Z_{u+\Delta}\}^2] \quad [3.11]$$

where Z_u and $Z_{u+\Delta}$ are the spatially distributed random functions at site locations u and $u + \Delta$. In this procedure, second-order stationary is typically assumed meaning that (1) the expected value of the random variable Z_u is a constant across the space, so the data available over the entire region of interest can be pooled, and (2) the two-point statistics do not depend on site locations u and $u + \Delta$, but only on their separation distance Δ .

The experimental stationary semi-variogram $\hat{\gamma}(\Delta)$ is estimated from a dataset as follows

$$\hat{\gamma}(\Delta) = \frac{1}{2N(\Delta)} \sum_{\alpha=1}^{N(\Delta)} \{z_{u_\alpha} - z_{u_\alpha+\Delta}\}^2 \quad [3.12]$$

where z_{u_α} denotes the data value at location u ; $N(\Delta)$ denotes the number of pairs of sites separated by Δ ; and $\{z_{u_\alpha}, z_{u_\alpha+\Delta}\}$ denotes the α th pair. For a given earthquake i at site j , the ground-motion residual is estimated as

$$Z(j) = \log Y_j - \log \bar{Y}_j = \varepsilon_j + \eta \quad [3.13]$$

The normalized residual $\tilde{\tau}_j$ and the normalized within-earthquake residual $\tilde{\varepsilon}_j$ are computed as

$$\tilde{\tau}_j = \frac{\varepsilon_j + \eta}{\sigma_{\varepsilon,j}}; \quad \tilde{\varepsilon}_j = \frac{\varepsilon_j}{\sigma_{\varepsilon,j}} \quad [3.14]$$

where $\sigma_{\varepsilon,j}$ denotes the standard deviation of the within-earthquake residual at site j . Note that the between-earthquake residual η is a constant across all the sites during a given earthquake. Thus, the experimental semi-variogram function of $\tilde{\varepsilon}$ can be obtained as follows (see Baker and Jayaram, 2008):

$$\begin{aligned} \hat{\gamma}(\Delta) &= \frac{1}{2N(\Delta)} \sum_{\alpha=1}^{N(\Delta)} \{\tilde{\varepsilon}_{u_\alpha} - \tilde{\varepsilon}_{u_\alpha+\Delta}\}^2 \\ &= \frac{1}{2N(\Delta)} \sum_{\alpha=1}^{N(\Delta)} \left[\frac{\ln(Y_{u_\alpha}) - \ln(\bar{Y}_{u_\alpha}) - \eta}{\sigma_{u_\alpha}} - \frac{\ln(Y_{u_\alpha+\Delta}) - \ln(\bar{Y}_{u_\alpha+\Delta}) - \eta}{\sigma_{u_\alpha+\Delta}} \right]^2 \\ &= \frac{1}{2N(\Delta)} \sum_{\alpha=1}^{N(\Delta)} \left[\frac{\ln(Y_{u_\alpha}) - \ln(\bar{Y}_{u_\alpha})}{\sigma_{u_\alpha}} - \frac{\ln(Y_{u_\alpha+\Delta}) - \ln(\bar{Y}_{u_\alpha+\Delta})}{\sigma_{u_\alpha+\Delta}} \right]^2 \\ &= \frac{1}{2N(\Delta)} \sum_{\alpha=1}^{N(\Delta)} \{\tilde{\tau}_{u_\alpha} - \tilde{\tau}_{u_\alpha+\Delta}\}^2 \end{aligned} \quad [3.15]$$

where σ_{u_α} is the standard deviation of the within-earthquake residual at location u_α . The equation involves an approximation due to the mild assumptions that

$$\frac{\eta}{\sigma_{u_\alpha}} = \frac{\eta}{\sigma_{u_\alpha+\Delta}}.$$

For the case of normalized between-earthquake residuals, the relation between semi-variogram $\hat{\gamma}(\Delta)$ and within-earthquake correlation $\hat{\rho}_\varepsilon(\Delta)$ is the following:

$$\hat{\gamma}(\Delta) = 1 - \hat{\rho}_\varepsilon(\Delta). \quad [3.16]$$

As can be seen, two approaches for estimating within-earthquake correlation are mentioned above: one is based on the estimation of sample semi-variogram while another is the direct evaluation of the Pearson's linear correlation coefficient. The former method is based on sample semi-variogram of normalized residuals

$$\hat{\gamma}(\Delta) = \frac{1}{2} \left[\frac{\sigma_d(\Delta)}{\sigma_\varepsilon} \right]^2.$$

Evaluation of between-earthquake standard deviation σ_ε is important here, because it can significantly affect the estimations of within-earthquake correlation (e.g. Goda and Hong, 2008a; Goda and Atkinson, 2009, 2010). On one hand, σ_ε for a given seismic event may be assessed by evaluating the sample semi-variogram for the data pairs with a sufficiently large separation distances Δ , e.g. between 100 and 200 km (Goda and Atkinson, 2009, 2010). It is assumed that between-earthquake residuals are uncorrelated at sufficiently long separation distances, and the semi-variogram is expected to attain a constant plateau level. Alternatively, σ_ε may be calculated directly as the event-based standard deviation of regression residuals directly. However, the value may be underestimated if the residuals are strongly correlated (Kawakami and Mogi, 2003; Hong *et al.*, 2009).

The second method, which uses equations 3.10a and 3.10b, implies evaluation of the Pearson's linear correlation coefficient for a given separation distance bin. Goda and Hong (2008a) and Goda and Atkinson (2009) compared the two methods by assessing differences in the estimated $\rho_\varepsilon(\Delta)$ in the numerical calculations. Goda and Atkinson (2009) showed that the method, which is based on the direct evaluation of the correlation coefficient, tends to result in a slightly more rapid decay of $\rho_\varepsilon(\Delta)$. At the same time, they noted that the possible differences of $\rho_\varepsilon(\Delta)$ based on two methods can be largely attributed to the effects of between-earthquake standard deviation and suggested creating an overall $\rho_\varepsilon(\Delta)$ model by averaging the results of the methods.

In practical calculation, when it is necessary to consider several earthquakes, obviously it is not possible to apply in the calculations the event-dependent between-earthquake standard deviation σ_ε , which was used for the estimation of within-earthquake correlation based on empirical data. Thus, to be consistent with the technique used for calculation of spatially correlation ground motion field, which is described in the next section, it may be necessary to use generalized between-earthquake standard deviation (e.g. associated with the used ground-motion prediction model) to estimate site-to-site correlation and to analyze possible differences between these estimations (e.g. Sokolov *et al.*, 2012).

3.2.3 Models of spatial correlation

The functions $\hat{\rho}_\epsilon(\Delta)$ and $\hat{\gamma}(\Delta)$ provide a set of experimental values for a finite number of separation distances Δ . A continuous function can be fitted based on these experimental values. The function $\rho_\epsilon(\Delta)$ may be represented by an exponential decay function (e.g. Boore *et al.*, 2003; Wang and Takada, 2005; Goda and Hong, 2008a):

$$\rho_\epsilon(\Delta) = \exp(a\Delta^b) \quad [3.17]$$

where a and b are the model parameters. Again, if the PSA is considered, the correlation function and parameters are the functions of T (Goda and Hong, 2008a). Goda and Atkinson (2009) introduced an extended version of equation (3.17), which is based on an assumption of uncorrelated ground-motion parameters at well-separated sites

$$\rho_\epsilon(\Delta, T) = \max\{\phi(T) \exp[-\alpha(T)\Delta^{\beta(T)}] - \phi(T) + 1, 0\} \quad [3.18]$$

where $\alpha(T)$, $\beta(T)$, and $\phi(T)$ are the model parameters.

Wang and Takada (2005) suggested to characterize the function $\rho_\epsilon(\Delta)$ by a single parameter – the so-called ‘correlation distance’ R_C . The correlation distance shows the site-to-site distance for which the correlation coefficient $\rho_\epsilon(\Delta)$ decreases to $1/e = 0.368$.

Alternatively, an exponential model of the semi-variogram can be used for modeling $\gamma(\Delta)$ (and thus $\rho_\epsilon(\Delta)$)

$$\gamma(\Delta) = a[1 - \exp(-3\Delta/b)] \quad [3.19]$$

where a and b are the sill and the range of the semi-variogram function, respectively. The sill of a semi-variogram equals to the variance of spatially distributed random function; while the range is defined as the separation distance Δ at which $\gamma(\Delta)$ equals 0.95 times the sill of the exponential semi-variogram. The relation between semi-variogram and within-earthquake correlation $\rho_\epsilon(\Delta)$ is the following:

$$\gamma(\Delta) = a[1 - \rho_\epsilon(\Delta)] \quad [3.20]$$

Some models (e.g. Goda and Hong, 2008a; Goda and Atkinson, 2009) interpolate empirical results obtained for moderate-to-long distances to meet the theoretical requirement of perfect correlation at zero-separation distance. The within-earthquake correlation model at relatively short separate distances (less than 5 km), where available data are scarce, was analyzed by Goda and Atkinson (2010). The data from SK-net recording stations, which are densely distributed in the Kanto (Japan) region, were used.

3.2.4 Numerical simulation of spatially correlated ground-motion parameters

In ground-motion models, the parameter of motion Y generated by earthquake i with magnitude M at a site j at distance R is estimated as a lognormally distributed random variable as $\ln Y_{ij} = N(\log \bar{Y}_{ij}, \sigma^2)$, where $\log \bar{Y}_{ij} = f(e_i, p_{i,j}, s_{i,j})$. In addition to the mean value of ground motion $\bar{Y}_{ij}$, we need to generate the standard normal variates (errors) of η_i and $\varepsilon_{i,j}$ (see equation 3.1). The dependence structure among the variates is described by the correlation matrix Σ .

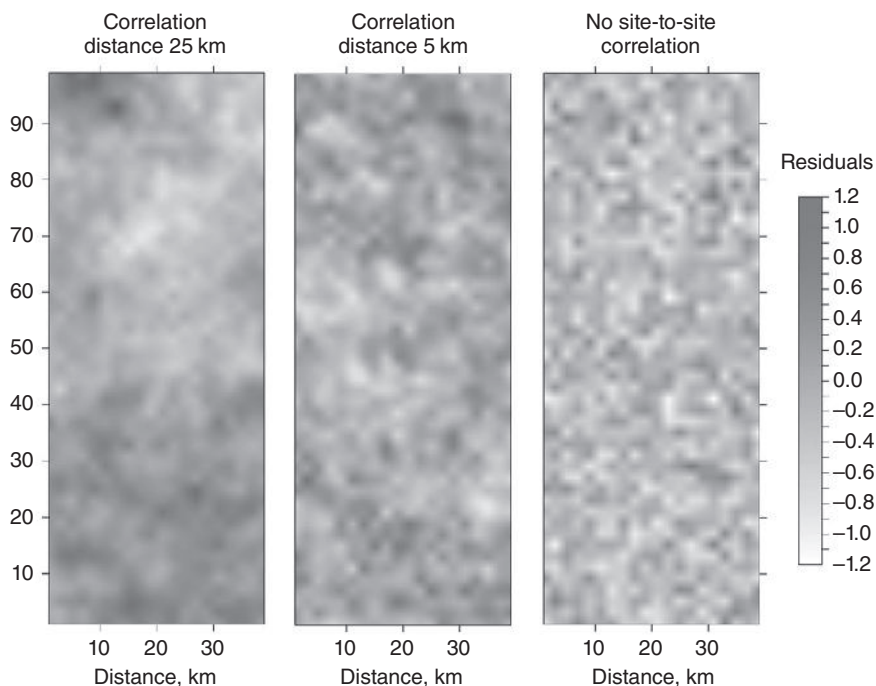
For the generation of the k -site random field of ground-motion values that are spatially correlated, it is necessary to generate a Gaussian vector of correlated standard normal variables (total residual term) $\mathbf{X} = [X_1, X_2, \dots, X_k]$ with a symmetric correlation matrix Σ , or $\mathbf{X} \sim N_k(0, \Sigma)$. The correlation matrix Σ is defined as follows:

$$\Sigma = \begin{pmatrix} 1 & \rho_{12} & \dots & \rho_{1k} \\ \rho_{21} & 1 & \vdots & \rho_{2k} \\ \vdots & \vdots & \ddots & \vdots \\ \rho_{k1} & \rho_{k2} & \dots & 1 \end{pmatrix}, \quad [3.21]$$

where ρ_{kl} is the empirical correlation coefficient calculated for the sites k and l separated by a distance Δ . Descriptions of the procedure for generation of the k -site random field of ground-motion error values that are spatially correlated may be found in Johnson (1987) and Park *et al.* (2007). The generation can be done in the following steps. First, a vector of independent standard normal variates $\mathbf{U} = [U_1, U_2, \dots, U_k]$ with standard deviation σ_T , or $\mathbf{U} \sim N_k(0, \sigma_T^2)$, is generated. Then, a correlation matrix Σ is constructed and a Cholesky decomposition is applied to represent the correlation matrix Σ as the matrix product of matrix $\mathbf{B}$ and its transposition $\mathbf{B}^T$, i.e., $\Sigma = \mathbf{B} \mathbf{B}^T$. The required vector $\mathbf{X}$ is obtained as $\mathbf{X} = \mathbf{B}\mathbf{U}$. These X_j values are added to the median ground-motion term $\ln \bar{Y}_{ij}$ to obtain a realisation of spatially correlated ground motions. Figure 3.1 shows examples of distribution of strong-motion residuals (peak ground acceleration (PGA), $\ln g, \sigma_T = 0.5$) generated for various correlation distances (25 and 5 km) and for the case of spatially uncorrelated ground motion (all the variability is within-earthquakes).

2.3.5 Estimation of correlation models from empirical data

As can be seen from the next sections, the treatment of ground-motion correlation is essential for the estimation of seismic hazard, damage and loss for widely located building assets (portfolios) and spatially distributed



3.1 Examples of distribution of strong-motion residuals (PGA, $\ln g$, $\sigma_T = 0.5$) generated for various correlation distances (25 and 5 km) and for the case of spatially uncorrelated ground motion (all the variability is within-earthquakes).

structures (lifelines). Therefore an important part of the estimations is selection of proper correlation model.

Several modern ground-motion attenuation equations (e.g. New Generation Attenuation (NGA) models; Power *et al.*, 2008) allow the recognition of the between-earthquake correlation, because the equations include specification of the between-earthquake and within-earthquake components of variability (Boore *et al.*, 1997; Douglas, 2003, 2006; Tsai *et al.*, 2006; Abrahamson *et al.*, 2008). Usually, within-earthquake standard deviation is found to be larger than that for between-earthquake standard deviation, although the relation depends on the vibration period (e.g. Bommer *et al.*, 2003; Bommer and Crowley, 2006). Atkinson (2006) and Morikawa *et al.* (2008) showed that the total standard deviation of ground-motion prediction may be reduced when using a single site-specific model or a region-specific correction factor, which is determined by grouping ground-motion data at specific stations of a dense strong-motion array. Tsai *et al.* (2006) showed that a reduction of the total standard deviation could be achieved if the path effect could be specified. This reduction, obviously, is related to the

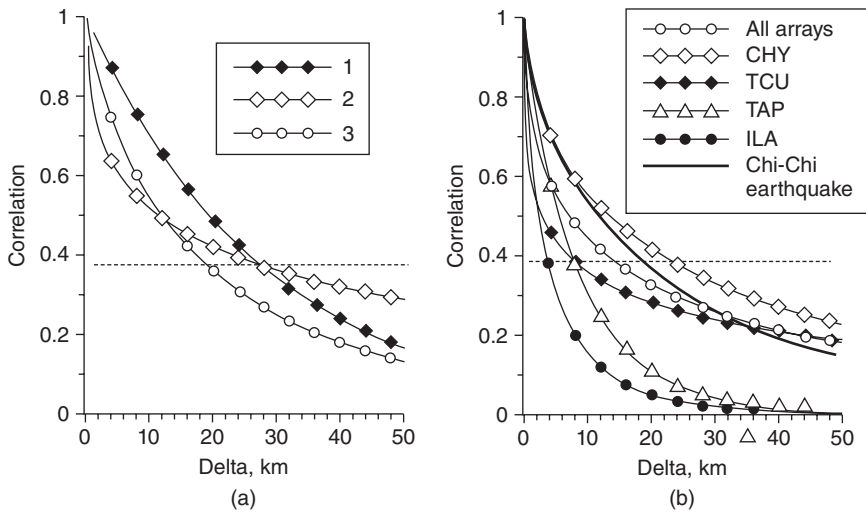
within-earthquake component, and Morikawa *et al.* (2008) obtained almost similar values for between-earthquake and within-earthquake standard deviations after the correction. Thus, the between-earthquake correlation ρ_η may vary, at a minimum, between 0.06 (Bommer *et al.*, 2003) and 0.5 (Morikawa *et al.*, 2008). The correlation coefficient is a relative quantity, so the variation of ρ_η is important, and equally the variation of the total standard deviation is important also.

The within-earthquake correlation should be evaluated for a given area empirically. A dense observation of records from numerous earthquakes is necessary for modelling the within-earthquake correlation structure; therefore the correlation structure has not been extensively investigated for conventional ground-motion attenuation equations mentioned above. Boore *et al.* (2003), Hok and Wald (2003) and Lin *et al.* (2006) considered one or few particular earthquakes in California; Wang and Takada (2005) analyzed separately a few earthquakes in Japan and the Chi-Chi earthquake in Taiwan. Kawakami and Mogi (2003) used records from several earthquakes in Japan (the Chiba array 30 km east of Tokyo; SIGNAL array in Tokyo), and Taiwan (SMART-1 array). Evans and Baker (2006) used the NGA database (Power *et al.*, 2008), which is primarily based on the Californian data and a few earthquakes in Taiwan (the Chi-Chi earthquake and large aftershocks). Goda and Hong (2008a) and Jayaram and Baker (2009) considered the Chi-Chi earthquake and some Californian earthquakes, and Hong *et al.* (2009) analyzed Californian data only. More recently, Goda and Atkinson (2009, 2010) used a large database collected in Japan (records from the K-NET and KiK-net strong-motion networks) to study spatial correlation for peak ground acceleration and pseudo-spectral acceleration at different periods from 0.1 to 5.0 s. Sokolov *et al.* (2010, 2012) analyzed characteristics of ground motion correlation using the database accumulated in Taiwan. The European Strong-Motion database (ESD) and the Italian Accelerometric Archive (ITACA) were used by Esposito and Iervolino (2011) for evaluation of the spatial correlation of PGA and peak ground velocity (PGV) residuals. Goda (2011) estimated event-specific (inter-event) variability of spatial correlation (peak ground motions and response spectra) using the consolidated ground-motion database consisting of 41 well-recorded earthquakes (occurred mostly in Japan).

The results reported by these research works reveal a different rate of decay of the within-earthquake correlation with separation distance. Table 3.1 summarizes the data used and the obtained results of some of the studies. Among the possible reasons of the difference, it is possible to mention different ground-motion prediction models, as well as different ground motion parameters, that were used for the analysis. Figure 3.2a compares within-earthquake correlation functions estimated for the 1999 Chi-Chi (Taiwan) earthquake in three research works. The ground-motion

Table 3.1 Parameters of site-to-site within-earthquake correlation functions $\rho_\Delta(\Delta)$ reported in literature

Reference	Function	Correlation distances, km	Data used
Wang and Takada (2005)	$\exp(\Delta/\alpha)$	27.3	The 1999 Chi-Chi (Taiwan) earthquake; PGV; ground-motion model developed for Japan by Midorikawa and Ohtake (2002)
Goda and Hong (2008a)	$\exp(a\Delta^b)$	28.7	The 1999 Chi-Chi (Taiwan) earthquake; PGA; NGA ground-motion model developed by Boore and Atkinson (2008)
Sokolov <i>et al.</i> (2010)	$\exp(a\Delta^b)$	18.2	The 1999 Chi-Chi (Taiwan) earthquake; PGA; ground-motion model developed in the study
Sokolov <i>et al.</i> (2010)	$\exp(a\Delta^b)$	34.1	The 2003 (M_w 6.8, Taiwan) earthquake; PGA; ground-motion model developed in the study
Sokolov <i>et al.</i> (2010)	$\exp(a\Delta^b)$	4.0–23.5	Records from 13 ($M_w > 6.0$) earthquakes in Taiwan; PGA; ground-motion model developed in the study; region-dependent estimations
Goda and Hong (2008a)	$\exp(a\Delta^b)$	<2.0 (≈ 1.7)	Records from 6 large earthquakes in California; PGA, ground-motion model developed in the study
Hong <i>et al.</i> (2009)	$\exp(a\Delta^b)$	<2.0 (≈ 1.4)	Records from 39 earthquakes in California; PGA, ground-motion model developed in the study
Goda and Atkinson (2009)	$\gamma \exp(a\Delta^b) - \gamma + 1.0$	19.0–29.0	Records from 155 earthquakes occurred in Japan; PGA; ground-motion model developed in the study; earthquake depth-dependent estimations



3.2 Within-earthquake correlation functions $\rho_s(\Delta)$ estimated for particular earthquakes. (a) The 1999 Chi-Chi earthquake, 1 – Wang and Takada (2005); 2 – Goda and Hong (2008a); 3 – Sokolov *et al.* (2010). (b) The data from moderate-to-large earthquakes ($M_w > 6.0$) in Taiwan (Sokolov *et al.*, 2010), estimations for particular strong-motion arrays in Taiwan. CHY array – thick sediments; TCU array – stiff soils in extended hilly areas; TAP array – triangular asymmetric alluvium-filled Taipei basin; ILA array – Quaternary alluvial Ilan basin.

models developed for Japan and the maximum value of horizontal peak velocity amplitudes (PGV) was considered by Wang and Takada (2005). The NGA model based on worldwide data (Boore and Atkinson, 2008) and average horizontal acceleration was used by Goda and Hong (2008a). Sokolov *et al.* (2010, 2012) considered the geometric averages of two horizontal peak amplitudes and the regional strong-motion prediction equation for peak ground acceleration.

It has been shown that the difference relates to the frequency content of ground motion (Goda and Hong, 2008a; Baker and Jayaram, 2008). On the other hand, the difference may be caused by regional peculiarities (Goda and Atkinson, 2009). Analysis of the within-earthquake correlation in Taiwan (Sokolov *et al.*, 2010, 2012) indicates that the correlation structure is highly dependent on local geology and on peculiarities of propagation path (azimuth-dependent attenuation). Application of region- or site-dependent correction reduces the within-earthquake correlation, especially at large distances.

Figure 3.2b shows several models of within-earthquake correlation estimated by Sokolov *et al.* (2010) using a dataset from the 13 moderate-to-large ($M_w > 6.0$) earthquakes in Taiwan separately for several arrays of the

TSMIP network: the CHY, ILA, TAP, and TCU arrays. The CHY array is located on the Chianan Plain – an area of thick sediments, from central west to southwestern Taiwan; most stations of the ILA and the TAP arrays are installed within alluvium-filled basins with relatively rapid lateral changes of depth of sediments; the stations of TCU array are located partly on the northern part of Chianan Plain and hilly areas in Northwestern Taiwan. The comparison clearly reveals the influence of peculiarities of local geological conditions on within-earthquake correlation. It seems that the correlation depends on spatial variability of geological characteristics of the area; the correlation should decrease for the sites that do not share the same geology, or for the sites that are not affected by similar site effects.

Note also that regression analysis, which is used for development of prediction equations, does not usually incorporate the spatial correlation structure. The influence of spatial correlation in the analysis was studied by Hong *et al.* (2009) and Jayaram and Baker (2010). It has been shown that the effects of incorporating spatial correlation on the estimated regression coefficients are insignificant; however, the variances of the between-earthquake and of within-earthquake residuals may change significantly (Jayaram and Baker, 2010).

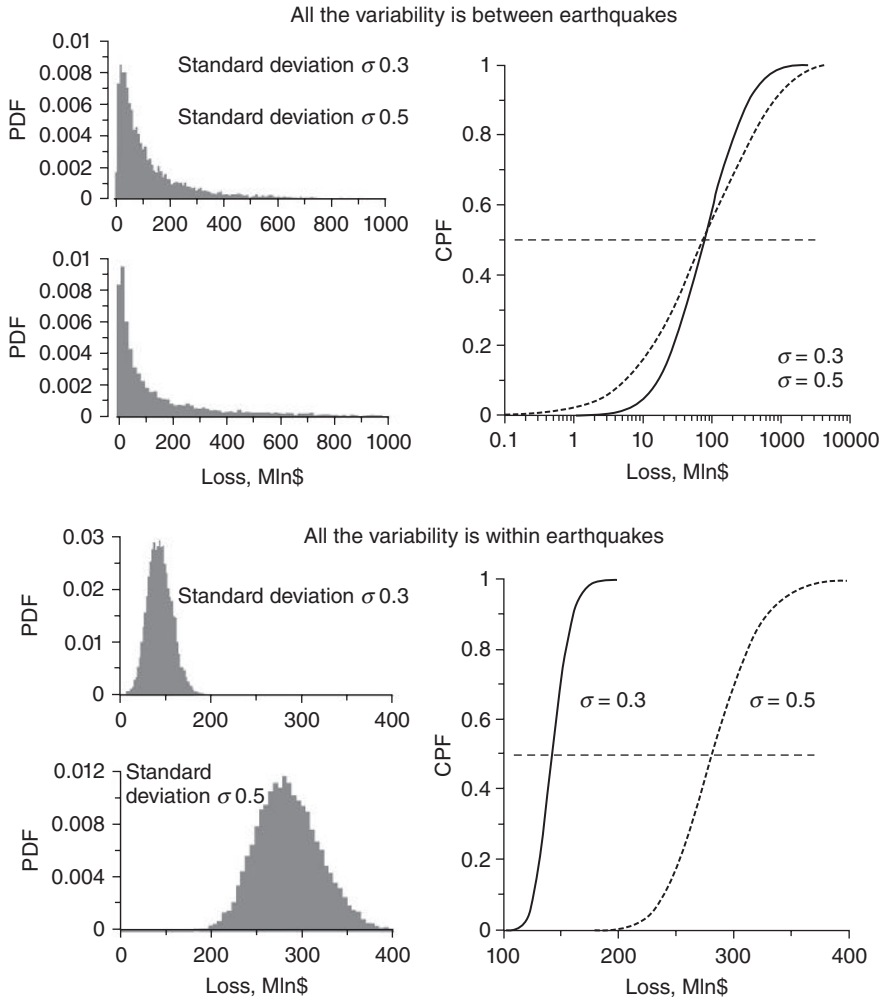
Bearing in mind that a key element of seismic hazard assessment and loss estimation is the consideration of uncertainties, which are classified as epistemic and aleatory, here it is necessary to note that the ground-motion correlation, in principle, depends on the chosen ground-motion model, because the correlation describes the behavior of residuals between the observations and predictions. Thus, different models for between-earthquake and within-earthquake correlation combined with the corresponding ground-motion models would represent a source of epistemic uncertainty. Different realizations of the random ground-motion field in terms of between-earthquake and within-earthquake residuals and ground-motion correlation represent aleatory variability.

3.3 Ground-motion correlation and seismic loss assessment

3.3.1 Example application for seismic loss estimation

To illustrate the influence of ground-motion correlation on estimations of seismic loss, we consider here the extreme cases of ground-motion correlation, i.e. all the variability between earthquakes ($\rho_\eta = 1.0$) and all the variability within earthquakes ($\rho_\eta = 0.0$; $\rho(\Delta) = 0.0$ for $\Delta > 0$ km). The loss estimations were performed for a set of hypothetical buildings mimicking a real building stock, which was constructed based on three types of buildings that are typical for Taiwan. The whole area of $22 \text{ km} \times 18 \text{ km}$ was

divided into cells of $1 \text{ km} \times 1 \text{ km}$, and different numbers of buildings were assigned, more or less randomly, to every cell. The total number of buildings in the area was 5478. An earthquake of magnitude 7.0, which located near the area, was considered and peak ground acceleration, together with the building-specific fragility curves, was used to evaluate ground shaking-induced structural damage. Details of the model and procedure may be found in Sokolov and Wenzel (2011a).



3.3 Characteristics of loss distribution (Probability Distribution Functions, PDF, and Cumulative Probability Functions, CPF) for the extreme cases of ground motion correlation estimated using 10000 simulations, single earthquake (Sokolov and Wenzel, 2011a).

For a given ground-motion amplitude generated for a cell, a single value of expected direct loss EDL_{ij} was estimated for every building. The loss values, which had been estimated for all buildings within a particular cell, were summarised to obtain a cell-specific loss. Then, a single total loss amount for a given generation of the ground-motion distribution was obtained as the sum of losses from all cells. The generated set of total loss values (10000 generations) is used for the estimation of the Probability Density Function (PDF) and Cumulative Probability Function (CPF) and the analysis of the parameters of loss distribution. The structure-to-structure correlation of damage (e.g. Lee and Kiremidjian, 2007) and the uncertainty of replacement costs (e.g. Porter *et al.*, 2002) were not considered.

Figure 3.3 shows results of loss estimation and Table 3.2 presents characteristics of loss distribution. Note that the value of total loss, which was estimated using the mean PGAs without consideration of uncertainty, is about US \$80 Mln. As can be seen, only mean values of loss do not depend on the correlation models but, of course, they are sensitive to standard deviation of ground motion.

For every particular earthquake scenario, the between-earthquake variability changes the level of ground motion at all location in similar way, i.e. the level will be lower or higher than the median estimates from a predictive equation. The larger level of ground motion at all locations will cause greater damage everywhere and higher total loss values and vice versa. Thus, if all of the ground motion variability is treated as between-earthquake ($\rho_\eta = 1.0$), this would lead to high variability of total loss. The

Table 3.2 Parameters of loss probability distribution (Mln US \$) evaluated for two extreme cases of ground-motion correlation (see text) and two values of standard deviation σ of ground-motion variation

Parameter of loss distribution	Ground-motion variability			
	All the variability is between earthquakes		All the variability is within earthquakes	
	σ 0.3	σ 0.5	σ 0.3	σ 0.5
Mean	115	290	115	290
Median	70	70	150	290
Standard deviation	140	360	9.2	35
Coefficient of variation	1.22	1.24	0.08	0.12
$P_E = 0.1$	270	580	135	330
$P_E = 0.05$	390	970	140	345
$P_E = 0.01$	670	1850	145	370

P_E = probability of exceedence.

mean and the median values of possible losses are not equal to each other in this case.

The within-earthquake variability, for every particular earthquake scenario, leads to fluctuations of ground motion from one location to another; stronger than the median estimates at one site and weaker at another site. Even if the large level of ground motion causes great damage at some locations, the damage at other locations may be small. Thus, if all of the ground motion variability is treated as within-earthquake ($\rho_\eta = 0.0$; $\rho(\Delta) = 0.0$ for $\Delta > 0$ km), the variability of total loss would be smaller than that for the other extreme case (all the variability is between earthquakes) analyzed above. However, the median values, which are affected by particularly great losses, are larger than those for the case of between-earthquake variability only. Note that the mean and the median values of possible losses are equal in this case.

3.3.2 Recent trends for seismic loss estimation studies considering ground-motion correlation

The knowledge of the loss distribution about the mean (e.g. variance or standard deviation) is very important for decision making and mitigation activities. For example, primary insurers are concerned with the central part of the distribution (mean and median values), while re-insurers deal mostly with the tail of the distribution. Several studies analyzed the influence of ground-motion uncertainty and correlation on characteristics of loss distribution for widely located building assets (portfolios) and spatially distributed structures (lifelines). Specific (scenario) earthquakes were considered by Lee *et al.* (2004), Lee and Kiremidjian (2007), Molas *et al.* (2006), Goda and Atkinson (2009), Crowley *et al.* (2008a,b), and Sokolov and Wenzel (2011a). Multiple earthquakes and loss probability curves were analyzed by Wesson and Perkins (2001), McVerry *et al.* (2004), Bommer and Crowley (2006), Park *et al.* (2007), Goda and Hong (2008b, 2009), and Sokolov and Wenzel (2011b).

Seismic loss estimation was performed using the techniques based on different ground-motion parameters, namely: modified Mercalli intensities (McVerry *et al.*, 2004; Molas *et al.*, 2006), peak ground acceleration (Wesson and Perkins, 2001; Sokolov and Wenzel, 2011a,b), and spectral acceleration or spectral displacement (Lee *et al.*, 2004; Crowley and Bommer, 2006; Lee and Kiremidjian, 2007; Park *et al.*, 2007; Crowley *et al.*, 2008a; Goda and Hong, 2008b), and using various descriptions of loss, i.e. the mean damage ratio (Wesson and Perkins, 2001; Lee *et al.*, 2004; Crowley and Bommer, 2006; Lee and Kiremidjian, 2007; Crowley *et al.*, 2008a; Goda and Atkinson, 2009; Sokolov and Wenzel, 2011b) and monetary loss (McVerry *et al.*, 2004; Park *et al.*, 2007; Goda and Hong, 2008b, 2009; Sokolov and Wenzel, 2011a).

Most of these studies considered the influence of between-earthquake and within-earthquake components adopting a single model of the between-earthquake correlation or analyzing, in addition to the model, two extreme cases (no correlation of ground motion and perfect correlation). Different models of within-earthquake correlation were used by Park *et al.* (2007), Goda and Atkinson (2009), Molas *et al.* (2006), Crowley *et al.* (2008a), and Sokolov and Wenzel (2011a,b).

Effects of variations in the between-earthquake correlation and in the within-earthquake correlation on seismic loss and damage estimations for extended objects (hypothetical portfolio) and critical elements (e.g. bridges) of a network were analyzed by Sokolov and Wenzel (2011a) for a single (scenario) earthquake and by Sokolov and Wenzel (2011b) for multiple earthquakes. They showed that the impact depends on the hazard and probability level of interest (return period). Characteristics of the analyzed portfolio or the set of critical elements of a network (e.g. seismic design and location of elements) may be of considerable importance.

The general findings obtained in the works mentioned above may be summarized as follows. The higher the within-earthquake correlation and between-earthquake correlation are, the larger the variation in losses to a portfolio and the higher probability of extreme loss values become. For the case of a single (scenario) earthquake, within-earthquake variability increases the possibility of obtaining extreme motion at one of multiple locations compared to the single-site probability. Between-earthquake variability increases the possibility of obtaining extreme motions at all considered locations simultaneously.

The variation in the shape of the within-earthquake correlation $\rho_\epsilon(\Delta)$ function may be also important (e.g. Sokolov and Wenzel, 2011a). The high rate of decrease of within-earthquake correlation with separation distance would increase influence of within-earthquake variability, which, in turn, increases possibility of large losses at particular locations.

Some applications require assessing the probability that a specific event will occur during a certain condition at least once or that several such events will occur simultaneously (joint hazard/risk). For example, there may be interest in knowing whether several vulnerable elements of a lifeline network (e.g. bridges, local substations of the power supply network or pipelines, etc.) are likely to be simultaneously affected by shaking strong enough to disable them (joint damage) or at least one element will be damaged, causing the failure of the system. Rhoades and McVerry (2001) and McVerry *et al.* (2004) analyzed the influence of between-earthquake and within-earthquake variation on such assessment; Sokolov and Wenzel (2011a,b) considered the effect of within-earthquake correlation. The increase of correlation distances, as well as the increase of between-earthquake correlation, would increase the probability of joint damage.

Summarizing the conclusions and recommendations made, for example, by Rhoades and McVerry (2001), Goda and Hong (2008b), and Sokolov and Wenzel (2011b), the following scheme for consideration of ground-motion correlation may be suggested: if information on correlation models is not available, it is first necessary to obtain upper and lower bound estimates by assuming the extreme characteristics of correlations (the perfect correlation: $\rho_\eta = 1.0$; $\rho_\varepsilon(\Delta) = 1.0$; and the uncorrelated ground motion: $\rho_\eta = 0.0$; $\rho_\varepsilon(\Delta) = 0.0$ for $\Delta > 0$ km). Second, if the difference between the estimates based on the extreme bounds is significant, one should make assumptions about the correlation based on reported models and results. However, it is necessary to bear in mind that the ground-motion correlation, in principle, depends on the chosen ground-motion model and the within-earthquake correlation structure may depend on local geology as well on azimuth-dependent attenuation along the propagation path (e.g. Sokolov *et al.*, 2010, 2012). Thus, a single generalized model of spatial correlation may not be adequate for some cases.

3.4 Future trends

The within-earthquake correlation depends on ground conditions of the sites. Baker and Jayaram (2008) and Jayaram and Baker (2009) discussed the influence of geotechnical characteristics (average shear-wave velocity on the top 30 m of the soil, V_{s30}) on the correlation for short-period ground-motion parameters. Sokolov *et al.* (2010, 2012) found a significant influence of geological conditions (both shallow and deep geology) on characteristics of the within-earthquake correlation in Taiwan (see Section 3.2.5 and Fig. 3.2). However, no systematic research on quantifying the dependence has been conducted so far. Thus, there is a necessity in introducing a model, which relates spatial correlation of ground motion parameters with characteristics of variation of geology in spatial extent. Such a model will be very useful for loss estimation in the regions, where strong motion data are scarce or not available; with the developed mode, spatial correlation structure can be characterized by sufficient geological and geotechnical databases.

Another important application of the geology-correlation models is characterization of uncertainty of Shake Maps (e.g. Wald *et al.*, 2008) using conditional simulation of ground-motion field, given observations from recording stations (Park *et al.*, 2007; Crowley *et al.*, 2008b).

Obviously, development of correlation models for ground-motion parameters, other than PGA, PGV and response spectra that may be used for regional loss estimation, such as seismic intensity and acceleration spectrum intensity (Bradley, 2010), is a fruitful area for further study.

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Ground motion selection for seismic risk analysis of civil infrastructure

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Abstract: This chapter presents a ground motion selection methodology, based on the generalized conditional intensity measure (GCIM) approach, which enables ground motion selection as part of a probabilistic seismic performance assessment. The basis of the approach, which involves the construction of, and selection of ground motions from, the conditional multivariate distribution of ground motion intensity measures (so-called GCIM distributions) is discussed. A case study is used to illustrate the methodology input requirements and salient features. Continuing developments facilitating the routine practical implementation of the methodology are discussed.

Key words: ground motion selection, generalized conditional intensity measure (GCIM) approach, seismic response bias, seismic demand curve.

4.1 Introduction

The continuing evolution toward the seismic design of engineered facilities based on their expected performance places an increased emphasis on the use of computational models to predict the seismic response of such facilities. A key consideration in time-domain computational models is selection of the input ground motions for seismic performance assessment. Many ground motion selection and/or modification methods have been proposed in literature over the past decade, and yet a lack of consensus remains (e.g. Katsanos *et al.*, 2010, and references therein).

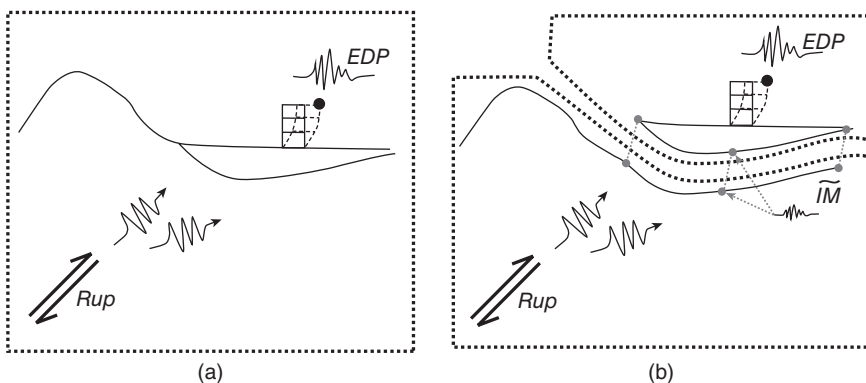
This chapter presents a ground motion selection methodology, which unlike the majority of ground motion selection methods, has a theoretically sound basis. The methodology enables ground motion selection as part of a probabilistic seismic performance assessment, and can be considered a generalization of the conditional mean spectra approach of Baker and Cornell (2006). The basis of the approach is the construction of a conditional multivariate distribution of a general set of ground motion intensity measures. Ground motions are then selected from random realizations of this multivariate distribution, with a problem-specific ‘weight-vector’ used to emphasize particular ground motion features of importance in the selection process. Subsequent to the use of the selected ground motions in

seismic response analysis, a simple procedure provides an estimate of the magnitude of any potential bias in the results of seismic response analyses, if ground motions were inadequately selected. The combination of these features of the approach makes it entirely holistic in that: any level of complexity in ground motion selection for any seismic response analysis can be exercised; users explicitly understand the simplifications made in the selected suite of ground motions; and an estimate of any bias associated with such simplifications is obtained. Finally, it is shown that the application of the methodology results in a consistent computation of the seismic demand curve irrespective of the conditioning intensity measure adopted. The theoretical details of the methodology and its salient features are presented using a particular case study. Continuing developments facilitating the routine practical implementation of the methodology are discussed.

4.2 Ground motion selection in seismic performance assessment

4.2.1 The need for ground motion selection

It is important to firstly understand the reasons why ground motion selection is needed in conventional seismic performance assessments. Two broad approaches are available to compute the seismic performance of a structure located at a particular site, as shown in Fig. 4.1. The first may be referred to as a ‘coupled’ approach (Fig. 4.1a), in which the phenomena of seismic rupture (*Rup*), wave propagation, local site and structural response are considered in a coupled fashion, and the seismic response



4.1 Comparison of: (a) ‘coupled’ and (b) ‘un-coupled’ computation of seismic response due to the occurrence of an earthquake rupture. In the latter case the problem is uncoupled via the representation of the ground motion at the interface via the term *IM* (after Bradley, 2012a).

(denoted by *EDP*) is computed. The second approach, referred to as an ‘uncoupled’ approach (Fig. 4.1b) treats the same problem in two (or possibly more) parts by introducing a conceptual boundary between the domains of (i) earthquake rupture and wave propagation; and (ii) engineering response of the local soil and structure. This uncoupled approach has numerous benefits, elaborated upon by Bradley (2012a); the most important of which is the ability to use different methodologies for each task. In particular, it may be impractical to perform earthquake rupture and wave propagation simulations for the problem under consideration due to a lack of expertise, input information (i.e. fault rupture details as well as seismic velocity structure of the propagation medium), and the perceived limitations of such models (often, the adequate simulation of high-frequency ground motion). Consequently, (simple) empirical ground motion prediction equations (GMPEs) are often used to provide the distribution of various ground motion intensity measures (represented by a intensity measure vector, *IM*) at the site of interest (e.g. peak ground acceleration). Most importantly, however, such GMPEs provide only the distribution of ground motion intensity measures, but not ground motion time histories. Hence, in such cases the problem becomes how to select ground motion time histories based on the obtained distributions of ground motion intensity measures.

4.2.2 Emphasis on probabilistic seismic hazard – consistent ground motions

Ground motions are typically selected on the basis of either deterministic or probabilistic seismic hazard analysis (DSHA and PSHA, respectively) (Bommer, 2002). For example, ground motions may be desired to represent the controlling earthquake obtained from DSHA, or to represent the ground motion hazard for one or more exceedance probabilities obtained from PSHA. While it is the author’s opinion that assessing the seismic performance of structures under deterministic and probabilistic hazard analyses provide different information and are complementary, the predominant use of PSHA in seismic design codes has resulted in PSHA – consistent ground motions being predominantly desired. The desire to obtain such ground motions is complicated by the fact that: (i) PSHA combines all the potential seismic sources which constitute a hazard at the site of interest, and hence the seismic hazard for a given exceedance probability is due to more than a single seismic source, and (ii) the ground motions at the site, for the exceedance probability considered, are not median motions expected if the seismic source was to rupture, but are a function of the features of the various seismic source and exceedance probability considered. The complexities entailed from the above two points make the selection of

ground motions consistent with a desired exceedance probability from PSHA non-trivial. The ground motion selection methodology discussed herein is applied for the selection of PSHA-consistent ground motions. However, because of its theoretical basis, minor modifications allow the methodology to also be applied to ground motion selection for scenario events (i.e. from DSHA).

4.2.3 An overview of contemporary ground motion selection

Current ground motion selection methodologies are burdened by the limited amount of information on the severity of a ground motion that is conveyed (often using a single ground motion intensity measure (*IM*), e.g. peak ground acceleration). For example, ground motion selection methods utilized in both research and engineering practice commonly select motions on the basis of their ‘match’ to some target response spectrum (e.g. see Katsanos *et al.*, 2010, and references therein). Such target response spectra maybe: (i) a design response spectra in seismic codes (ASCE, 2006; ATC-58, 2007; CEN, 2003); (ii) the response spectra of a single earthquake scenario specified in DSHA, or inferred as the governing scenario from PSHA (McGuire, 1995); or (iii) the response spectra of a single earthquake scenario conditioned on a particular spectral acceleration ordinate (Baker, 2011), among many others. Several studies have also recently proposed methods to select ground motion suites which account for both the mean and variability in response spectra for such target response spectra (Jayaram *et al.*, 2011; Kottke and Rathje, 2008; Wang, 2011).

The primary limitation of the aforementioned ground motion selection methods stems from the fact that only characteristics of the ground motion represented in terms of (elastic) spectral accelerations are considered, whereas it is well acknowledged that the severity of a ground motion, in general, depends on its intensity, frequency content and duration. Spectral accelerations, by definition represent the peak response of an elastic single-degree-of-freedom oscillator of a specific period, and therefore do not explicitly account for many other important features of ground motions, such as the duration and energy (both total and its temporal accumulation) of ground shaking, among others. It should be noted that while causal parameters, such as earthquake magnitude, source-to-site distance, etc., are often considered in addition to the target spectrum, they are only implicit factors affecting a ground motion, but are not explicit measures of the severity of the ground motion itself.

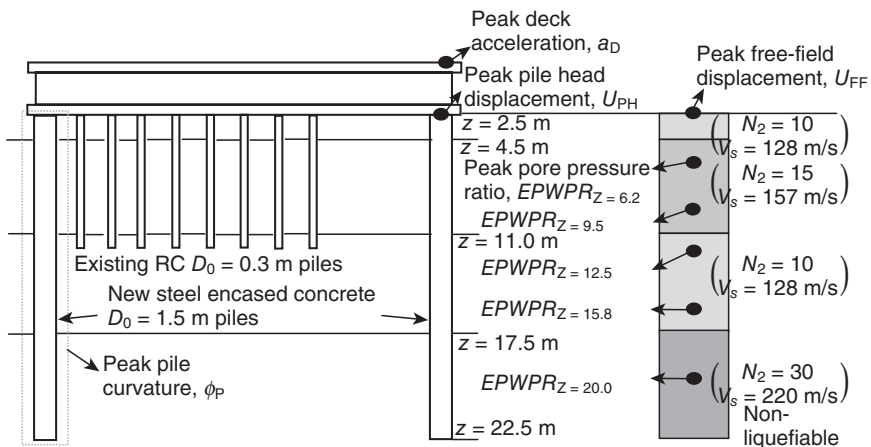
The ground motion selection methodology presented herein was developed bearing in mind the aforementioned limitations of conventional methods. This chapter is intended to provide an overview of the methodol-

ogy, whose technical details have been previously elaborated elsewhere (Bradley, 2010a, 2012a, 2012b).

4.3 Case study: bridge foundation soil system

In order to illustrate the ground motion selection procedure discussed herein, a particular case study is considered as discussed below.

The seismic response model considered is a bridge–foundation–soil system representing the transverse direction of the Fitzgerald Avenue bridge in Christchurch, New Zealand. Several previous seismic response analysis studies have been performed for this structure, in both the transverse (Bowen and Cubrinovski, 2008) and longitudinal (Bradley *et al.*, 2009a) directions. Figure 4.2 illustrates a schematic view of the transverse direction of the bridge structure and foundation soil. It can be seen that the soil stratigraphy is composed of primarily four layers with normalized Standard Penetration Test (SPT) blowcounts of $N_1 = 10$, 15, 10 and 30. As a result, the first three soil layers are potentially liquefiable when subjected to even moderate ground shaking. The bridge structure, comprising a deck, pier, and footing, is supported on both existing reinforced concrete (RC) piles of 0.3 m diameter which are founded within the liquefiable deposit, and proposed new steel encased RC piles of 1.5 m diameter which are founded in competent soils at greater depth. Details on the constitutive modelling, and other numerical details are elaborated in Bowen and Cubrinovski (2008) and Bradley (2012a).



4.2 Schematic illustration of transverse direction of the bridge–foundation–soil system (Bradley *et al.*, 2009a); and seismic response parameters of interest (after Bradley, 2012a).

For the purposes of this chapter, the aforementioned bridge–foundation–soil system is considered to overlie a hypothetical rock site in Los Angeles (latitude = 34.053; longitude: –118.243). The conditions at the rock site include a 30-m averaged shear wave velocity of $V_{s30} = 760$ m/s and a depth to the 2.5 km/s shear wave velocity horizon of $Z_{2.5} = 1.0$ km. The potential seismic sources affecting the site are as provided by the 1996 USGS earthquake rupture forecast (Frankel *et al.*, 1996).

4.4 The generalized conditional intensity measure (GCIM) approach

4.4.1 Theoretical details

Bradley (2010a) justifiably proposed that for a given earthquake scenario (i.e. given a causal earthquake magnitude, source-to-site distance, local site properties, etc.) any arbitrary vector of ground motion intensity measures, $\mathbf{IM}$, has a multivariate lognormal distribution (i.e. $\mathbf{IM}|Rup$ has a multivariate lognormal distribution, where ‘ $|Rup$ ’ indicates conditioning on a specific earthquake rupture scenario), herein referred to as the generalised conditional intensity measure (GCIM) approach. $\mathbf{IM}$ may include any scalar intensity measures of a ground motion, e.g. peak ground acceleration, pseudo-spectral acceleration, and significant duration. Based on the properties of a multivariate lognormal distribution, it then follows that the conditional distribution of $\mathbf{IM}|Rup$ given the occurrence of $IM_j = im_j$ is also a multivariate lognormal distribution. The distribution of each IM_i in $\mathbf{IM}$ conditioned on $IM_j = im_j$ can therefore be obtained from:

$$f_{IM_i|IM_j} = \sum_{k=1}^{N_{Rup}} f_{IM_i|Rup_k, IM_j} P_{Rup_k|IM_j} \quad [4.1]$$

where $f_{IM_i|IM_j} = f_{IM_i|IM_j}(im_i|im_j)$ is the probability density function (pdf) of IM_i given $IM_j = im_j$; $f_{IM_i|Rup_k, IM_j} = f_{IM_i|Rup_k, IM_j}(im_i|rup_k, im_j)$ is the pdf of IM_i given $Rup = rup_k$ and $IM_j = im_j$; and N_{Rup} is the number of potential seismic ruptures causing a hazard at the site. Given that $\mathbf{IM}|Rup$ has a multivariate lognormal distribution, it follows that $\mathbf{IM}|Rup, IM_j$ also has a multivariate lognormal distribution (Johnson and Kotz, 1972), and in particular, that for each IM_i in $\mathbf{IM}$, $IM|Rup, IM_j$ has a univariate (conditional) lognormal distribution which can be expressed as:

$$f_{IM_i|Rup, IM_j} \sim \text{LN}(\mu_{\ln IM_i|Rup, IM_j}, \sigma_{\ln IM_i|Rup, IM_j}^2) \quad [4.2]$$

where $f_X(x) \sim \text{LN}(\mu_{\ln X}, \sigma_{\ln X}^2)$ is short-hand notation for X having a lognormal distribution with mean $\mu_{\ln X}$ and variance $\sigma_{\ln X}^2$. The conditional mean and standard deviation (square root of the variance) in Equation (4.2) can then be obtained from Equations (4.3) and (4.4), respectively:

$$\mu_{\ln IM_i|Rup, IM_j} = \mu_{\ln IM_i|Rup} + \sigma_{\ln IM_i|Rup} \rho_{\ln IM_i, \ln IM_j|Rup} \epsilon_{\ln IM_j} \quad [4.3]$$

$$\sigma_{\ln IM_i|Rup, IM_j} = \sigma_{\ln IM_i|Rup} \sqrt{1 - \rho_{\ln IM_i, \ln IM_j|Rup}^2} \quad [4.4]$$

where $\mu_{\ln IM_i|Rup}$ and $\sigma_{\ln IM_i|Rup}$ are the mean and standard deviation of $\ln IM_i$ for a given rupture, which are commonly obtained from ground motion prediction equations (GMPEs) (e.g. Boore and Atkinson, 2008); $\rho_{\ln IM_i, \ln IM_j|Rup}$ is the correlation coefficient between $\ln IM_i$ and $\ln IM_j$ for a given earthquake rupture, commonly provided by empirical correlation equations (e.g. Baker and Jayaram, 2008; Bradley, 2011c); and the parameter, $\epsilon_{\ln IM_j}$, is given by:

$$\epsilon_{\ln IM_j} = \frac{\ln IM_j - \mu_{\ln IM_j|Rup}}{\sigma_{\ln IM_j|Rup}} \quad [4.5]$$

Equations (4.1)–(4.5) provide the necessary mathematical details to compute the conditional marginal distribution of IM_i given $IM_j = im_j$ for all IM_i in $\mathbf{IM}$. The multivariate distribution of $\mathbf{IM}|Rup, IM_j$ can therefore be uniquely defined by mean and standard deviation vectors of length, $\mu_{\ln \mathbf{IM}|Rup, IM_j}$ and $\sigma_{\ln \mathbf{IM}|Rup, IM_j}$, respectively; and a correlation matrix, $\rho_{\ln \mathbf{IM}|Rup, IM_j}$ (Johnson and Wichern, 2007). The i th element of the mean and standard deviation vectors are given by Equations (4.3) and (4.4). Specifically:

$$\begin{aligned} \mu_{\ln \mathbf{IM}|Rup, IM_j}(i) &= \mu_{\ln IM_i|Rup, IM_j} \\ \sigma_{\ln \mathbf{IM}|Rup, IM_j}(i) &= \sigma_{\ln IM_i|Rup, IM_j} \end{aligned} \quad [4.6]$$

and the i th row, k th column element of the correlation matrix is given by (Johnson and Wichern, 2007):

$$\rho_{\ln \mathbf{IM}|Rup, IM_j}(i, k) = \frac{\rho_{ik} - \rho_{ij}\rho_{kj}}{\sqrt{1 - \rho_{ij}^2} \sqrt{1 - \rho_{kj}^2}} \quad [4.7]$$

where $\rho_{ik} = \rho_{\ln IM_i, \ln IM_k|Rup}$ is shorthand notation for the correlation between $\ln IM_i$ and $\ln IM_k$ for a given rupture, Rup , which can be obtained from empirical correlation equations (discussed subsequently). Equations (4.3)–(4.7) provide sufficient information to uniquely characterise the multivariate distribution $\mathbf{IM}|Rup, IM_j$ (Johnson and Wichern, 2007).

It is worth noting that in the particular case in which: (i) only a single causal earthquake is considered; (ii) only spectral accelerations are considered in the vector, $\mathbf{IM}$, and the conditioning intensity measure, IM_j ; and (iii) only the conditional mean, rather than the conditional distribution is considered; the above approach is equivalent to the ‘conditional mean spectra’ approach proposed initially by Baker and Cornell (2006) and subsequently discussed by Baker (2011).

4.4.2 **IMs** which can be presently considered in ground motion selection

There are a large number of ground motion intensity measures which have been proposed over several decades as indicators of the severity of a ground motion for various seismic response analysis problems. In theory, any of these ground motion intensity measures can be considered in the proposed ground motion selection framework. Several practical requirements however, which remove many intensity measures from consideration at present, are that if a set of N_{IM_i} ground motion **IMs**, **IM**, wish to be considered then: (i) GMPEs must be available to predict the marginal mean and standard deviation of (the logarithm of) each of the N_{IM_i} **IMs** in **IM** (i.e. $\mu_{\ln IM_i|Rup}$ and $\sigma_{\ln IM_i|Rup}$); (ii) correlation equations must be available to predict the correlation between each of the N_{IM_i} **IMs** in **IM** and IM_j (i.e. $\rho_{\ln IM_i, \ln IM_j|Rup}$); and (iii) correlation equations are also required between the $N_{IM_i}(N_{IM_i} - 1)/2$ combinations of IM_i pairs in **IM** itself (i.e. $\rho_{\ln IM_i, \ln IM_k|Rup}$). Points (i) and (ii) above are required to obtain the conditional mean and standard deviation vectors (i.e. Equation (4.6)), while (iii) is further required to determine the correlation matrix of the vector **IM** $|Rup$, IM_j (i.e. Equation (4.7)). Because the availability of GMPEs for a particular IM_i is required in order to develop correlation equations involving this IM_i (e.g. see Bradley, 2011a), then it follows that the availability of correlation equations between various **IM** pairs is the governing restriction on which **IMs** may be considered in the proposed ground motion selection methodology.

Table 4.1 illustrates various intensity measure pairs for which, as far as the author is aware, empirical ground motion correlation equations have been developed. It can be seen that correlation equations are available between common peak-amplitude-based **IMs** such as $SA(T)$, PGA and PGV . Correlation equations are also available for the response-spectrum-based **IMs**: acceleration spectrum intensity, ASI (Von Thun *et al.*, 1988), spectrum intensity, SI (Housner, 1952), and displacement spectrum intensity, DSI (Bradley, 2011b), which can be considered to represent the average high-, moderate, and low-frequency intensity of a ground motion, respectively (Bradley, 2011b). In addition to these aforementioned peak-amplitude-based **IMs**, Table 4.1 also illustrates that correlations are available for **IMs** such as cumulative absolute velocity, CAV (EPRI, 1988) and arias intensity, IA (Arias, 1970), which are strongly influenced by cumulative ground motion features, and also the significant duration parameters, $Ds575$ and $Ds595$ (Bommer and Martinez-Pereira, 1999; Bradley, 2011a).

In the subsequent ground motion selection examples to follow, the intensity measure vector **IM** = { $SA(T)$, PGA , PGV , ASI , SI , DSI , CAV , $Ds575$, $Ds595$ } will be adopted (where $SA(T)$ is considered at nine periods T = {0.05, 0.1, 0.2, 0.3, 0.5, 1.0, 2.0, 5.0, 10.0}), i.e. a total of 17 **IMs** were considered. The following GMPEs were used for predicting these various **IMs**:

Table 4.1 Empirical correlation equations which can be used to predict the correlation between the logarithm of two intensity measures for a given earthquake rupture,

$\rho_{\ln IM_j, \ln IM_i | Rup}$

<i>IM</i>	<i>SA(T)</i>	<i>PGA</i>	<i>PGV</i>	<i>ASI</i>	<i>SI</i>	<i>DSI</i>	<i>CAV</i>	<i>Ds575</i>	<i>Ds595</i>	<i>IA</i>
<i>SA(T)</i>	BJ08	B11(a), B07	B11(b)	B11(a)	B11(a)	B11(c)	B11(d)	B11(e)	B11(e)	B07
<i>PGA</i>		–	B11(b)	B11(a)	B11(a)	B11(c)	B11(d)	B11(e)	B11(e)	B07
<i>PGV</i>			–	B11(b)	B11(b)	B11(c)	B11(d)	B11(e)	B11(e)	–
<i>ASI</i>				–	B11(a)	B11(c)	B11(d)	B11(e)	B11(e)	–
<i>SI</i>					–	B11(c)	B11(d)	B11(e)	B11(e)	–
<i>DSI</i>						–	B11(d)	B11(e)	B11(e)	–
<i>CAV</i>							–	B11(e)	B11(e)	–
<i>Ds575</i>		symmetric						–	B11(e)	–
<i>Ds595</i>									–	–
<i>IA</i>										–

B07 = Baker (2007); BJ08 = Baker and Jayaram (2008); B11(a) = Bradley (2011c); B11(b) = Bradley (2010c); B11(c) = Bradley (2011b); B11(d) = Bradley (2011d); B11(e) = Bradley (2011a).

Source: Bradley (2012b).

Boore and Atkinson (2008) (*SA*, *PGA*, *PGV*); Bradley (2009) (*ASI*); Bradley *et al.* (2009b) (*SI*); Bradley (2011b) (*DSI*); Campbell and Bozorgnia (2010) (*CAV*); and Bommer *et al.* (2009) (*Ds575*, *Ds595*). References for the various empirical correlation equations for each of the potential IM_i combinations are given in Table 4.1.

The ground motion selection methodology also requires the specification of a conditioning intensity measure, IM_j . Herein, $SA(3.0)$ is used as the IM_j . As noted by Bradley (2012a, 2012b), an IM_j should be selected which represents the amplitude of ground motion near the range of vibration frequencies which control the governing seismic response mechanisms. Importantly, as demonstrated by Bradley (2012a), the use of the discussed ground motion selection method can consistently achieve results which are independent of the selected IM_j .

4.5 Ground motion selection using generalized conditional intensity measure (GCIM)

The benefit of the GCIM approach is that IM/IM_j provides the exact distribution of intensity measures of potential ground motions with $IM_j = im_j$ which may be observed at the site. The GCIM distributions are therefore the ‘target’ that should be used in selecting a set of ground motions for seismic response analysis. The theory underlying a semi-automated procedure for selection of ground motions using the GCIM distribution is firstly presented below, followed by examples elucidating its salient features.

4.5.1 Realizations from the GCIM distributions

As illustrated in the previous section, the construction of the GCIM distributions involves essentially two steps: (i) determining the probability that if a ground motion was observed with $IM_j = im_j$, it was caused by rupture $Rup = rup_k$, $P_{Rup|IM_j}$; and (ii) given the observed ground motion with $IM_j = im_j$ from rupture rup_k , what is the distribution of the other ground motion intensity measures in $\mathbf{IM}$. While the previous section presented the analytical formulation of the distribution of $IM_i|IM_j$, $f_{IM_i|IM_j}$, this distribution can also be obtained via simulation. In the case of simulation, the two equivalent steps in obtaining $f_{IM_i|IM_j}$ are: (i) obtain a random earthquake rupture rup^{nsim} (where the superscript *nsim* represents the *n*th simulation) from the seismic hazard deaggregation probabilities, $P_{Rup|IM_j}$; and (ii) for this randomly drawn rupture, rup^{nsim} , obtain a random realisation of $\mathbf{IM}$, $\mathbf{IM}^{nsim}$, from the GCIM distribution. In order to select ground motions consistent with the GCIM distribution, use is made of this simulation approach elaborated below.

The first level of the simulation approach involves obtaining a random rupture, rup^{nsim} , from the deaggregation probability mass function, $P_{Rup|IM_j}$. This is done by drawing a uniform random number over the interval [0,1], and then obtaining the corresponding number from the cumulative form of $P_{Rup|IM_j}$. The second level of the procedure is the classic generation of correlated random variables from a multivariate normal distribution. As such, first an uncorrelated standard normal random vector is simulated, $\mathbf{u}^{nsim}$ (i.e. each of the N_{IM_i} elements in $\mathbf{u}^{nsim}$ is drawn from a standard normal distribution independently). Using this vector of uncorrelated random variables, a correlated vector is obtained from:

$$\mathbf{v}^{nsim} = \mathbf{L}\mathbf{u}^{nsim} \quad [4.8]$$

where $\mathbf{L}$ is the Cholesky decomposition of the correlation matrix (i.e. $\rho_{\ln IM_i|IM_j,Rup} = \mathbf{L}\mathbf{L}^T$ where ^{T} is the vector transpose). Using this correlated random vector, each of the random IM_i values in $\mathbf{IM}^{nsim}$ can be obtained from:

$$\ln IM_i^{nsim} = \mu_{\ln IM_i|Rup,IM_j} + \sigma_{\ln IM_i|Rup,IM_j} v_i^{nsim} \quad [4.9]$$

where $v_i^{nsim} = \mathbf{v}^{nsim}(i)$ is the *i*th element of $\mathbf{v}^{nsim}$; and $Rup = rup^{nsim}$.

4.5.2 Selecting ground motions using the $\mathbf{IM}^{nsim}$ realizations

The random realizations $\mathbf{IM}^{nsim}$ obtained based on the procedure in the previous section will be consistent with the ‘target’ GCIM distribution (i.e. the set of simulated $\mathbf{IM}^{nsim}$ vectors will have the same mean, standard devia-

tion and correlation structure as the theoretical ‘target’ distribution, $\mathbf{IM}|\mathbf{IM}_j$). Therefore, for each realization, ideally a ground motion can be obtained (either from databases of as-recorded motions (e.g. Chiou *et al.*, 2008) or via ground motion simulation methodologies (e.g. Boore, 2003)) which has the identical intensity measure values as the simulated vector $\mathbf{IM}^{nsim}$. However, because of several restrictions (e.g. the finite number of observed ground motions available) finding such a ground motion will not generally be possible. Therefore, for each random realization, it is instead attempted to select the ground motion which has the minimum misfit, or residual, as compared to the randomly realized vector $\mathbf{IM}^{nsim}$. If the misfit between a particular m th ground motions intensity measure vector, $\mathbf{IM}^m$, and the realized intensity measure vector, $\mathbf{IM}^{nsim}$, is relatively small, then it is likely that the empirical distribution of the selected ground motion set, $\widehat{\mathbf{IM}}$, will be representative of $\mathbf{IM}|\mathbf{IM}_j = im_j$ obtained from the GCIM approach (since $\widehat{\mathbf{IM}}$ will be statistically similar to $\mathbf{IM}^{nsim}$, and $\mathbf{IM}^{nsim}$ is drawn from $\mathbf{IM}|\mathbf{IM}_j$).

The residual of a m th prospective ground motion compared to the $nsim$ th random realization from the GCIM distributions, $r_{m,nsim}$, is given by:

$$r_{m,nsim} = \sum_{i=1}^{N_{IM_i}} w_i \left[\frac{\ln IM_i^{nsim} - \ln IM_i^m}{\sigma_{\ln IM_i | Rup^{nsim}, IM_j}} \right]^2 \quad [4.10]$$

where IM_i^{nsim} is the value of IM_i from the $nsim$ th simulation; IM_i^m is the value of IM_i of the m th prospective ground motion; w_i is the i th element of the vector of normalized weights (i.e. $\sum w_i = 1$) which indicate the importance of each of the IM s in ground motion selection; $\sigma_{\ln IM_i | Rup^{nsim}, IM_j}$ is the conditional standard deviation of $\ln IM_i$ given $IM_j = im_j$ for the randomly drawn rupture of the $nsim$ th realization; and N_{IM_i} is the number of IM s in the vector $\mathbf{IM}$. The functional form of the residual accounts for several factors (Bradley, 2012b); the most important of which it is the hierarchy as to the importance of each of the different intensity measures in the vector $\mathbf{IM}$. For example, it may be considered that, for a particular seismic response problem, the cumulative effects of a ground motion are less important than the ground motion amplitude. In such a case, it would be desired to place a larger ‘weight’ on minimizing the misfit between the intensity measures indicative of ground motion amplitude than those indicative of ground motion cumulative effects. Hence it can be seen that for each realization of the GCIM distributions, the residual for all of the prospective ground motions can be computed via Equation (4.10), and the ground motion with the smallest residual value selected.

Because of the fact that ground motions are selected based on random realizations of the vector $\mathbf{IM}$, then the set of selected ground motions will generally be different for different sets of realizations. For ground motion sets of a reasonable size which are required for performance-based earth-

quake engineering problems (e.g. 25 ground motions for a given IM_j level), the variability of ground motion sets due to the use of realizations is likely insignificant relative to the variability among ground motions themselves (i.e. within a single set). However, for smaller ground motion sets (e.g. a set of seven ground motions is commonly adopted based on seismic design guideline specifications) the variability between different sets of records selected using different sets of realizations may be undesirably large. In order to avoid this issue, for each ground motion set obtained, an overall residual is computed based on the difference between the ‘target’ conditional distributions and those of the selected ground motion set:

$$R = \sum_{i=1}^{N_{IM_i}} w_i (D_{IM_i})^2 \quad [4.11]$$

where D_{IM_i} is the Kolmogorov–Smirnov (KS) test statistic, which defines the difference between the target GCIM distribution and the empirical distribution of the selected ground motion set for IM_i . Specific details of the KS test statistic are presented subsequently in section 4.5.4. Equation (4.11) can be evaluated for different ground motion sets selected using the aforementioned methodology, and the set with the lowest R value chosen for use in the seismic response analysis problem considered.

It is noted that as the proposed ground motion selection algorithm requires only knowledge of the values of IM^m for the m th prospective ground motion, then the methodology can be used in the selection of either as-recorded amplitude scaled ground motions and also synthetic/simulated ground motions. The application to as-recorded ground motions is presented below, while readers are referred to Bradley (2012b) for details regarding synthetic/simulated ground motions.

4.5.3 Application to amplitude-scaled as-recorded ground motions

Because the GCIM distribution, $IM|IM_j = im_j$, is the conditional distribution of IM given $IM_j = im_j$, then for consistency, all ground motions used for seismic response analysis should have $IM_j = im_j$ (Bradley, 2010a). Thus, in the case of ground motions which are amplitude-scaled, a first step is the determination of the amplitude-scale factor which is required to modify the ground motion to have $IM_j = im_j$. For the majority of ground motion intensity measures, which scale analytically with amplitude-scale factor (Bradley, 2012b), the relationship between the scaled and unscaled values of IM_i , i.e. IM_i^{scaled} and IM_i^{unscaled} , respectively, are given by:

$$IM_i^{\text{scaled}} = IM_i^{\text{unscaled}} (SF)^\alpha \quad [4.12]$$

where SF is the amplitude scale factor; and α is an integer. Intensity measures, such as PGA , PGV , and SA , among others, scale linearly with ampli-

tude scale factor and therefore have $\alpha = 1$. On the other hand, intensity measures, such as IA , scale quadratically with scale factor (i.e. $\alpha = 2$), and significant duration, D_s (e.g. Bradley, 2011a, equation 1), is independent of amplitude scale factor (i.e. $\alpha = 0$). Hence, in the situation considered here, where the unscaled intensity measure IM_j of the m th ground motion can be computed, $IM_j^{m, \text{unscaled}}$ and the target value is defined by the value for which the GCIM distributions are constructed (i.e. $IM_j = im_j$), then the required scale factor for ground motion m is given by rearranging Equation (4.12) to obtain:

$$SF_m = \left(\frac{im_j}{IM_j^{m, \text{unscaled}}} \right)^{1/\alpha} \quad [4.13]$$

Note that SF_m is independent of the individual realized ground motion values, IM^{sim} , and therefore the scale factor for each prospective ground motion need only be computed once (for $IM_j = im_j$). As a result, each amplitude-scaled ground motion will have a unique intensity measure vector IM^m , with elements obtained from:

$$IM_i^m = IM_i^{m, \text{unscaled}} (SF_m)^\alpha \quad [4.14]$$

Hence, the intensity measure values (which scale analytically with scale factor) of all ground motions can be pre-computed (before executing the ground motion selection algorithm), which significantly reduces computational demands.

4.5.4 Representativeness of the selected motions compared to the GCIM distributions

Because ground motion selection is desired for a finite number of N_{gm} ground motions, then a comparison of the appropriateness of the N_{gm} ground motions as representative of $IM_i | IM_j$ (for all i) must be done so using statistical goodness-of-fit tests. The majority of ground motion intensity measures of engineering interest are continuous variables, and the statistical test for such cases is presented here, while tests for discrete variables can be found elsewhere (Bradley, 2010a).

For continuous intensity measure variables, the adequacy of a particular suite of ground motions with respect to a pre-defined theoretical distribution of a single IM_i can be assessed by the KS goodness-of-fit test (e.g. Ang and Tang, 2007, p. 293–296). The KS test measures the absolute difference between the theoretical cumulative distribution function (CDF) and the empirical distribution function (EDF) of the sample, which is mathematically given by:

$$D_{IM_i} = \max_{IM_i} |F_{IM_i | IM_j}(im_i | im_j) - S_{N_{gm}}(im_i)| \quad [4.15]$$

where D_{IM_i} is the KS test statistic for IM_i based on N_{gm} ground motions; $F_{IM_i|IM_j}(im_i | im_j)$ is the theoretical CDF (obtained from the GCIM approach); and $S_{N_{gm}}(im_i)$ is the EDF (of IM_i) of the suite of ground motions selected. The null hypothesis that the distribution of IM_i of the suite of ground motions is representative of the theoretical distribution is rejected, if D is greater than the critical KS test statistic for a given confidence level, α . The result of the KS test can be easily interpreted graphically based on whether or not the EDF of the selected ground motions intersects the KS test statistic 'bounds'. Further explanation is given in Bradley (2010a).

4.5.5 Explicit measures of ground motion vs implicit causal parameters

Conventional ground motion selection is typically based on the consideration of explicit measures of the ground motion and implicit criteria based on ground motion causal parameters. The most common example of this is the explicit consideration of the pseudo-response spectral ordinates of a ground motion, and the implicit consideration of causal parameters, such as rupture magnitude, source-to-site distance, focal mechanism, site shear wave velocity, among others, which is prevalent in both research and practice (ASCE, 2006; Bommer and Acevedo, 2004; Katsanos *et al.*, 2010; Kottke and Rathje, 2008; NZS 1170.5, 2004; Wang, 2011). Such causal parameters are often considered because of the acknowledged limitation of the explicit parameters used. For example, a range of applicable earthquake magnitudes, source-to-site distances, etc. is typically specified to obtain a set of prospective ground motions before selecting from this prospective set based on spectral shape, because it is acknowledged that response spectra provide little insight into the cumulative and duration phenomena of a ground motion (Bommer and Acevedo, 2004; Bommer *et al.*, 2004; Bradley, 2011a; Wang, 2011).

The use of implicit causal parameters in aiding ground motion selection has two primary limitations. Firstly, the applicability criteria for each causal parameter are often ad-hoc. For example, although moment magnitude is generally recognized as an important causal parameter when selecting ground motions, there is generally no consensus as to what is the range of moment magnitude which will lead to unbiased results without being overly restrictive in reducing the size of the prospective ground motion dataset. This is further exacerbated in the case of a probabilistic analysis in which multiple causative ruptures typically contribute to the seismic hazard for a given exceedance probability. Secondly, and more importantly, causal parameters such as moment magnitude, source-to-site distance, etc., are parameters that are only implicit factors affecting a ground motion, but are not explicit measures of the severity of the ground motion itself.

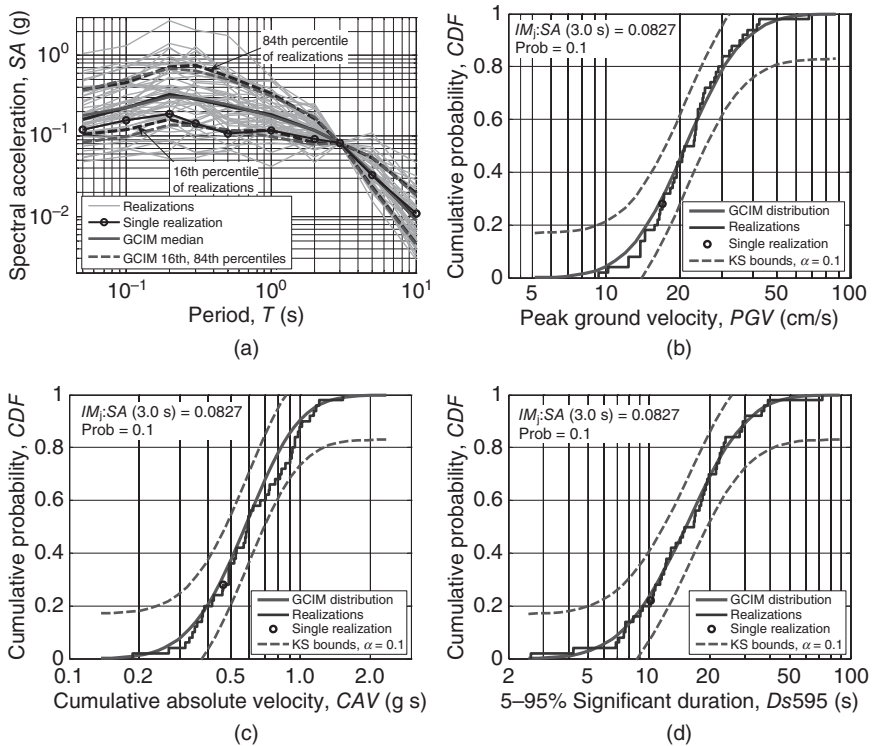
The fact that the GCIM methodology and the proposed GCIM-consistent ground motion selection algorithm can account for a large number of (explicit) intensity measures of a ground motion, means that it is possible to select ground motions without the need for ad-hoc causal parameter criteria. For example, ground motions resulting from small magnitude ruptures will likely provide a large residual when fit to a single realization of a GCIM distribution which is dominated by larger magnitude events because, even if they happen to have a similar response spectral shape (which is not affected by amplitude scale factor), they are likely to have other intensity measures, such as significant duration, which significantly differ from those which would be expected from larger magnitude events (Bradley, 2010a) (recall that the GCIM distributions are computed considering the deaggregation of the seismic hazard, i.e. $P_{Rup|IM_j}$). It is worth bearing in mind that the previous statement implies that GCIM distributions and ground motion selection are conducted using a sufficient number of IM_s ; further details are given in the subsequent applications.

4.6 Application of the ground motion selection methodology

4.6.1 Realizations from the GCIM distributions

Figure 4.3 illustrates 50 IM^{sim} realizations obtained from simulation of the distribution $IM|IM_j$ for a value of $IM_j = SA(3.0) = 0.0827\text{ g}$, which has an exceedance probability of 10% in 50 years for the case study site considered. In Fig. 4.3b–d, the realizations of PGV , CAV and D_{s595} are depicted in the form of their EDF (Ang and Tang, 2007), while in Fig. 4.3a the individual realizations of multiple $SA(T)$ ordinates are shown. In each of the plots in Fig. 4.3, the theoretical GCIM distribution (Bradley, 2010a), from which the realizations have been simulated, is also shown. In Fig. 4.3b–d, it can be seen that the simulated realizations are representative of the analytical distributions provided by the GCIM approach, as evident from the EDF of the realizations lying ‘inside’ the KS rejections bounds (for a confidence level of 0.1). The similarity of the median, 16th and 84th percentiles of the realizations and GCIM distribution illustrated in Fig. 4.3a also demonstrates the representativeness of the realized SA ordinates (explicit comparisons can be made for individual $SA(T)$ ordinates similar to Fig. 4.3b–d, although this is not shown for brevity).

Figure 4.3a also illustrates the specific values obtained from a single realization, IM^{sim} . For this single realization, it can be seen that for $T < 3\text{ s}$ the realized SA ordinates are similar to the 16th percentile of the GCIM distribution, while for $T > 3\text{ s}$ the ordinates are similar to the median of the GCIM distribution. Figure 4.3b–d also illustrate that for this single



4.3 Realizations of the vector IM/IM_j for the particular site at an exceedance probability of 10% in 50 years: (a) pseudo-acceleration response spectra, SA; (b) peak ground velocity, PGV; (c) cumulative absolute velocity, CAV; and (d) 5-95% significant duration, Ds_{595} (after Bradley, 2011b).

realization the PGV , CAV , and Ds_{595} values are below the median of the GCIM distribution. This single realization therefore serves to illustrate the effect of correlation between the different IM_s considered.

4.6.2 Effect of weight vector on semi-automated ground motion selection

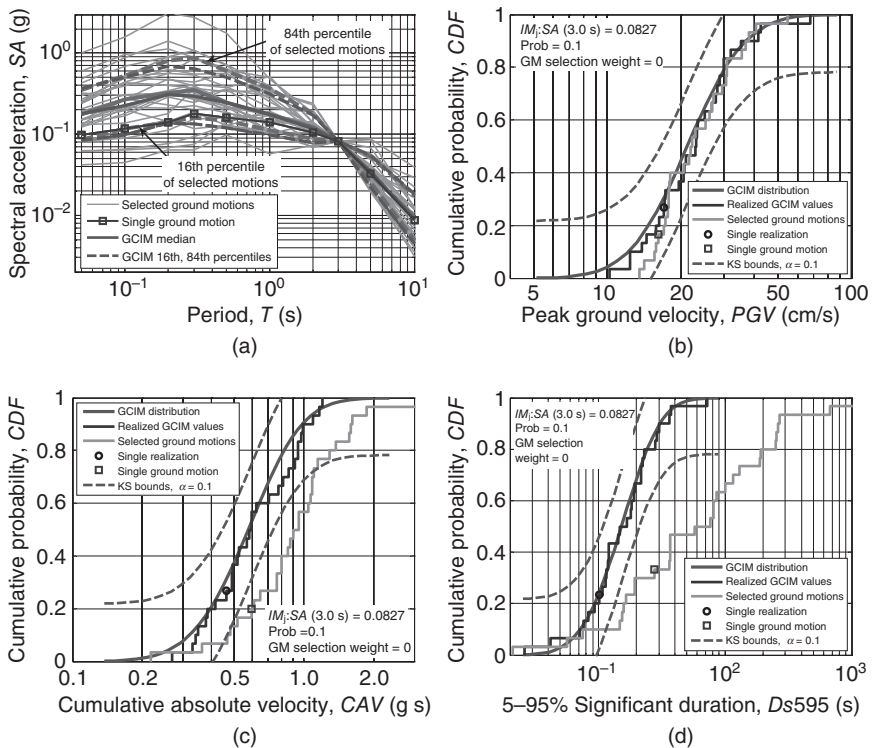
In order to illustrate the effect of different intensity measures in the selection of ground motions, this section presents details of ground motions selected using the previously presented algorithm for two different weight vectors, w_i . In both cases, the NGA database (Chiou *et al.*, 2008) of as-recorded ground motions is used to provide the set of prospective ground motions which can be selected. No ‘screening-criteria’ of causal earthquake parameters, such as ranges of magnitude or source-to-site distance were

used. This includes the specific filtering applied to a ground motion record (since, for example, excessive filtering of long period ground motion content will produce reduced spectral amplitudes at long periods, resulting in the ground motion having a high residual when compared with a realized **IM** vector). A total of 30 ground motions are selected using the first 30 realizations presented in the previous section.

Firstly, consider the contemporary procedure of ground motion selection based solely on the response spectral ordinates of a ground motion. As such, the weight vector considered was:

$$w_i(IM_i) = \begin{cases} 1/9 & \text{if } IM_i = SA \\ 0 & \text{otherwise} \end{cases} \quad [4.16]$$

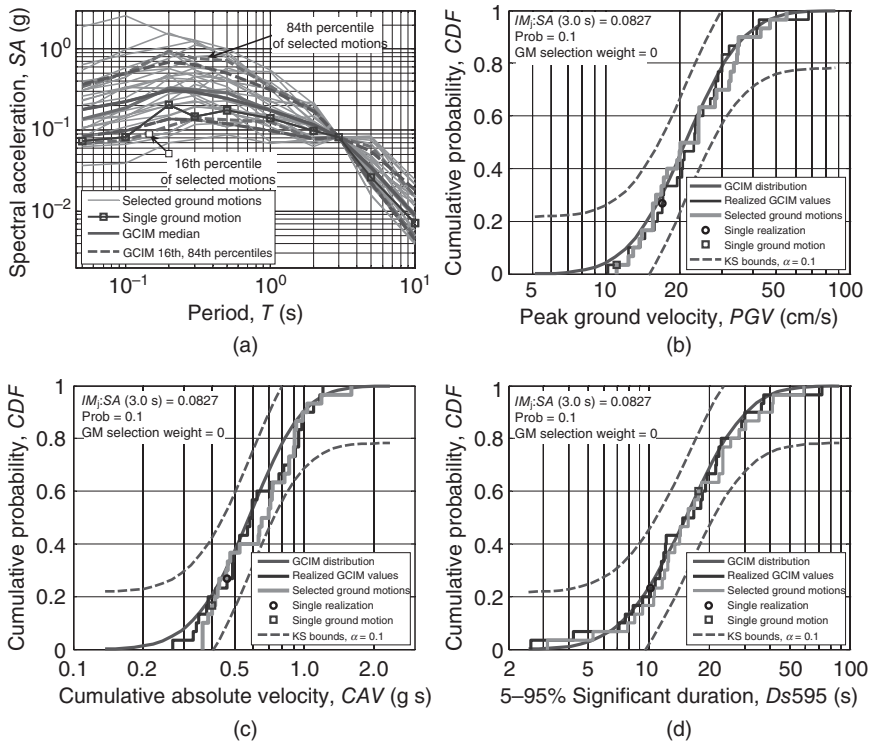
That is, a weight of 1/9 was given for each of the nine different *SA* ordinates in **IM**, and zero weight for all other *IM*_s. Figure 4.4 illustrates the details of the ground motion set which was selected based on the realizations in Fig. 4.3 using the weight vector given by Equation (4.16). Firstly, it can be seen that the response spectra of the selected ground motions, depicted in Fig. 4.4a provide a good representation of the GCIM distribution as indicated by the similarity between the median, 16th and 84th percentiles of *SA* values of the selected ground motion set and the GCIM distribution. The observation that the selected ground motions provide an adequate representation of *SA* ordinates from the target GCIM distribution is not surprising, given that ground motions whose spectral ordinates (at the nine periods considered) most closely match the individual realizations were selected. This can be explicitly appreciated by comparing the similarity of the response spectra of the single ground motion in Fig. 4.4a which was selected based on the single realization presented in Fig. 4.3a. Figure 4.4b–d illustrate the EDFs of *PGV*, *CAV* and *Ds595*, respectively, of the selected ground motions (as well as the random realizations) in comparison with the GCIM distributions. It should be reiterated that none of these three *IM*_s were considered in selecting the ground motions (i.e. the ‘GM selection weight’ in the figure text for each *IM*_{*i*} is 0.0). Despite this, it can be seen in Fig. 4.4b that the *PGV* distribution of the selected ground motion records, for this particular example, is consistent with the GCIM distribution, an observation which was also the case for *PGA*, *ASI*, *SI*, and *DSI*. This observation can possibly be attributed to the fact that *SA* ordinates over a wide period range have been explicitly considered (i.e. Equation (4.16)), and that *PGV* exhibits a moderately strong correlation of approximately 0.8 with *SA* over the period range 0.5–4.0 s (Bradley, 2011d). In contrast, Fig. 4.4c and d illustrate that the ground motions selected based on the weight vector of *SA* ordinates alone have biased distributions of *CAV* and *Ds595* (indicated by the EDF of the selected ground motions intersecting the KS rejection bounds). The specific reasons for this result



4.4 Semi-automated ground motion selection based on the realizations in Fig. 4.3 using a weight vector with uniform weights for nine SA ordinates: (a) pseudo-acceleration response spectra, SA; (b) peak ground velocity, PGV; (c) cumulative absolute velocity, CAV; and (d) 5-95% significant duration, Ds_{595} (after Bradley, 2012b).

are discussed subsequently; however, at present it is important to emphasize that if this ground motion set was used for seismic response analyses in which the cumulative effects and duration of strong ground motion are important, then they will lead to biased results (see Bradley, 2010a, p. 1337, for further details).

From the observation in Fig. 4.4, that ground motion selection based on only SA ordinates lead to selected ground motions with biased distributions of CAV, Ds_{575} and Ds_{595} , an alternative weight vector was considered in the ground motion selection algorithm by including these three additional IM s. In assigning the weight vector values, it was decided to give 70% of the total weight uniformly for SA ordinates, and 10% weight to each of CAV, Ds_{575} and Ds_{595} . This decision to associate 70% weight to amplitude-based IM s (at various frequencies) and 30% weight to IM s which account for cumulative and duration effects is done so for the purposes of



4.5 Semi-automated ground motion selection based on the realizations in Fig. 4.3 using a weight vector with nine *SA* ordinates, as well as *CAV*, *Ds575* and *Ds595*: (a) pseudo-acceleration response spectra, *SA*; (b) peak ground velocity, *PGV*; (c) cumulative absolute velocity, *CAV*; and (d) 5–95% significant duration, *Ds595* (after Bradley, 2012b).

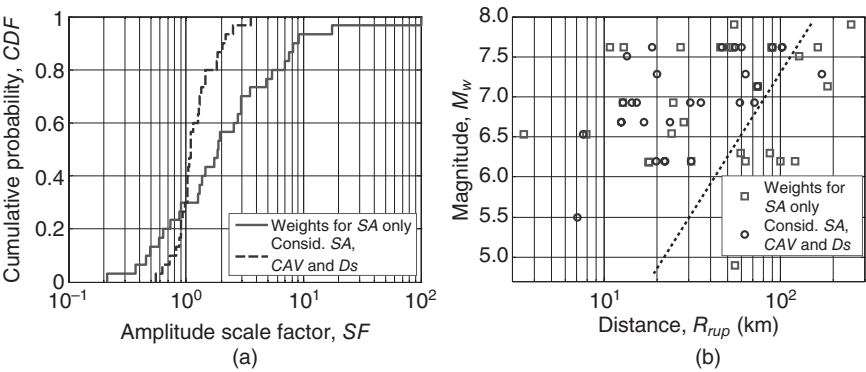
the example. As already noted, in a practical context, the decision should be guided based on those *IM*_{*s*} to which the seismic response of the facility considered is most sensitive (based on the analyst's insight and experience). Hence, mathematically the weight vector is given by:

$$w_i(IM_i) = \begin{cases} 0.7 * 1/9 & \text{if } IM_i = SA \\ 0.1 & \text{if } IM_i = CAV, \\ & Ds575, Ds595 \\ 0 & \text{otherwise} \end{cases} \quad [4.17]$$

Figure 4.5 illustrates the details of the ground motion set which was selected using the weight vector given by Equation (4.17). Similar to Fig. 4.4, it can be seen that the response spectra and *PGV*'s of the selected ground motions depicted in Fig. 4.5a provide a good representation of the

GCIM distribution, as indicated by the similarity between the median, 16th and 84th percentiles of SA values of the selected ground motion set and the GCIM distribution. However, in contrast to Fig. 4.4, Fig. 4.5c and d illustrate that the ground motions selected based on the weight vector given by Equation (4.17) have unbiased distributions of CAV and $Ds595$. Although not shown, the selected ground motions also have unbiased distributions of PGA , ASI , SI , DSI and $Ds575$. Hence, unlike those ground motions selected in Fig. 4.4 (based on SA ordinates only) the ground motions selected in Fig. 4.5 will not lead to biased seismic response analyses in cases where the CAV , $Ds575$ and $Ds595$ values of a ground motion are important (Bradley, 2010a, Figure 7).

Despite the above ground motion selection algorithm giving no consideration to (implicit) causal parameters, it is insightful to examine such parameters in order to explain the observations in Fig. 4.4 and Fig. 4.5. Figure 4.6 and Table 4.2 provide details regarding the amplitude scale factors and the magnitude and source-to-site distance ($M_w - R_{rup}$) distribu-



4.6 Comparison of: (a) amplitude scale factors; and (b) magnitude–distance distribution of the ground motions selected based on two different weight vectors (after Bradley, 2012b).

Table 4.2 Comparison of the scale factors, SF , and mean magnitude and distance of the selected ground motion sets using two different weight vectors

Weight vector	Considering SA only	Considering SA , CAV , Ds
Median SF	1.9	1.1
[10th,90th] SF percentiles	[0.5,9]	[0.8,2]
Mean magnitude, $\overline{M_w}$	6.94	6.99
Mean distance, $\overline{R_{rup}}$	65.4	44.5

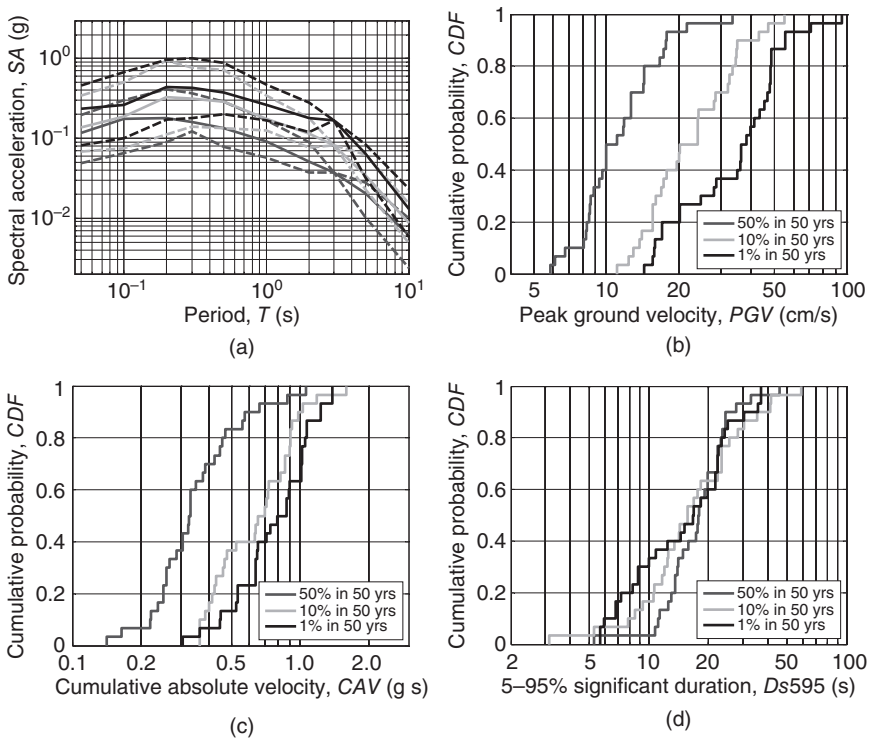
Source: Bradley (2012b).

tion of the ground motions selected in Fig. 4.4 and Fig. 4.5. It can be seen that there is a noticeable difference in the distribution of amplitude scale factors required for these ground motion sets, with median, 10th and 90th percentile scale factors of 1.9, 0.5 and 9.0, respectively, for the ground motions selected considering only *SA* ordinates, and median, 10th and 90th percentile scale factors of 1.1, 0.8 and 2.0, respectively, for the ground motions selected considering *SA*, *CAV*, *Ds575* and *Ds595*. That is, in the latter case the amplitude scale factors are much closer to 1.0, than the former case. Further insight into the reason for the larger scale factors required for the ground motion selected based on *SA* ordinates only can be obtained from the $M_w - R_{rup}$ distribution in Fig. 4.6b. Figure 4.6b illustrates that, for this particular example, when selecting ground motions based on *SA* ordinates only, the selected ground motions tend to have larger source-to-site distances for a given M_w . For example, 10 of the 30 ‘*SA* only’ ground motions have a $M_w - R_{rup}$ combination ‘below’ the line indicated in Fig. 4.6b, while there is only a single ground motion selected based on considering *SA*, *CAV*, *Ds575* and *Ds595*. As a result, such ground motions tend to have a lower *SA*(3.0) amplitude, thus requiring a larger amplitude scale factor. Given that significant duration increases, on average, with source-to-site distance (Bommer *et al.*, 2009) this is the likely reason for the bias observed in the *CAV*, *Ds575* and *Ds595* distributions of the ground motions selected considering *SA* ordinates only (i.e. Fig. 4.4c and d).

4.6.3 Ground motion selection for different exceedance probabilities

Following the results obtained in Fig. 4.5, the ground motion selection algorithm was used to select ground motions for two other exceedance probabilities of 50% and 1% in 50 years. All other details used to obtain the selected set of ground motions in Fig. 4.5 were retained (in particular, the weight vector is given by Equation (4.17)). Similar to Fig. 4.4 and Fig. 4.5, sets of ground motions for these two additional exceedance probabilities were obtained with unbiased distributions of the considered intensity measures. Hence, for clarity in the figures to follow, comparisons between the ground motions selected at the three exceedance probabilities omit the underlying GCIM distributions and realizations.

Fig. 4.7 compares the ground motions selected for the three different exceedance probabilities considered. In Fig. 4.7a, the statistics of the *SA* ordinates of the selected ground motions are depicted via the median, 16th and 84th percentiles, while in Fig. 4.7b–d, the EDF is used. Table 4.3 provides a comparison between the median IM_j values of ground motions selected for those IM_j s presented in Fig. 4.7. It is worth noting that the 50%, 10% and 1% probability of exceedance values of $IM_j = SA(3.0)$ were 0.038,

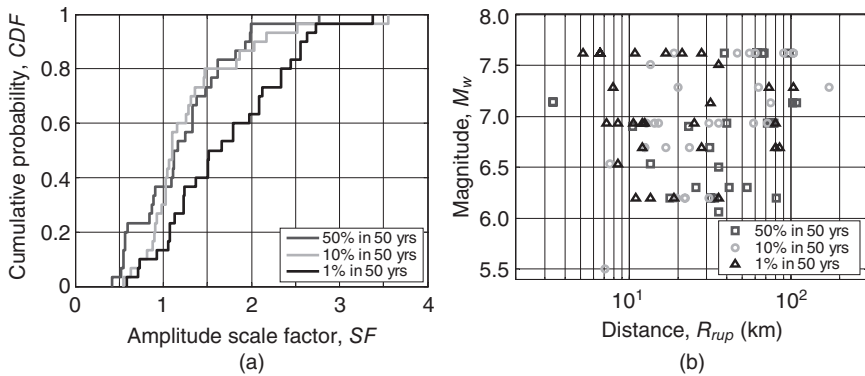


4.7 Distributions of ground motions selected for three different exceedance probabilities: (a) pseudo-acceleration response spectra, SA ; (b) peak ground velocity, PGV ; (c) cumulative absolute velocity, CAV ; and (d) 5–95% significant duration, D_{s595} (after Bradley, 2011b).

Table 4.3 Median IM_i values of the selected ground motions for various exceedance probabilities (ratios with the 10% in 50 years value are shown in brackets)

Exceedance probability	$SA(0.1)$	$SA(0.3)$	$SA(1.0)$	$SA(3.0)$	PGV	CAV	D_{s595}
50% in 50 yrs	0.18 (0.95)	0.16 (0.52)	0.09 (0.5)	0.038 (0.46)	10.5 (0.48)	0.33 (0.49)	17.9 (1.1)
10% in 50 yrs	0.19	0.31	0.18	0.083	22.0	0.68	15.7
1% in 50 yrs	0.27 (1.4)	0.43 (1.4)	0.27 (1.5)	0.17 (2.0)	37.5 (1.70)	0.84 (1.2)	16.8 (1.1)

Source: Bradley (2012b).



4.8 Comparison of: (a) amplitude scale factors and (b) magnitude–distance distribution of the ground motions selected for three different exceedance probabilities (after Bradley, 2012b).

0.083 and 0.17 g, respectively. That is, the $SA(3.0)$ amplitude changes by a factor of approximately 2, as the exceedance probability level increases each time. In Fig. 4.7a and Table 4.3, it can be seen that the largest difference between the SA ordinates of the selected ground motions for the three exceedance probabilities is for $SA(3.0)$ and decreases as the vibration period of SA tends to be away from $T = 3.0$ s. Figure 4.7b illustrates that the median PGV value of the selected ground motions at the 50% and 1% exceedance probabilities have ratios of 0.48 and 1.7 with the median PGV value for 10% exceedance probability. Similarly, Figure 4.7c illustrates that the ratio of the median CAV values for the 50% and 10% exceedance probabilities is 0.49, while the same ratio of the 1% and 10% exceedance probabilities is 1.2 (notably less than the equivalent ratio for PGV , for example). Figure 4.7d illustrates that, in contrast to the other aforementioned IM_s , the median values of $Ds595$ are similar for the three different exceedance probabilities. A thorough explanation for the IM_i properties of the selected ground motions for the 50%, 10%, and 1% exceedance probabilities observed in Fig. 4.7 is given in Bradley (2012b).

Similar to Figs 4.6, Fig. 4.8 compares the amplitude scale factors and $M_w - R_{rup}$ distributions of the selected ground motion sets for the three different exceedance probabilities. For the 50%, 10% and 1% exceedance probabilities, it can be seen that the median scale factors of the selected ground motion sets were 1.14, 1.10 and 1.60, respectively. It can also be seen that the EDFs of scale factors for the 50% and 10% exceedance probabilities are similar. The fact that the EDF of scale factors for the 1% exceedance probability is slightly larger than that for the 50% and 10% exceedance probabilities is likely a reflection of the reduced number of as-recorded ground motions (in the prospective database) with large $SA(3.0)$ values.

Despite this, comparison with Fig. 4.6a illustrates that even for this rare exceedance probability, the scale factors of the selected ground motions are notably less than those that were obtained (for the 10% exceedance probability) when selecting ground motions based on SA ordinates alone.

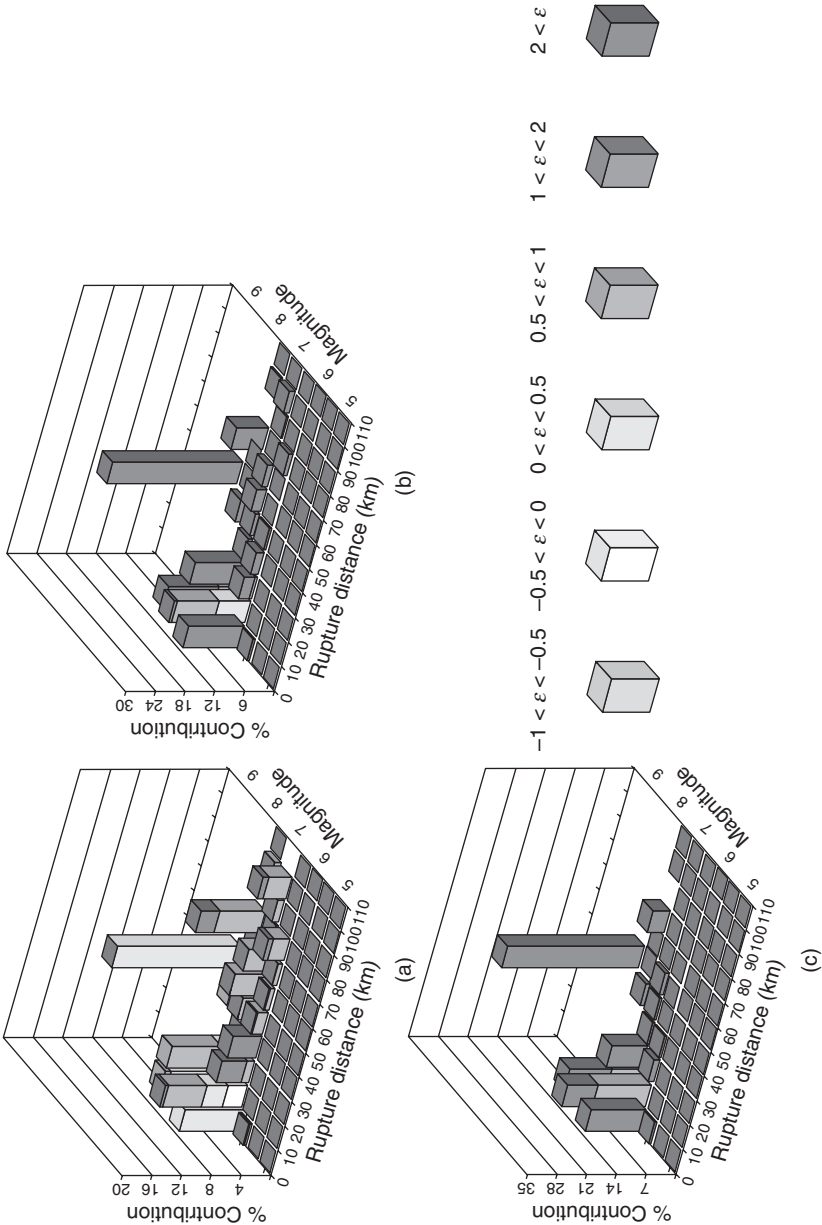
Figure 4.8b illustrates the $M_w - R_{rup}$ distribution of the ground motions selected for these three exceedance probabilities. It can be generally seen that while the distribution of causal magnitude (M_w) is similar, the ground motions selected for the rarer exceedance probabilities tend to have smaller source-to-site distances (R_{rup}). In order to add further meaning to the results presented in Figs 4.8b, 4.9 illustrates the seismic hazard deaggregation results for $SA(3.0)$ for the 50%, 10%, and 1% exceedance probabilities for which the three aforementioned ground motion sets have been selected. Comparison of the deaggregation results in Fig. 4.9 illustrates that as the exceedance probability reduces, the contribution of near-source magnitude 6.5–7.5 ruptures and a large magnitude rupture at approximately 55 km increases. Table 4.4 provides a comparison between the mean M_w and R_{rup} values from deaggregation with those of the selected ground motion sets. Again it is emphasized that no causal parameters (i.e. M_w , R_{rup} and SF , among others) were considered in the ground motion selection algorithm. Despite this, it can be seen in Table 4.4 that the agreement between the mean M_w and R_{rup} values from deaggregation and the selected ground motion sets is excellent with differences of approximately 0.1, 0.1, and 0.2 magnitude units and 5, 10 and 1 km distance units for the 50%, 10% and 1% exceedance probabilities, respectively.

The above results serve to illustrate that the proposed ground motion selection algorithm can be used without the use of implicit causal parameter criteria, and still select ground motions with causal parameters which are approximately similar to those from seismic hazard deaggregation, as well as amplitude scale factors which are not significantly different from 1.0 (Bradley, 2012b).

Table 4.4 Mean values of magnitude, M_w , and source-to-site distance, R_{rup} , of the selected ground motions in comparison with those obtained from seismic hazard deaggregation

Exceedance probability	50% in 50 yrs		10% in 50 yrs		1% in 50 yrs	
	Deagg.	Selected motions	Deagg.	Selected motions	Deagg.	Selected motions
Magnitude, M_w	6.99	6.87	7.07	6.99	7.10	6.87
Distance, R_{rup}	44.5	49.2	35.2	44.5	30.9	29.4

Source: Bradley (2012b).

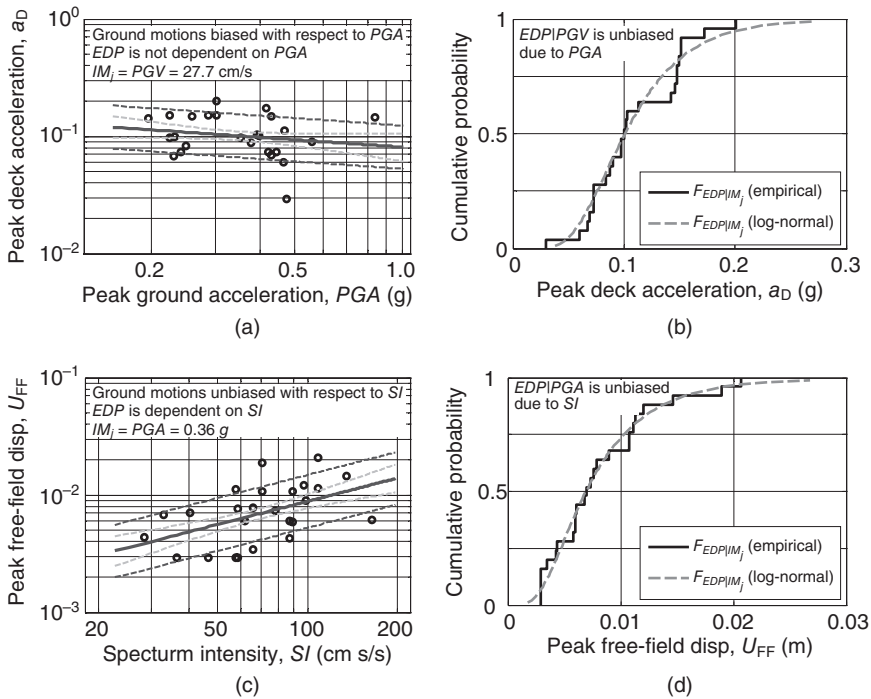


4.9 Deaggregation of the SA(3.0) seismic hazard for exceedance probabilities of: (a) 50% in 50 years; (b) 10% in 50 years; and (c) 1% in 50 years (after Bradley, 2012b).

4.7 Checking for bias in seismic response analysis due to ground motion selection

In the selection of ground motion records, it was necessary to define a weight vector, w_i , specifying the hierarchy of importance of various ground motion intensity measures in **IM**. As a result, the set of selected ground motions may have an empirical distribution, for one or more intensity measures which were given a relatively low (or even zero) weighting, which differs from the theoretical distribution provided by the GCIM approach. Such a weighting would have been assigned on the assumption that the particular intensity measure represented aspects of the ground motion which are not important in the considered seismic response model. Such an assumption can be easily checked, and if incorrect an estimate of bias can be obtained. Bradley (2010a) provides the theoretical details behind such bias estimation, and emphasis here is placed on the illustration of this procedure. The ground motion sets discussed below were obtained by Bradley (2012a).

Figure 4.10 illustrates two of the possible three outcomes from examining bias of the distribution of seismic demand due to the selected set of ground motions used in seismic response analysis (for a different case study structure). In the first case, Fig. 4.10a illustrates the dependence of the peak deck acceleration, a_D , as a function of the *PGA* of ground motions which were scaled to $PGV = 27.7$ cm/s (10% exceedance in 50 years). For this particular exceedance probability, the ground motion set used to obtain the results in Fig. 4.10 had a minor bias in the distribution of *PGA* values of the ground motions scaled to this value of *PGV* (see Bradley, 2012a, figure 4a). Figure 4.10a however illustrates that a_D is not dependent on the *PGA* values of the selected ground motions, and therefore there is no bias in the distribution of $EDP|IM_j$ due to *PGA* (Figure 4.10b). Figure 4.10c illustrates the second possible outcome where, unlike Fig. 4.10a, it can be seen that for ground motions scaled to $IM_j = PGA = 0.36$ g, the distribution of peak free-field displacement, U_{FF} , has a significant dependence on the *SI* values of the selected ground motions. However, despite this dependence of $U_{FF}|PGA$ on *SI*, because the distribution of ground motion selected were consistent with the theoretical distribution (see Bradley (2012a, figure 9d)), the distribution of the demand is also unbiased (Fig. 4.10d). Bradley (2010a) presents an example of the third possible outcome, in which both the seismic demand considered is dependent on a particular intensity measure, and the selected ground motions have a distribution of this intensity measure which is significantly different from the theoretical distribution, consequently resulting in a biased distribution of seismic demand that can be approximately estimated.



4.10 Examples of checking for bias in the distribution of seismic demand due to improper ground motion selection: (a) & (b) where the seismic demand is not dependent on the intensity measure; and (c) & (d) where the seismic demand is dependent on the intensity measure, but the ground motions selected are not biased (after Bradley, 2012a).

4.8 Seismic demand curve computation

PSHA is often conducted to assess the likelihood of specific values of ground motion intensity being exceeded at a particular site. For a particular structure located at such a site it is also of interest to compute the likelihood of structural response exceeding some particular value (which may imply the onset of a particular level of damage). Such a relationship defining the likelihood of exceedance of a particular measure of seismic response is referred to as a seismic demand curve. The seismic demand curve is a useful metric in quantifying the seismic performance of a structure due to ground shaking, because it accounts for both the likelihood of various levels of ground motion shaking at a site (i.e. from PSHA) and also the relationship between ground shaking and seismic response, including its uncertainty. For a vector of seismic responses (so-called engineering demand parameters, **EDP**), the seismic demand curve can be computed by:

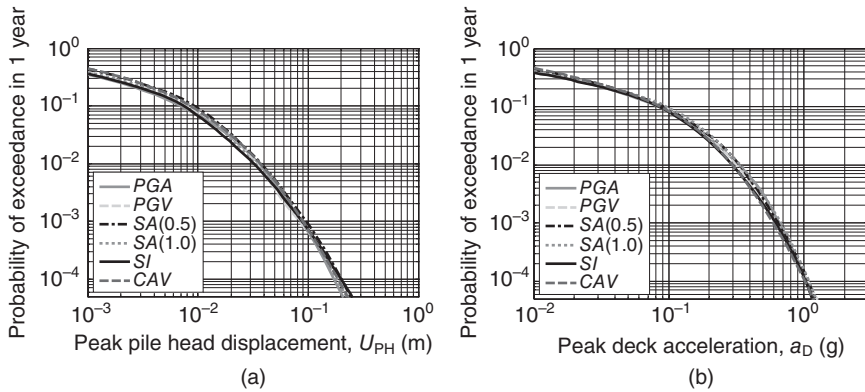
$$\lambda_{EDP}(\mathbf{edp}) = \int_{IM_j} P_{EDP|IM_j}(\mathbf{edp}|im_j) \left| \frac{d\lambda_{IM_j}(im_j)}{IM_j} \right| dIM_j \quad [4.18]$$

where $\lambda_{EDP}(\mathbf{edp})$ is the seismic demand curve of **EDP**, and quantifies the rate of exceedance of various values of the seismic responses considered; $\lambda_{IM_j}(im_j)$ is the rate of exceedance of the conditioning intensity measure, IM_j , obtained from PSHA; and $P_{EDP|IM_j}(\mathbf{edp}|im_j)$ is the probability of the vector of seismic responses, **EDP**, exceeding specific values **edp**, which is obtained from the set of seismic response analyses conducted using ground motions selected by the aforementioned methodology for $IM_j = im_j$. The assumption behind the computation of the seismic demand curve as given by Equation (4.18) are elaborated upon in Bradley (2012a), but can be considered to be consistently satisfied if ground motions are selected using the discussed methodology, which practically requires six key steps (Bradley, 2012a):

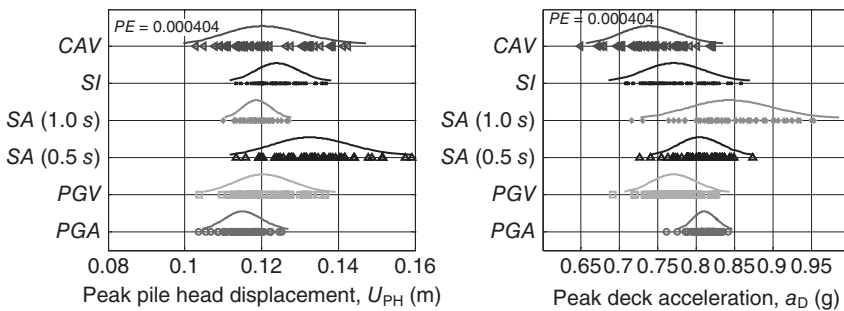
- Selection of the set of intensity measures **IM**, which sufficiently represent the ground motion severity.
- Choose a conditioning intensity measure, IM_j , from **IM** and compute the ground motion hazard $\lambda_{IM_j}(im_j)$.
- Selection of ground motions based on the GCIM distribution $f_{IM|IM_j}(\mathbf{im}|im_j)$ in Equation (4.1).
- Perform seismic response analysis and use the results to parametrize $P_{EDP|IM_j}(\mathbf{edp}|im_j)$ (e.g. Bradley, 2012a).
- Ensure that the considered **IM**, is sufficient with respect to **EDP**.
- Compute the seismic demand curve via Equation (4.18).

Because of its simplicity in implementation, the *IM*-based approach is the most common methods for computation of the seismic demand curve (as given by Equation (4.18)). One observed detrimental consequence of the *IM*-based approach is that the practical computation of the seismic demand curve appears to be dependent on the intensity measure used in its computation (Bradley, 2012a). Bradley (2012a) demonstrated theoretically the independence of the seismic demand curve on the choice of conditioning intensity measure, and noted that inconsistent seismic demand curves would be obtained, if ground motions selected were inconsistent with the GCIM distribution, $f_{IM|IM_j}(\mathbf{im}|im_j)$, and the seismic response analyses are sensitive to such intensity measures.

Figure 4.11 presents examples of the seismic demand curve for the case study structure computed using six different conditioning intensity measures for peak pile head displacement, U_{PH} , and peak deck acceleration, a_D (Bradley, 2012a). It can be seen that the demand curves computed via different IM_j s are the same in a qualitative sense, consistent with the theory previously presented. Furthermore, the majority of the differences between



4.11 Computation of the seismic demand curve using various conditioning intensity measures, IM_s for: (a) peak pile head displacement, U_{PH} ; and (b) peak deck acceleration, a_D . (modified from Bradley, 2012a).



4.12 Comparison of the uncertainty in seismic demand curve for an annual exceedance probability of 4.04×10^{-4} (i.e. 2% in 50 years) due to the finite number of ground motions used for each intensity measure level: (a) peak pile head displacement; and (b) peak deck acceleration. (after Bradley, 2012a).

the seismic demand curves depicted in Fig. 4.11 can be explained due to the uncertainty resulting from the finite number of ground motions used as illustrated in Fig. 4.12 (Bradley, 2012a). It can be seen that the uncertainty in the estimated demands due to bootstrap sample of the finite number of seismic response analyses is large relative to the difference between the mean values of the demand obtained using different conditioning intensity measures.

4.9 Software implementations

Despite the fact that many studies have highlighted the differences between the uniform hazard spectra (UHS) and individual earthquake scenarios, and therefore its inappropriateness for use in ground motion selection (see Bradley, 2010a, and references therein), it remains the most common ‘target’ for selecting ground motion records, most likely due to its presence in seismic design codes. The reluctance of the earthquake engineering profession to depart from UHS-based selection of ground motions is arguably, because of its simplicity to implement relative to methodologies with sounder theoretical bases.

In order to improve the ease with which the GCIM approach can be utilized, therefore increasing its potential for uptake in earthquake engineering practice, the method was implemented in the open-source software, OpenSHA (Bradley, 2010b; Field *et al.*, 2003). OpenSHA is an object-oriented, web- and GUI-enabled and freely available software developed as a joint venture between SCEC and USGS (www.opensha.org/). A MATLAB implementation of the ground motion selection algorithm (using the resulting output file from the aforementioned OpenSHA implementation) used for obtaining the results in this manuscript is available from the author upon request. The algorithm is computationally efficient, taking approximately 1 s to obtain a set of 30 recorded ground motions (on a 4 GB RAM 1.40 GHz dual core laptop computer).

4.10 Conclusions and future trends

As with any emerging method, the successful adoption of the proposed ground motion select methodology requires both: (i) demonstration of its capability and (ii) its ease of application in a practical environment. This chapter, as well as the previously presented material in Bradley (2010a, 2012a, 2012b), essentially provide the underlying theoretical details and salient features of the methodology, as well as its application to a single case study. However, clearly further case studies using the methodology are warranted for further demonstration of the benefits relative to alternative methodologies.

The ease of application in a practical environment requires both adequate software in which the methodology is implemented, as well as the availability of the required input information for the particular problems under consideration. As noted in the previous section, in an effort to address the first point, GCIM distributions and random realizations for use in the proposed methodology have been implemented in the open-source software OpenSHA, while a MATLAB version of the ground motion selection aspects of the methodology is also available. It is intended to implement

this algorithm in OpenSHA upon further investigation into user-interface preferences and interfacing with as-recorded ground motion databases (e.g. the PEER NGA database), as well as further investigation on incorporation of simulated ground motions.

As was noted with respect to Table 4.1, in order for ground motion intensity measures to be utilized in this methodology, empirical prediction equations must be available. While the examples in this chapter made use of 17 different ground motion intensity measures, there is still scope for the consideration of other intensity measures which may be important for particular problems. Furthermore, it is well known that GMPEs for different tectonic regions vary significantly, such that a GMPE for an active shallow crustal tectonic region is not applicable for a subduction slab tectonic region. Empirical ground motion and correlation equations are most readily available for active shallow crustal tectonic regions, but are lacking for other tectonic regions (Bradley, 2012b). Hence, there is a clear potential to improve the application of the proposed ground motion selection methodology for subduction interface, subduction slab and stable continental regions, by developing empirical prediction equations for various intensity measures due to earthquakes in such regions. It is envisaged that the implementation of the GCIM method in OpenSHA will facilitate the application of the ground motion selection method to other tectonic regions because of its open-source nature.

For regions in which the necessary empirical ground motion and correlation equations currently exist (e.g. active shallow crustal tectonic regions), the benefits provided by the proposed ground motion selection methodology can be more widely distributed by providing ‘default’ ground motion sets for common seismic response problems (e.g. mid-rise structures) in large population centres. This will enable projects which do not possess the budget for ‘site-specific’ ground motion selection, to at least employ ‘region-specific’ ground motion selection, and will further facilitate the uptake of the methodology by a wider audience.

Finally, it was noted at the beginning of the chapter that the methodology described here is oriented toward the selection of ground motions consistent with PSHA outputs (i.e. to quantify seismic performance for one or more exceedance probabilities, including computation of the seismic demand curve). However, it was also mentioned that quantifying the seismic performance of a structure given a particular seismic rupture (i.e. obtained from a deterministic seismic hazard analysis) provides complementary information to that of the probabilistic case. The ground motion selection methodology presented can easily be adapted for a particular scenario event; in such a case, the conditional distribution $f_{IM|IM_j}(im|im_j)$ based on which ground motions were selected is simply replaced by the unconditional distribution, $f_{IM}(im)$.

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Assessing and managing the risk of earthquake-induced liquefaction to civil infrastructure

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Abstract: This chapter presents a framework for managing earthquake-induced liquefaction for infrastructure assets using a risk-based approach. At a site-specific level, a design has to demonstrate that the specified performance criteria can be met, both from a safety and operational perspective. However, large infrastructure systems such as pipelines, highways and railways must consider the whole range of geohazards along their route, of which liquefaction is only one. A framework is presented for the assessment of liquefaction risk, using existing tools and methodologies for assessment of both liquefaction hazard and of the infrastructure response to liquefaction occurrence. The risk framework considers the likelihood of different modes of liquefaction-induced ground failure and their consequence to infrastructure systems. The chapter presents a useful overview of liquefaction risk assessments in practice, with a focus on simple high-level approaches that support informed, but not risk-averse, decision making.

Key words: liquefaction, permanent ground deformation, liquefaction-induced damage, liquefaction hazard, liquefaction risk, risk framework.

5.1 Introduction

Recent destructive earthquakes, e.g. Kobe (1995), Chi-Chi, Taiwan (1999), Kocaeli, Turkey (1999) and Christchurch, New Zealand (2010 and 2011) are prime examples of the potential for earthquake-induced liquefaction to cause significant damage and disruption to civil infrastructure. Numerous cases of significant damage to buildings, bridges, roads, pipelines and ports resulting from the failure of the underlying soils were observed in these and many other earthquakes.

Methodologies and approaches for the assessment of liquefaction potential are the subject of significant and ongoing research, and this chapter briefly summarises some of the extensive body of technical literature in this area. The assessment of the potential for liquefaction is presented in the wider context of a *risk management framework* for civil infrastructure systems. A risk management approach is important for owners and

operators of civil infrastructure to understand liquefaction in the context of the risk it presents, and therefore make appropriate decisions in terms of risk treatment requirements. Liquefaction risk is the product of the *likelihood* of liquefaction-related hazards (of a certain magnitude), and the *consequence* in terms of the effect that liquefaction hazards will have on the system performance. Put simply, liquefaction hazard alone does not necessarily present a problem to infrastructure owners and operators; however, where the consequence of the liquefaction is unacceptable to the overall performance, risk mitigation measures may be required.

The framework presented in this chapter could be used for any natural or man-made hazards that are likely to impact on infrastructure projects. The focus of the paper is on civil infrastructure systems, which could include:

- transportation networks (road, rail, airports and seaports);
- water and wastewater networks; and
- energy generation and distribution networks.

Figure 5.1 illustrates the effects of liquefaction on infrastructure.

There are particular features of such infrastructure systems that merit special consideration in a risk framework, such as their significant linear extent, which means they pass through many different geological settings and are therefore vulnerable to many geohazards. For individual buildings on the other hand, the risk presented by liquefaction potential is reasonably well constrained and, provided that the overall performance requirements of the building are clearly defined, liquefaction risks can be managed through appropriate foundation design. The challenges for infrastructure systems, due to their wide geographical distribution, are greater, and the required performance is often more complex.

The stages of the risk management framework presented in this chapter follow the standard steps (Free *et al.*, 2006) of:

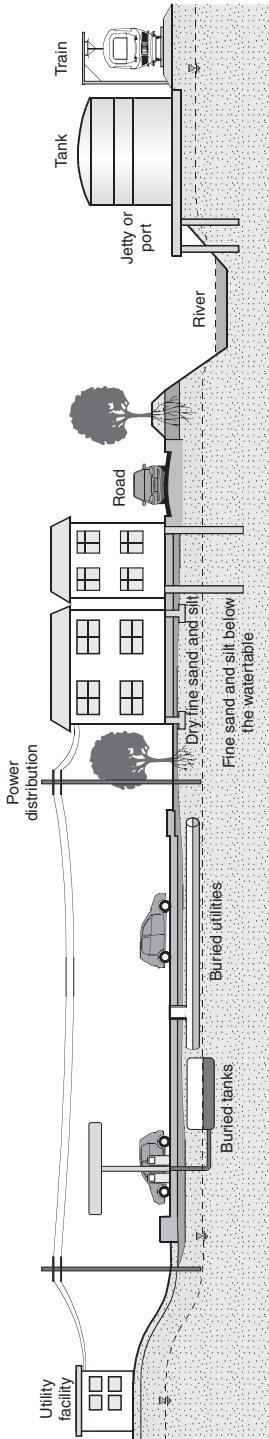
- risk assessment (hazard identification, risk estimation, risk evaluation);
- risk treatment;
- risk acceptance; and
- risk communication.

The risk terminology used in this chapter follows the definitions provided in ISO Guide 73. The focus of this chapter is on the risk assessment stage of the framework, with the subsequent stages being dependent on the specific requirements of the owners/operators.

5.1.1 A framework for liquefaction risk assessment

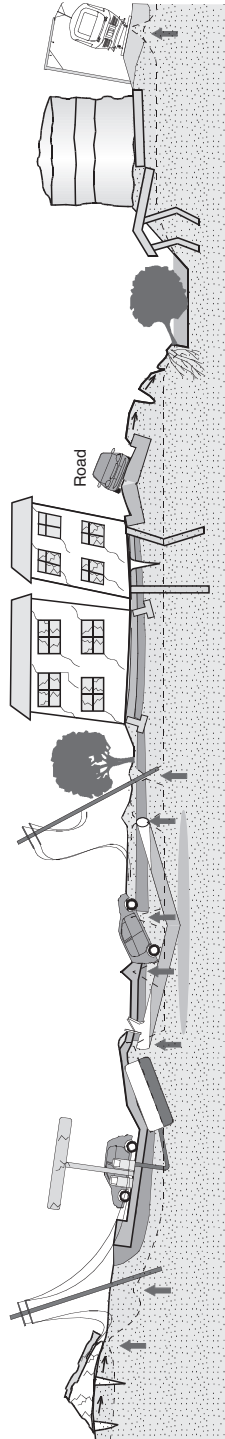
As described above, the *assessment* of liquefaction risk is the focus of this chapter, in terms of likelihood and its impact. The framework presented in

Before the earthquake



Areas of flat and gently sloping, low lying land with high groundwater support buildings and roads, rail, buried utilities and tanks under normal conditions.

During and after the earthquake



Translational failure

of slopes and undermining of foundations and infrastructure.

Flotation of buried structures

Tanks, pipes and manholes are buoyant in liquefied ground and disturb or break through to surface. Pipes rupture, causing flooding and contamination.

Ground settlement

Ground loss due to cyclic densification and/or erosion of sand ejecta.

Bearing failure

Full or partial bearing failure due to reduction in bearing capacity, resulting in differential settlement tilting or over-turning.

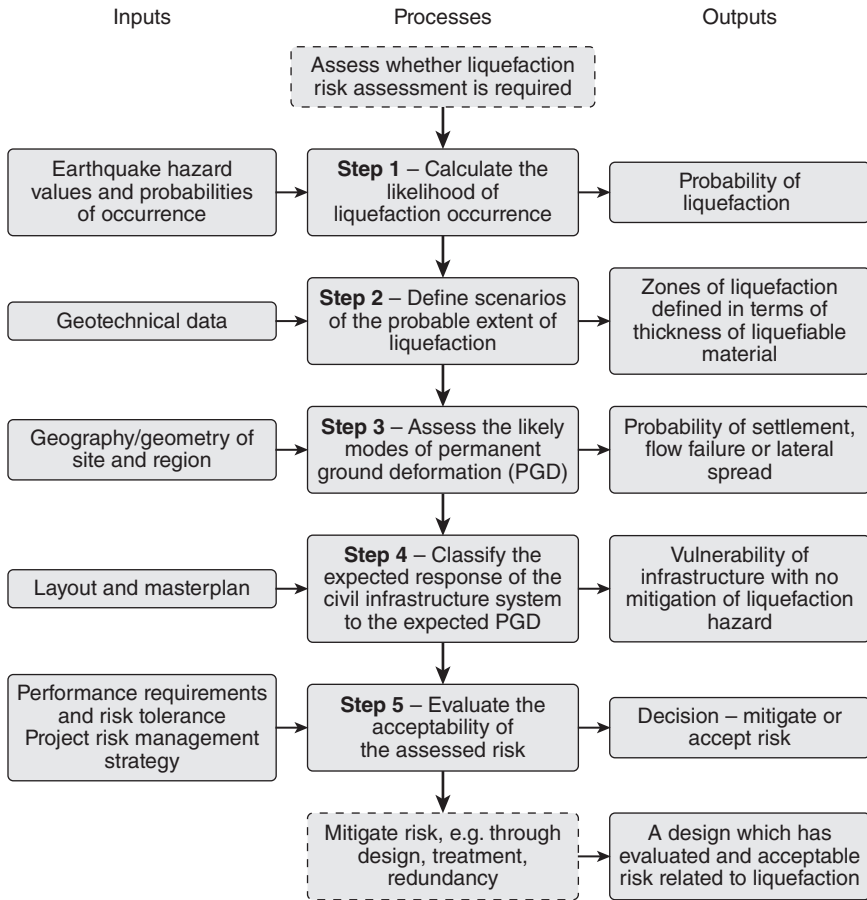
Lateral spreading

Ground movement toward free face of river bank, coast or man-made cutting. Cracks along banks or cutting extend inland, damaging buildings and infrastructure and partial closure of waterways.

Embankment failure

Full or partial bearing failure of ground beneath embankment or levee leading to translational or rotational failure and damage to infrastructure.

5.1 Liquefaction and its effects on infrastructure.



5.2 Outline framework for a risk-based approach to designing a civil infrastructure system to resist liquefaction.

Fig. 5.2 shows the key steps involved in assessing liquefaction risk, which are discussed in further detail in the following sections. To fully define the risk, detailed assessments of liquefaction potential, which make up the main body of knowledge in this area (e.g. Idriss & Boulanger, 2008), are in fact only one component of the overall framework, and the consequences of liquefaction require a similar level of understanding. There is a far smaller body of work with regard to the consequences of liquefaction, and this chapter provides guidance to inform the reader how to proceed with their risk management and also identifies aspects where researchers could make valuable contributions to fill gaps in the methodology in the future.

5.2 Hazard identification

The framework presupposes that the presence of soil *susceptible* to liquefaction has been identified, based on historical, geological or geotechnical criteria (e.g. Youd & Perkins, 1978, Seed *et al.*, 2003). The hazard identification stage should not only evaluate that there is potential for liquefaction to occur and the potential lateral and vertical extent over which it will occur, but also how this will manifest itself at a site, and how it could potentially damage infrastructure.

The following two sections describe the assessment of liquefaction potential and consequent permanent ground deformation in more detail. Figure 5.1 provides a summary of the potential liquefaction hazards and potential responses of infrastructure.

5.2.1 Liquefaction potential

The evaluation of liquefaction potential is based on the comparison of seismic demand on a soil with capacity of the same soil to resist liquefaction. The seismic demand is expressed in terms of the cyclic stress ratio (CSR), which is the ratio of the induced horizontal cyclic stresses normalised by the vertical consolidation stress (Equation 5.1).

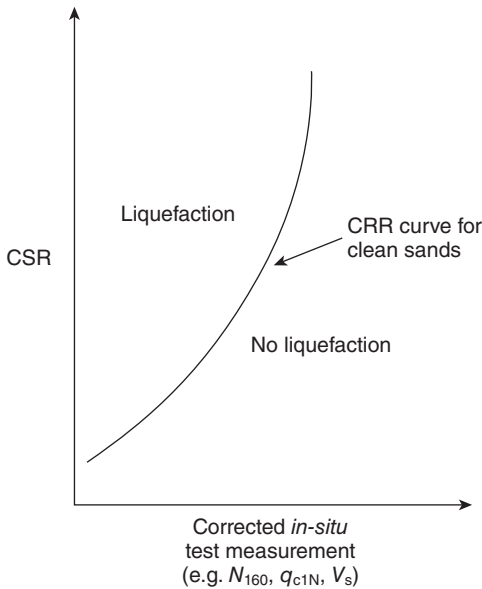
$$\text{CSR} = \frac{\tau_{\text{av}}}{\sigma'_v} = 0.65 \cdot \frac{a_{\text{max}}}{g} \cdot \frac{\sigma_v}{\sigma'_v} \cdot \frac{r_d}{\text{MSF}} \quad [5.1]$$

where: a_{max} is the peak ground acceleration at the surface, g is acceleration due to gravity, σ_v/σ'_v is the ratio of total to effective vertical stress in the soil layer, r_d is a depth-based stress reduction coefficient and MSF is a magnitude scaling factor.

The 'liquefaction resistance' is expressed in terms of the cyclic resistance ratio (CRR), which is evaluated based on laboratory testing or, most commonly, on *in-situ* testing.

A database of case histories in which evidence of liquefaction has or has not been observed is used to relate the earthquake-induced CSR for each site and the *in-situ* test results available. A CRR curve that bounds the conditions at which liquefaction has historically been observed can then be drawn (e.g. Fig. 5.3), which allows the factor of safety against liquefaction potential to be evaluated for a given *in-situ* resistance, and a given CSR.

It should be clarified that the definition of liquefaction occurrence in the above-described procedure is based on field observations, mainly at ground surface level, of typical liquefaction effects (sand boils, lateral spreading, settlements, etc.) which is obviously rather subjective. Among the available *in-situ* tests, the Standard Penetration Test (SPT) was the first test used for



5.3 Schematic diagram showing the simplified approach for assessing liquefaction potential of a soil.

the evaluation of the liquefaction potential and hence the corresponding empirical charts are based on a larger set of case histories. On the other hand, the Cone Penetration Test (CPT) provides continuity of data over depth which allows the identification of even thin liquefiable layers. Given the degree of uncertainty and approximation in both procedures, ideally the SPT–CPT correlations should be used as complementary information rather than following only the one or the other. The Becker Penetration Test (BPT) database is rather limited as this test is only used in soils with large particles (e.g. gravels), where SPTs and CPTs cannot provide accurate results, while the shear wave velocity (V_s) measurements are used more rarely either in absence of other data or to complement the CPT or SPT based evaluations.

Since the early work of Seed & Idriss (1971, 1982) and Seed *et al.* (1985), numerous empirical correlations and modifications of the employed empirical correction factors in the liquefaction evaluation process have been proposed in the literature. Youd *et al.* (2001), based on the findings of two workshops on evaluation of liquefaction resistance of soils (1996 NCEER and 1998 NCEER/NSF workshops), provided updated SPT, CPT and V_s correlations. Since the state-of-the-art summary by Youd *et al.* (2001), the database has been enhanced with a number of case histories from more recent earthquakes and therefore a number of studies have attempted to

update available correlations and/or have revisited some of the established corrections and procedures used in the liquefaction evaluation, e.g. Cetin *et al.* (2004), Seed *et al.* (2003), Moss *et al.* (2006) and Idriss & Boulanger (2006, 2008).

5.2.2 Deterministic vs probabilistic approaches

Liquefaction potential is related to a number of uncertain variables, including *in-situ* resistance, ground water conditions, the composition of the soil, as well as the loading factors (amplitude and duration of the ground shaking). For any site, particularly a linear site extending over long distances, there are significant degrees of uncertainty associated with all these variables, which mean that to present liquefaction potential as a simple binary case of 'liquefaction'/'no liquefaction' is an over-simplification. The majority of methodologies do adopt this type of deterministic approach to assess liquefaction potential, based on empirical evidence, numerical modelling or first principles. However, a number of researchers (e.g. Juang *et al.*, 2002; Cetin *et al.*, 2004; Moss *et al.*, 2006) have developed probabilistic approaches based on the statistical scatter of the empirical data. They provide a very useful framework for taking into consideration the uncertainty in input parameters and presenting the results in the form of a probability of liquefaction (P_L). These methods express the probability of liquefaction as a function of both loading terms (peak ground acceleration and moment magnitude M_w) and resistance terms (e.g. SPT blow counts, fines content and vertical effective stress). However, these reliability-based probabilistic methods are not fully probabilistic methods as the ground motion hazard is not considered in a probabilistic manner. Since in most cases ground motion hazard is assessed and defined probabilistically using probabilistic seismic hazard analysis (PSHA), this leads to some inconsistency and confusion.

As design procedures move towards a performance-based approach (Kramer & Mayfield, 2007), it may be appropriate to determine the probability of liquefaction at a given site subjected to ground motions at a range of hazard levels, and to understand facility performance if these levels are exceeded. There are several methods that combine probabilistic ground motions with conventional liquefaction potential procedures (Kramer & Mayfield, 2007; Juang *et al.*, 2008). In these approaches the standard liquefaction assessment follows a PSHA in which the joint probability distribution of the two key input parameters of the conventional liquefaction assessment, PGA and earthquake moment magnitude for earthquake scenarios, is determined. Fully probabilistic methods for liquefaction potential evaluation (Goda *et al.*, 2011) should take into account uncertainties in both ground motion (i.e. earthquake occurrence and ground-motion intensity)

and soil resistance (i.e. material properties, ground profile, etc.). These methods also employ outcomes of a PSHA, in the form of a seismic hazard curve and associated deaggregation results, to account for the joint probability distribution of ground motion parameters and moment magnitude of earthquake scenarios. The joint probability distribution is then integrated with reliability-based liquefaction evaluation procedures. The outcome of such a fully probabilistic analysis is a direct estimate of the return period of liquefaction, rather than a factor of safety or probability of liquefaction conditional upon ground shaking for a specific return period.

Whether probabilistic or deterministic approaches are used, there is always a need for engineers to rigorously manage the uncertainties in their calculations. Within a deterministic framework, engineering judgement can be used to assign qualitative ratings to the probability of liquefaction. For example, the simultaneous occurrence of several low-probability scenarios that would give rise to liquefaction, would have a very low probability of occurrence (Idriss & Boulanger, 2008). On major infrastructure projects, it is often considered appropriate to use probabilistic approaches in conjunction with deterministic ‘sense checks’.

5.2.3 Laboratory testing

Laboratory testing of ‘undisturbed samples’, typically simple shear, triaxial or torsional cyclic tests, can be used to obtain a direct estimate of the cyclic strength of saturated sands. However, laboratory testing is not used as widely as the conventional empirical liquefaction evaluation method due to its high cost and difficulties associated with sample disturbance during both sampling and reconsolidation. It is widely accepted (e.g. Seed *et al.*, 2003) that cyclic simple shear and cyclic torsional shear tests are the preferred laboratory methods to evaluate the liquefaction resistance, as they can reproduce more accurately the field loading conditions compared with cyclic triaxial testing.

5.2.4 Emerging practice – numerical modelling

Whilst numerical modelling of liquefaction (mainly finite difference and finite element modelling) has been developing for over 30 years, it still represents ‘emerging practice’ as its use is still limited due to time and data constraints. The particular relevance of these techniques in the context of this chapter is their use in exploring risk management options and in reducing uncertainties associated with risks that have been identified as unacceptable by infrastructure owners and operators. Within a risk management

framework, the value of undertaking costly analyses and tests can be compared with the magnitude of the risk based on simpler assessment techniques and thus justified in specific instances.

The empirical assessment of the liquefaction potential, described in the previous sections of this chapter, does not provide insight into the triggering mechanism of liquefaction or the physics of the actual phenomenon. However, numerical modelling of liquefaction does have the potential to be used not only to improve our understanding of the liquefaction process and but also to predict the liquefaction response. The distinct advantage of such analyses is that the triggering of liquefaction, the post-liquefaction stability and the resulting displacements can be estimated in a single time domain analysis.

Several advanced elasto-plastic constitutive models have been developed, such as those based on nested yield surfaces (e.g. Prévost, 1977; Yang & Elgamal, 2008), general plasticity (e.g. Pastor *et al.*, 1990; Ling & Yang, 2006) and bounding surface plasticity theories (e.g. Manzari & Dafalias, 1997; Li, 2002; Papadimitriou & Bouckovalas, 2002; Loukidis & Salgado, 2009). The use of such sophisticated numerical tools is based on first capturing the element behaviour from laboratory tests and calibrating the model parameters, before considering the entire boundary value problem. The calibration of advanced constitutive models is often demanding in terms of the required laboratory data and time and therefore the use of such models is still rather limited in engineering practice. Furthermore, the accurate prediction of the liquefaction response in the field when using advanced numerical tools is of course highly dependent on the adopted ground model, both in terms of geometry and the characteristics of the strata but also the ground motion, each of which has its own uncertainties and limitations.

5.3 Hazard quantification

5.3.1 Permanent ground deformation

Modes of permanent ground deformation resulting from earthquake-induced liquefaction, include:

- reduction in strength leading to reduced bearing capacity and hence settlement;
- volumetric strain-related settlement;
- lateral spreading;
- flow failure; and
- tension cracks and pressure ridges.

Once the potential for liquefaction hazard to occur has been assessed, a key stage of the risk assessment framework shown in Fig. 5.2 is to quantify the hazard, e.g. the amount of potential permanent ground deformation (PGD) that could occur in an event. The principal modes of PGD due to liquefaction are lateral, either flow failure or lateral spreading, or vertical, due to either bearing capacity failure or volumetric strain-induced settlements. The extent of PGD and its mode, will dictate the response of infrastructure and hence the *impact* side of the risk equation. If ground deformations are not expected to cause damage, then the risk is negligible and there is no need for further assessment.

The evaluation of liquefaction-induced ground deformations, both vertical and horizontal, is complex. The amount of movements can be estimated using empirical relationships or numerical modelling. The empirical relationships, although widely used, have a significant degree of uncertainty, which is not always rigorously considered.

Uncertainty associated with the assessment of PGD arises from the following sources:

- the level of earthquake hazard (which itself is correctly described probabilistically);
- the likelihood of liquefaction triggering as discussed in Section 5.2.1;
- the calculation methodology used (see following sub-sections);
- the *in-situ* soil parameters (e.g. penetration resistance), which are often based on limited data and variable ground conditions.

5.3.2 Lateral spreading

Lateral spreading results from a combination of static disturbing forces on a slope, additional inertial forces due to an earthquake, and reduced strength of the liquefied material. According to Seed *et al.* (2003) there are three main approaches to estimate liquefaction induced lateral deformations:

- Numerical methods (finite difference/finite element, see Section 5.2.4) employing advanced constitutive models (e.g. Valsamis *et al.*, 2010). The majority of the existing publications though focus on simulation of centrifuge test results.
- Statistically derived empirical methods based on back-analyses of earthquake case histories. Rauch & Martin (2000), Youd *et al.* (2002) and Zhang & Zhao (2005) rely on magnitude and epicentral distance for the prediction of ground surface displacements, while Shamoto *et al.* (1998), Hamada (1999), Zhang *et al.* (2004) and Aydan *et al.* (2005) employ either the peak ground acceleration or velocity (PGA/PGV) as an intensity measure for the lateral spreading prediction.

- Simple static limit equilibrium approach coupled with Newmark (1965) sliding block analysis (e.g. Olson & Johnson, 2008).

5.3.3 Flow failure

Flow failure occurs when the static shear stresses on sloping ground exceed the residual shear strength of the soil. Displacements due to flow failures can be very large (up to tens of metres). Flow failures have the potential to significantly disrupt infrastructure over a large area. The *potential* for flow failure to occur is reasonably straightforward to evaluate. Standard slope stability analysis procedures can be used, substituting the residual undrained shear strength of the liquefied layer for its static properties, where a factor of safety below unity indicates a flow failure hazard.

A number of empirically derived relationships exist for the estimation of residual shear strength (e.g. Olson & Stark, 2002; Yoshimine *et al.*, 1999; Stark & Mesri, 1992). These empirical relationships are significantly dependent on the dataset used, and the assumptions made for the back analysis of an observed flow failure, such as void redistribution, pre-liquefaction properties and geometry (Idriss & Boulanger, 2008). Residual shear strength parameters can also be obtained from laboratory tests although such tests are dependent on obtaining true undisturbed samples. As noted by Kramer (1996) 'evaluation of the residual shear strength of a liquefied sand is one of the most difficult problems in contemporary geotechnical earthquake engineering practice'. This observation still holds true and appropriate consideration of the effect of uncertainties and possible ranges of parameters is an important consideration for a risk-based approach.

5.3.4 Settlement

Settlement due to liquefaction can develop both during the seismic event and after the earthquake due to dissipation of excess pore water pressures as part of the reconsolidation of the liquefied soil. Several methods have been proposed to calculate liquefaction-induced ground deformations, including numerical and analytical methods, laboratory testing-based methods, and field testing-based methods. Most of the available methods, with the exception of numerical modelling, are limited to the prediction of the post-liquefaction vertical movements.

Tokimatsu & Seed (1987) correlated laboratory data (cyclic triaxial and simple shear tests) and SPTs, through empirical relationships between relative density and SPT blow counts, to develop chart solutions for the estimation of limiting shear and post-cyclic volumetric strains. Ishihara & Yoshimine (1992) used laboratory data to propose correlations between the reconsolidation volumetric strain and the factor of safety against

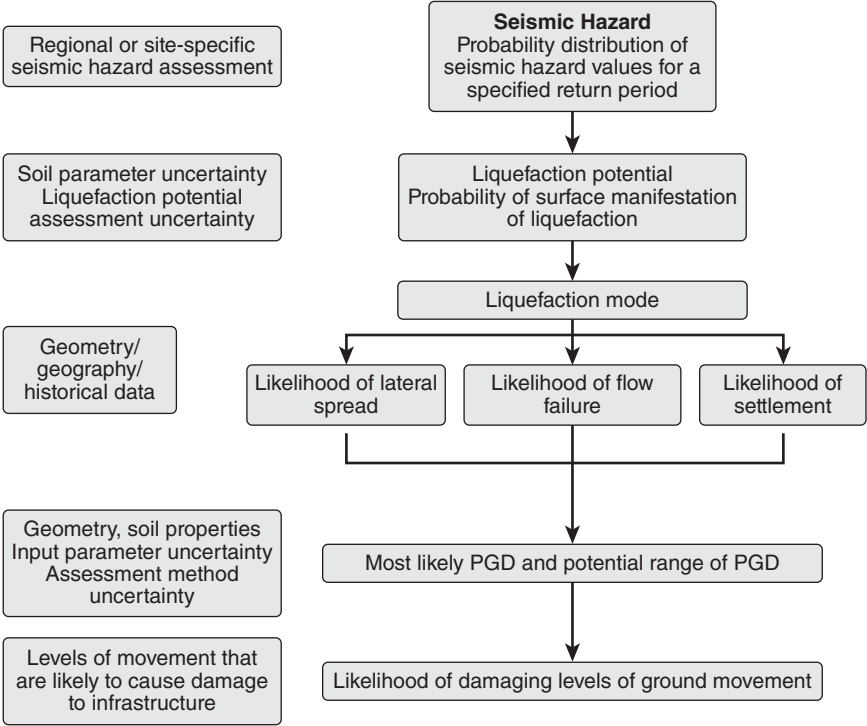
liquefaction. More recently, Shamoto *et al.* (1997, 1998) and Wu & Seed (2004) also proposed charts for estimating cyclically induced reconsolidation volumetric strains based on laboratory data, but their charts have not been used as widely as those by Tokimatsu & Seed (1987) and Ishihara & Yoshimine (1992). A comprehensive review of the available methods can be found in Cetin *et al.* (2009a,b). Furthermore, based on the work of Ishihara & Yoshimine (1992) and Zhang *et al.* (2002), Idriss & Boulanger (2008) proposed empirical charts which relate the post-liquefaction volumetric strain with the normalised CPT tip resistance. Even without significant settlement due to volumetric strains, the reduction in shear strength resulting from liquefaction can cause a bearing capacity failure, where the underlying soil no longer has sufficient residual strength to support the building loads (see Section 5.4.3).

5.3.5 Likelihood of PGD

A risk-based approach should consider the probability of different modes of ground deformation occurring, and then evaluate the probability of exceeding defined amounts of settlement or lateral movement. Based on the ideas of Kramer & Mayfield (2007), as discussed in Section 5.2.2, Juang *et al.* (2010) noted that their framework for assessing the conditional probability of liquefaction as a function of peak ground acceleration and magnitude values could be extended to evaluate the probability of exceeding a prescribed liquefaction-induced lateral spreading displacement or settlement.

Using a probabilistic approach for such calculations becomes computationally complex, where the joint probability distributions of a number of dependent and independent variables need to be considered. In practice, for a linear infrastructure system extending over a significant distance, it is unlikely that a full probabilistic analysis would be feasible. However, the elements of the framework presented by Juang *et al.* (2010) should all be considered for a quantitative assessment of the potential risk to an infrastructure system due to liquefaction-induced ground deformation. These elements are shown in the left hand boxes in Fig. 5.4. Various input data are required to undertake this type of assessment. The assessment framework shown in Fig. 5.4 requires a significant degree of engineering judgement; it is the authors' view that pure statistical approaches are not appropriate in the context of this chapter.

Qualitative estimates of probability are commonly expressed using a five-point scale similar to that shown in Table 5.1. Where engineering judgement is heavily relied upon for an assessment of risk, it is important to define the scales in order to provide consistency.



5.4 Framework for a risk-based assessment of the likelihood that liquefaction will occur *and* cause damage to an infrastructure system.

Table 5.1 Typical five-point qualitative scale of likelihood with approximate ranges of calculated probability

Likelihood category	Suggested probability range
Extremely unlikely	<0.1
Unlikely	0.1–0.3
Moderate	0.3–0.7
Likely	0.7–0.9
Extremely likely	>0.9

5.4 Response of infrastructure to liquefaction hazards

The potential response of various types of infrastructure to liquefaction hazards is illustrated in Figs 5.5 to 5.10.



(a)



(b)

5.5 Sand ejecta due to liquefaction in Christchurch, New Zealand earthquake of February 2011: (a) sand ejected to surface along a series of sand volcanoes and (b) sand ejecta and sand volcano vents in tension cracks (courtesy EEFIT, UK).

5.4.1 Linear systems

Damage to highways due to liquefaction most often comprises settlement or lateral spreading of embankments or cuttings, causing cracks and uneven pavement surfaces (see Fig. 5.6). In general, such damage is relatively quick and cheap to repair, in comparison, for example, to damage caused by major landslides.

Bridge embankments are vulnerable to lateral spreading, due to the combination of susceptible river channel soils, and a sloping or free face. A commonly observed form of minor damage to bridges resulting from liquefaction is settlement of approach embankment fill, which is straightforward to repair, and does not present a significant risk in terms of the bridge performance.

The consequences of bridge collapse are significant (Fig. 5.7). In cases where a risk assessment shows that there is potential for this to occur, even where the likelihood is low, actions should be taken to mitigate this risk.

Liquefaction-induced lateral ground movement is one of the most pervasive causes of earthquake pipeline damage (O'Rourke & Hamada, 1992) as shown in Fig. 5.8.

5.4.2 Ports and harbours

Ports and harbours are particularly vulnerable to liquefaction due to the combination of recently placed fill material, coastal and shallow marine soils, and high water tables. Widespread damage to the Port of Kobe caused direct losses of US\$11 billion in 1995. Some 90% of the port's 187 berths were destroyed due to the liquefaction of reclaimed fill material (Bird & Bommer, 2004). The impact of this damage was not only the direct costs of repair and reconstruction and closure, but also a much longer-term effect on the activities at the port, since many operations that were transferred away from the port after the earthquake never returned. Two important observations from the damage to the Port of Kobe were the general absence of damage to piled structures, and the effectiveness of ground improvement in mitigating the risk of liquefaction.

In the Haiti earthquake (12 January 2010), the main port facility at Port-au-Prince suffered extensive liquefaction-induced damage (Fig. 5.9). In this case the impact was significant and extended well beyond the direct damage incurred, since the port was inoperable after the earthquake and relief aid could not be brought in to Haiti via the port.

5.4.3 Buildings

In the context of civil infrastructure systems discussed in this chapter, buildings are in fact of secondary importance. Nonetheless, many systems rely



(a)



(b)

5.6 Lateral spreading damage to roads and utilities following the Christchurch, New Zealand earthquakes of 2010 and 2011: (a) lateral spreading toward free face of river bank resulting in damage to road and utilities corridor beneath the road and (b) damage to utility corridor at a bridge crossing caused by lateral spreading induced deformation of bridge embankment (courtesy EEFIT, UK).



5.7 Infrastructure response to liquefaction: collapse of bridge deck due to liquefaction-induced lateral spreading of embankments and reduced lateral support to bridge piers following the Luzon, Philippines, earthquake of July 1990 (courtesy EEFIT, UK).

on buildings housing administrative facilities, plant operations, personnel and storage. Observations of earthquake damage show that compared to ground shaking, landslides and tsunamis, liquefaction is less likely to cause conventional collapse of buildings or fatalities (Bird & Bommer, 2004). Liquefaction-induced damage to buildings typically includes foundation settlement (Fig. 5.10) or tilting/displacement due to lateral spreading, although structures with poor foundations can collapse due to liquefaction-induced foundation movements.

If the residual shear strength of the liquefied soil beneath a foundation is sufficiently low (see Section 5.3.4), then bearing capacity failure will occur causing a structure to uniformly settle or tilt, or to ‘punch’ through to stiffer layers below. These types of failure are generally accompanied by heaving of the ground around the foundation. The potential for a post-earthquake bearing capacity failure can be estimated from simple static bearing capacity formulae, by substituting the residual undrained shear strength of the liquefied layers. Analysis of bearing capacity failure potential during an earthquake needs to consider the effects of eccentricity of loading due to the earthquake forces. Structures that are safe against bearing capacity failure may still experience significant settlements, depending on the strains (shear and volumetric) that develop in the underlying soils.



(a)



(b)

5.8 Floatation of underground structures: (a) damage to footpath paving in Izmit Turkey due to floatation of utility pipes following liquefaction during Kocaeli earthquake of August 1999 and (b) damage to paved area and utilities in vicinity of petrol station due to floatation of below ground tanks following liquefaction during Christchurch, New Zealand, earthquake of February 2011 (courtesy EEFIT, UK).



5.9 Damage to port structures and warehouses due to liquefaction-induced lateral spreading following the Haiti earthquake of January 2010 (courtesy EEFIT, UK).

Buildings respond very differently to uniform settlement, differential settlement and differential lateral movement; the latter having the most damaging potential. Buildings on piled foundations have significantly higher resistance than those on shallow foundations, and buildings on stiff shallow raft foundations are less vulnerable than those on individual pad footings. Structural system and construction quality are also contributory factors.

5.4.4 Methods of assessment

For a typical site-specific design, a preferred approach for zones of potentially liquefiable material is often to mitigate the liquefaction through design (e.g. ground improvement or avoidance, or for structures, use of piled foundations). However, for a risk-based approach, it is important to evaluate the impact of the liquefaction on the infrastructure system, in order to make a risk-informed decision as to whether the liquefaction should be mitigated, or whether the risk is acceptable with no additional measures. For a linear infrastructure system particularly, avoidance of liquefaction is difficult, and mitigation will be costly.



5.10 Damage to foundations in Christchurch, New Zealand, following the earthquakes of 2010 and 2011 showing liquefaction-induced settlement of building foundations (courtesy EEFIT, UK).

The assessment of possible infrastructure response to liquefaction-induced ground movement can be done by:

- ***Use of case histories.*** There are numerous case histories of the effect of liquefaction on infrastructure (e.g. Bird & Bommer, 2004). Typically, these tend to be biased towards cases where liquefaction caused damage rather than where it did not. Case histories of earthquake-induced damage are extremely dependent on local/regional conditions, and are not easily extrapolated to other regions.
- ***Engineering judgement.*** For a high-level evaluation of the potential for permanent ground deformation to damage infrastructure, use of engineering judgement is essential. In many cases, a combination of judgement and experience will enable designers to evaluate the potential for ground deformation to cause damage, and what level of damage could be expected. This will allow the risk associated with liquefaction to be placed in a framework for comparison with other risks to the system, and therefore inform a decision about mitigation measures.

- **Analytical methods.** For a detailed analysis, an analytical approach which considers both the potential ground deformation resulting from liquefaction, and the response of a specific structure or lifeline to the demand imposed by the ground deformation, can be used. For specific vulnerable items, an analytical approach is appropriate. However, as always, the less information is available, the more uncertain the analytical results will be, and a balance is required between the requirements of the analysis in terms of cost, time and resources, and the reliability of the output.
- **Fragility curves.** Fragility curves, mainly used to assess the potential for damage to existing infrastructure (for example, in earthquake loss estimation for insurance or disaster management purposes), define the probability of a particular type of structure being in a given damage state as a result of a defined level of *demand* (a function of earthquake hazard). They can be derived from analysis, expert opinion, or case history data, or often a combination of all three. Many published fragility curves exist for the response of different building types to ground shaking. However, there are very few curves presently available for the response of buildings, bridges, roads or pipelines to liquefaction-induced permanent ground deformation. The HAZUS tool developed by the Federal Emergency Management Agency (FEMA, 2003) to estimate losses resulting from a number of natural hazards including earthquakes, presents fragility curves for permanent ground deformation. However, the simplifications involved in terms of the assessment of the expected PGD and of the infrastructure response mean that these types of curves are only appropriate for large portfolio analyses, not for individual structures or lifelines.

5.5 Tolerable risks and performance levels

For a risk assessment, it is important to clarify the *risk metric*. There are many units and scales that can be used to define risks, including:

- costs (direct or indirect);
- time (impact on programme and down-time during operation);
- safety (to workers and the general public);
- less tangible measures such as reputational damage to an owner/operator, or societal impact.

Damage related to ground deformation has the potential to cause all of the above types of loss. For a risk-based approach as presented in this chapter, it is important to assess and communicate risks in a consistent format and time frame to the overall risk management methodology for the project.

Design criteria for any project should have defined levels of risk tolerance for different types of risk. Risks associated with liquefaction are no different from any other sources of risk, whether they relate to other natural hazards, or man-made events. All project risks are assessed relative to the tolerable cost, delay or safety risks, and where the risks are beyond the project tolerance, risk mitigation or reduction will be required. In assessing the tolerability of liquefaction risks, the overall performance requirements for the project in the event of an earthquake should be considered. Seismic design codes clearly define acceptable performance under different levels of ground shaking. Where *life safety* (i.e. a structure may experience significant damage, and may not be functional after an earthquake, but it will not collapse and will not present a risk to life) is the performance requirement for ground shaking with a 10% probability of exceedance in 50 years, significant damage due to liquefaction should also be acceptable, provided that life safety can be demonstrated. Uniform risk due to various potential hazards should be the aim of a risk-based approach to design.

5.6 Conclusions

Liquefaction is widely understood to have the potential to cause significant damage, delay and disruption to civil infrastructure. Methods for the assessment of liquefaction potential and of the resulting PGD have been presented in this chapter. As with any natural hazard, earthquake-induced liquefaction must be treated as a *risk item* in project planning and design. If the occurrence of liquefaction was certain, then the design approach would be more straightforward. However, as has been discussed in this chapter, the occurrence and the impact of liquefaction is an extremely uncertain area of geotechnical earthquake engineering for the following reasons:

- The occurrence of the design earthquake in the first place is uncertain, and must be considered probabilistically.
- The ground shaking induced by the design earthquake, although typically defined in terms of a single acceleration or response spectrum should be defined in terms of a probability distribution.
- The potential for liquefaction to occur is uncertain in terms of hazard, *in-situ* properties of the soil and methodology itself. Each of these areas can be treated as certain but clearly the results are then a *single point estimate* where there is actually significant variance around this value.
- The permanent ground deformation, either lateral or vertical, which results from liquefaction, is the damaging hazard, not merely the occurrence of liquefaction. A similar set of uncertainties exist for the assessment of PGD, in terms of input data and methodologies.

- The response of buildings and other infrastructure to liquefaction-induced PGD typically relies upon expert opinion and empirical data, both of which have their associated uncertainties.

This chapter has presented a framework (Fig. 5.2) for appraising each of these areas of uncertainty in a logical manner. However, the overarching requirement in any risk-based approach to liquefaction hazard is *explicit consideration of the uncertainties*, in order to obtain a robust estimation of the likelihood of the hazard occurring, and the possible consequence if it does occur. By understanding the uncertainties, it is possible to define the most likely scenario, but also the worst case, which is essential for a rigorous risk management approach.

Finally, the importance of aligning risks due to liquefaction with the overall project risk management strategy is reiterated. Risk-based decisions must be made within a clear framework of tolerable and unacceptable risks, whatever the underlying hazard event may be. The cost of the risk event and the benefit of undertaking mitigation measures are both fundamental part of risk-based decision making.

5.7 References

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Seismic risk analysis and management of civil infrastructure systems: an overview

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Abstract: This chapter summarizes the current state of the art on seismic risk analysis and management for civil infrastructure systems. In seismic risk analysis and seismic loss estimation, uncertainties are prevalent in hazard assessment, ground condition, ground motion prediction equations, building stock and infrastructure exposure, and vulnerability of infrastructure. The uncertainties often manifest as vagueness, missing information, etc., and the need for different formalisms of uncertainties is highlighted. Finally, a broader treatise of risk analysis and management is provided.

Key words: risk analysis, risk management, decision making.

6.1 Introduction

The primary role and purpose of risk analysis and management are to quantify uncertainty and to facilitate sound and efficient decision making. Faced with catastrophic earthquakes, seismic risk, which can be defined as potential for unwanted negative consequences, needs to be assessed accurately. It may be evaluated as the probability that a specified loss will exceed some quantifiable value during a given exposure time. The current framework for seismic risk analysis addresses multiple earthquake-induced hazards (i.e. occurrence of strong ground shaking, surface rupture, liquefaction, landslide, and tsunami), vulnerability/fragility of structural/nonstructural components and infrastructure systems (i.e. occurrence of damage given hazard), and consequences due to damage (i.e. casualty and economic loss, including both direct and indirect costs). In the last decade, remarkable progress has been made in this field (e.g. Ellingwood 2001; Wen 2001; FEMA/NIBS 2003; Crowley *et al.* 2005; Baker and Cornell 2008; Goda and Hong 2008; see also other chapters in this book). The developed frameworks serve as valuable decision support tools for earthquake risk mitigation, and facilitate the prioritization of available options from the reliability as well as economic viewpoints.

In seismic risk analysis and seismic loss estimation, uncertainties are prevalent in hazard assessment, ground condition, ground motion

prediction equations, building stock and infrastructure exposure, and vulnerability of infrastructure (e.g. Crowley *et al.* 2005; Kwona and Elnashai 2006; Goda and Hong 2008). Models and input parameters used in quantifying risk are subject to assumptions, validity/quality of underlying data, and data scarcity/variability (Walley 1996). Thus, consideration of uncertainty in any risk analysis is unavoidable. Typology and definition of uncertainty within the engineering community are broad and often conflicting. In risk analysis, uncertainty can be categorized into aleatory and epistemic uncertainty. Aleatory uncertainty (variability) is due to natural heterogeneity or stochasticity of a physical process and it cannot be reduced, while epistemic uncertainty is due to ignorance or subjectivity, which can be reduced with availability of more information. In the context of earthquake disasters, various uncertainty propagation methodologies have been discussed in the literature (e.g. Kaplan 1981; Paté-Cornell 1996; Baker and Cornell 2008; Bradley *et al.* 2009; Ching *et al.* 2009). Modeling and treatment of epistemic uncertainties are critically important.

One of the major challenges in seismic risk analysis is the treatment and characterization of uncertainty associated with earthquake risks. Our experience and knowledge are insufficient to envisage possible earthquake scenarios completely and comprehensively. In future disasters, there will always be surprises and new lessons will be learned. Recent examples for such surprises include damaging ground shaking and widespread liquefaction due to the February 22 2011 Christchurch (New Zealand) earthquake (despite the fact that the earthquake size was moderate) and devastating tsunami damage due to the March 11 2011 Tohoku (Japan) earthquake. These cases clearly show that our capability for predicting what might happen in a future earthquake disaster, the chance of its happening, and extent of its consequences needs to be improved. This highlights importance of extending the current approaches/methods for risk analysis and management by broadening the scope/perspective on epistemic uncertainties.

Potential remedies for quantifying epistemic uncertainties are to adopt a broader formalism of uncertainties in comparison with the current definition of risk/uncertainty in probabilistic seismic risk analysis (where all possible events are considered to be captured comprehensively). For such purposes, imprecise interval probability theory (Walley 1996) and evidence theory (Dempster 1967; Shafer 1976) may be adopted. These uncertainty theories relax some of the axioms of classical probability (e.g. total sum of the event probabilities equals one) and thus are able to capture vagueness and ambiguity (to which numerical probability values may not be assigned with full confidence). An introduction to several uncertainty theories will be given in Section 6.2. The aim of this introduction is to broaden our knowledge basis beyond the conventional definition of uncertainty in probabilistic seismic risk analysis. An application of the interval probability

theory and Italian flags to earthquake engineering risk management problems is presented in Chapter 9.

The role of quantitative/qualitative risk analysis is to provide objective evidence on potential risk by utilizing available scientific tools. Owing to unavoidable uncertainties and complexities of civil engineering structures under seismic risk, risk assessment and management are influenced by various factors (beyond physical characteristics of engineering materials and systems). They include risk perception (e.g. expert versus public), risk criteria (individual versus societal, or voluntary versus involuntary), political process, and risk communication. Depending on history of past events and public concern/reaction, objective risk assessment results may be disputed and on occasion overruled. Nevertheless, it is important to develop quantitative decision support tools for earthquake risk mitigation. Such tools are useful to quantify/compare seismic risks for different options and to facilitate informed decision making. Aiming at summarizing salient features of available analytical and decision making frameworks for assessing seismic risk quantitatively, several key seismic risk analysis tools are discussed in Sections 6.3 and 6.4. Specifically, Section 6.3 is focused on the acceptable risk criteria for different stakeholders (e.g. $F-N$ curves and life quality index). In Section 6.4, measures for seismic risk management, such as earthquake insurance, retrofitting, and strengthening, are discussed from cost–benefit analysis perspective as well as multiple criteria decision making perspective, through illustrative examples. Subsequently, conclusions and future trends of seismic risk analysis and management are mentioned.

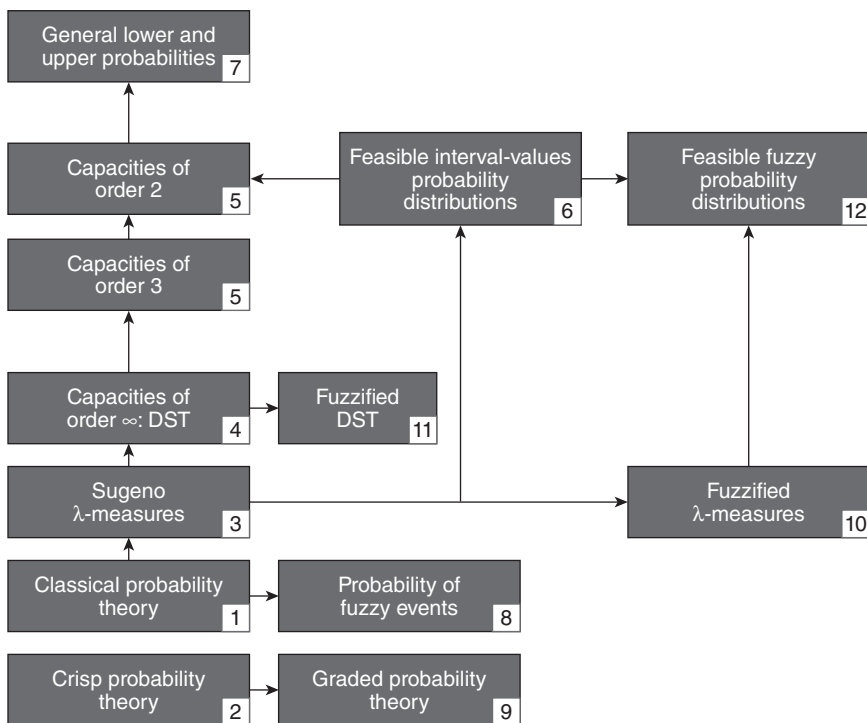
6.2 Uncertainty in risk analysis

Traditionally, aleatory and epistemic uncertainties are referred to as risk and uncertainty, respectively (Paté-Cornell 1996). Knight (1921) made a distinction between risk and uncertainty; risk is something to which mathematical probabilities can be assigned either through a priori knowledge or from the statistics of past experience, whereas uncertainty is referred to as randomness, which cannot be explained. Blockley (1995) and Klir and Yuan (1995) have broadly categorized epistemic uncertainty into *vagueness* and *ambiguity*. Ambiguity is due to unclear distinction of various alternatives, which is further divided into discord (conflict) and non-specificity. Vagueness (imprecision) refers to lack of definite or sharp distinction. The taxonomy of uncertainty, albeit to a different degree, is reflected in seismic risk analysis. Indeed, ignorance and variability require different methods of uncertainty propagation (Ferson and Ginzburg 1996). Subsequently, Klir (2004) proposed a ‘generalized information theory’ (GIT) to develop broader treatment of uncertainty quantification.

GIT is an outcome of two generalizations (Klir 2004): the first one entails the realization of information defined as a statistical measure (Shannon and Weaver 1964), and the second one entails the generalization of classical set theory by fuzzy set theory (Zadeh 1965). The GIT framework for uncertainty theories is provided in Fig. 6.1. Figure 6.1 indicates different well-developed uncertainty theories (each arrow in the figure indicates a change to a higher level of generality). As well, the numbers inside the square box indicate ordering of these theories by their levels of generality.

Essentially, the ultimate goal of GIT is to enable risk analysts to quantify prevalent uncertainty. In order to do this, the following four levels are proposed (Klir 2004):

- *Level 1:* find an appropriate mathematical representation of the conceived type of uncertainty.
- *Level 2:* develop a calculus by which this type of uncertainty can be properly manipulated.
- *Level 3:* find a meaningful way of measuring relevant uncertainty in any situation that can be formalized in the theory.



6.1 Ordering of uncertainty theories by levels of their generality (after Klir 2004).

- *Level 4:* develop methodological aspects of the theory, including procedures for making various uncertainty principles operational within the theory.

Rigorous and probabilistic applications of information theories are reported in the literature (e.g. Kaplan 1981), and the need for different uncertainty quantification techniques is debatable (e.g. North 2010; Aven 2011). To illustrate different uncertainty quantification techniques, the following theories, imprecise (interval) probability (Cui and Blockley 1990; Walley 1991), possibility theory (Zadeh 1978; Dubois and Parade 1988), and evidence theory (Dempster 1967; Shafer 1976), which may be useful for dealing with uncertainties related to seismic risk assessment, are discussed below. This is not a comprehensive coverage of the uncertainty theories; interested readers should consult with Klir (2004) and Walley (1991) for more detailed information.

6.2.1 Imprecise (interval) probability

The theory of imprecise (interval) probability was first developed by Walley (1996) as a generalization of probability. It uses an interval $[\underline{P}(A), \bar{P}(A)]$ to represent uncertainty about an event A , with lower probability $\underline{P}(A)$ and upper probability $\bar{P}(A)$ where $0 \leq \underline{P}(A) \leq \bar{P}(A) \leq 1$. The imprecision in the representation of the event A is defined by

$$\Delta P(A) = \bar{P}(A) - \underline{P}(A). \quad [6.1]$$

Evolution of the theory of imprecise probability is discussed by Aven (2011). Walley (1991, 1996) and Klir (2004) have provided the following justification for the theory:

- Imprecision of probabilities is needed to reflect the amount of information on which they are based. The imprecision should decrease with the amount of statistical information.
- Total ignorance can be properly modeled by vacuous probabilities, which are maximally imprecise (i.e. each covers the whole range $[0,1]$), but not by any precise probabilities.
- Imprecise probabilities are easier to assess and elicit than precise ones.
- We may be unable to assess probabilities precisely in practice, even if that is possible in principle, because we lack the time or computational ability.
- A precise probability model that is defined on some class of events determines only imprecise probabilities for events outside the class.
- When several sources of information are combined, the extent to which they are inconsistent can be expressed by the imprecision of the combined model.

6.2.2 Possibility theory

A possibility distribution represents a state of knowledge about an issue, and distinguishes between what is ‘plausible’ and what is ‘less plausible’, and between what is ‘surprising’ and what is ‘expected’ (Dubois and Parade 1988). An earlier idea of ‘possibility’ was supported by Shackle (1961) who stated ‘... a man cannot, in general, tell what *will* happen, but his conception of nature of things, the nature of the men and their institutions and affairs, and of the non-human would enable him to form a judgment as to whether any suggested thing *can* happen’. Zadeh (1978) later formulated the contemporary possibility theory as an extension of fuzzy set theory, in which a possibility distribution is defined numerically by the membership function. In that paper, uncertainty analysis using fuzzy arithmetic was performed by triangular fuzzy numbers (TFNs) of input parameters. To illustrate the concept, assume that force F is computed as $F = KU$, where K is the stiffness (kN/mm) and U is the displacement (mm). Assume further that both K and U are obtained as TFNs, as $\text{TFN}_K(337.5, 450, 562.5)$ and $\text{TFN}_U(75, 100, 125)$. Applying the fuzzy arithmetic (e.g. Tesfamariam *et al.* 2006), F is obtained to be $\text{TFN}_F(25\,312.5, 45\,000, 70\,312.5)$. The TFN_F can be interpreted as *possibility distribution*.

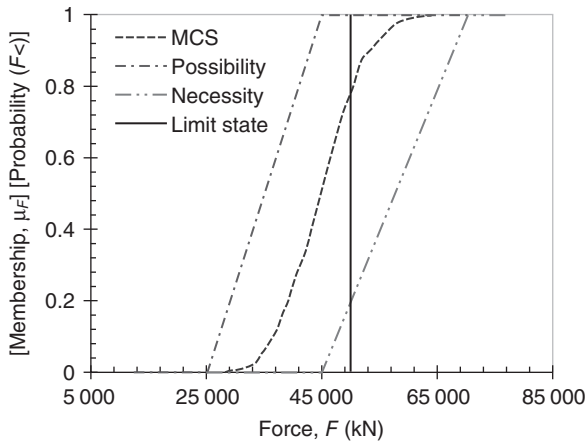
Epistemic uncertainty in possibility theory is described using ‘dual measures’ of possibility denoted by Π and necessity denoted by N . The dual measures Π and N can be interpreted as Dempster–Shafer’s *plausibility* and *belief*, respectively (this is discussed in the subsequent subsection). It has been shown that the cumulative distribution function is bounded by a possibility distribution (Joslyn and Booker 2008), i.e. ‘true probability’ measure of failure (risk) lies between necessity N and possibility Π measures. The possibility plot of the force F is shown in Fig. 6.2. To illustrate this concept further, assume that the TFNs for K and U represent triangular distribution, and Monte Carlo simulation (MCS) is performed to compute the cumulative probabilistic distribution shown in Fig. 6.2. It is interesting to note that the MCS results are bounded by the possibility function.

Assume that the critical force $F_{\text{cr}} = 50\,000$ kN. From the MCS result, $P(F < F_{\text{cr}})$ can be computed to be 0.78. The possibility and necessity measures in terms of the F_{cr} can be defined by the following relationships:

$$\Pi(F \leq F_{\text{cr}}) = \sup_u \min[\mu_F(u), \mu_{F_{\text{cr}}}(u)] \quad [6.2]$$

$$N(F \leq F_{\text{cr}}) = \inf_u \max[1 - \mu_F(u), \mu_{F_{\text{cr}}}(u)] \quad [6.3]$$

where $\Pi(F \leq F_{\text{cr}})$ and $N(F \leq F_{\text{cr}})$ are read as ‘the possibility that $F \leq F_{\text{cr}}$ ’ and ‘the necessity that $F \leq F_{\text{cr}}$ ’, respectively; $\mu_F(u)$ = membership function of F to any value u ; $\mu_{F_{\text{cr}}}(u)$ = membership function of F_{cr} for any value u ; $\sup_u$



6.2 Bounds for the probability measure based on the possibility function.

= supremum, the largest value over the universe of discourse of u ; min = minimization operator; $\inf_u$ = infimum, the smallest value over the universe of discourse of u ; and max = the maximization operator. Thus, from Fig. 6.2 and Equations 6.2 and 6.3, the possibility Π and necessity N can be computed to be 1 and 0.222. Indeed, the MCS result is bounded between the two results, i.e. $N < \text{MCS} < \Pi$.

6.2.3 Evidence theory (Dempster–Shafer, DS, theory)

Evidential reasoning or Dempster–Shafer (DS) theory (Dempster 1967; Shafer 1976) provides a generalization for Bayesian theory. The DS theory is suited to handle non-specific, conflicting, and ambiguous information. The DS theory combines the rigor of probability theory with the flexibility of rule-based systems.

Let $x = \{x_1, x_2, \dots, x_n\}$ be a mutually exclusive and exhaustive set of payoffs (for n criteria), which is referred to as the *frame of discernment*. The power set c has a total of $2^{|x|}$ subsets in the domain, where $|x|$ is the cardinality of the *frame of discernment*. For example, if $|x| = n = 3$, the power set c contains 8 subsets X_l ($l = 1, 2, \dots, 8$), i.e.,

$$c = \{\emptyset, \{x_1\}, \{x_2\}, \{x_3\}, \{x_1, x_2\}, \{x_2, x_3\}, \{x_1, x_3\}, \{x_1, x_2, x_3\}\}.$$

Three important concepts in DS theory are *basic probability assignment*, *belief*, and *plausibility*. The *basic probability assignment* (p) expresses the proportion of all available relevant evidence that supports the claim that a particular element of the power set c belongs to the (sub)set X_l but to no

other particular subset of X_l (Klir 1995). Essentially, this property shows that, for example, $\{x_2, x_3\}$ is a subset of X_l but does not belong to $\{x_1, x_2, x_3\}$. For a given evidence, every subset X_l for which $p(X_l) \neq 0$ is called a *focal element*, where $p(X_l) \rightarrow [0, 1]$; $p(\phi) = 0$ and $\sum_{X_l \subseteq X} p(X_l) = 1$.

The lower bound, belief (*bel*), for a set X_l is defined as the sum of all *basic probability assignments* (*bpa*) of the proper subsets X_r of the set of interest X_l , i.e., $X_r \subseteq X_l$. The general relation between *bpa* and belief can be written as

$$bel(X_l) = \sum_{X_r \subseteq X_l} p(X_r) \quad [6.4a]$$

where it can be shown that

$$bel(\phi) = 0; \quad bel(x) = 1 \quad [6.4b]$$

The upper bound, *plausibility* (*pl*), is the summation of *basic probability assignment* of the sets X_r that intersect with the set of interest X_l , i.e., $X_r \cap X_l \neq \phi$, and therefore it can be written as

$$pl(X_l) = \sum_{X_r \cap X_l \neq \phi} p(X_r) \quad [6.5]$$

The belief interval is an interval representing range [*plausibility* – *belief*], where ‘true’ probability may lie. A narrow belief interval represents more precise probabilities. It can be shown that the probability is determined uniquely if $bel(X_l) = pl(X_l)$; in other words, probability theory is applicable only where all probabilities are unique and disjoint (Yager 1987). The belief interval of ‘one’ implies that no information is available for X_l and ‘zero’ implies that it has been completely confirmed by $p(X_l)$.

If the decision making process requires information from multiple sources, i.e., more than one decision maker provides the assessment of p_{ij} , where i and j are indices for alternatives and criteria, respectively) aggregation (fusion) of evidence is required to estimate a valuation function $V(A_i)$. The DS rule of combination strictly emphasizes the agreement among multiple sources and ignores all conflicting evidence through *normalization*. A strict *conjunctive logic* through an AND-type operator (product) is employed in the aggregation of evidence. The DS theory also assumes that the sources of information are independent. The DS rule of combination determines the joint $p_{DS}(X_l)$ from the aggregation of basic probability assignments $p_1(X_p)$ and $p_2(X_q)$ as follows:

$$p_{DS}(X_l) = \frac{\sum_{X_p \cap X_q = X_l} p_1(X_p) p_2(X_q)}{1 - K}; \text{ when } X_l \neq \phi, \quad p_{DS}(\phi) = 0 \quad [6.6]$$

where $K = \sum_{X_p \cap X_q = \phi} p_1(X_p)p_2(X_q)$ is the *degree of conflict* between two sources of evidence. The denominator $(1 - K)$ is a normalization factor, which standardizes the effect of conflicting evidence on aggregation. The DS rule of combination has interesting characteristics, e.g. the DS rule of combination $\oplus$ is both *commutative* (i.e. $p \oplus q = q \oplus p$) and *associative* (i.e. $s \oplus (p \oplus q) = (s \oplus p) \oplus q$), but not *idempotent* (i.e. $p \oplus p \neq p$). This property makes the DS rule of combination flexible to update the evidence, as more information is provided, but the order of the evidence is not important.

A hypothetical example for data fusion of seismic damage classification is provided below. Two experts, decision makers DM_i ($i = 1, 2$), have evaluated a bridge damage during an earthquake reconnaissance. The damage classification used is C_j ($j = 1, 2, 3, 4$), where C_1 = none, C_2 = medium, C_3 = severe, and C_4 = collapse damage states. The three experts' assessment and *bpa* are summarized below:

$$DM_1 \quad p_1(C_1) = 0.40; p_1(C_1, C_2, C_3) = 0.4$$

$$DM_2 \quad p_2(C_2) = 0.30; p_2(C_2, C_3) = 0.2$$

Both decision makers' assigned *bpa* values do not sum to unity. Thus, the remaining is regarded as ignorance, which is $p_1(C_1, C_2, C_3, C_4) = 0.20$ and $p_2(C_1, C_2, C_3, C_4) = 0.50$. By assigning the *bpa* to different elements, e.g. $p_1(C_1)$ and $p_1(C_1, C_2, C_3)$, indeed can reflect the decision maker's uncertainty in the damage classification. Using the DST rule of combination (Equation 6.6), the information from the DMs are aggregated and summarized below and in Table 6.1.

Thus, the final DST results are: $p(C_1) = 0.25$; $p(C_2) = 0.23$; $p(C_2, C_3) = 0.15$; $p(C_1, C_2, C_3) = 0.25$; and $p(C_1, C_2, C_3, C_4) = 0.13$.

Table 6.1 DS rule of combination for two decision makers

	{C1}	{C2}	{C3}	{C4}	{C1, C2}	{C1, C3}	{C1, C4}	{C2, C3}	{C2, C4}	{C3, C4}	{C1, C2, C3}	{C1, C2, C4}	{C1, C3, C4}	{C2, C3, C4}	{C1, C2, C3, C4}
DM1	0.40	0	0	0	0	0	0	0	0	0	0.40	0	0	0	0.20
DM2	0	0.30	0	0	0	0	0	0.20	0	0	0	0	0	0	0.50
DST	0.25	0.23	0	0	0	0	0	0.15	0	0	0.25	0	0	0	0.13

6.3 Risk analysis

6.3.1 Qualitative and quantitative risk assessment

Risk assessment methods can be categorized into *quantitative* or *qualitative* analysis. Qualitative risk analysis uses judgment and expert opinion to evaluate the probability and consequence values. This subjective approach may be sufficient to assess the risk of a system, depending on available resources. By contrast, quantitative analysis relies on probabilistic/statistical methods and databases that identify numerical probability and consequence values for risk assessment. This objective approach examines the system in greater detail.

The selection of a quantitative or qualitative method depends on the availability of data for evaluating hazard and the level of analysis needed to make a confident decision. Qualitative methods offer analyses without detailed information; intuitive and subjective processes may have great influence, resulting in different outcomes/conclusions depending on those who use them. Quantitative analysis generally provides a more uniform understanding among different individuals; however, it requires quality data for accurate results. A combination of both qualitative and quantitative analyses can be considered depending on the situation.

6.3.2 Risk perception

Risk perception is critical in decision making under uncertainty and is of paramount importance for successful risk management, as it determines the satisfaction that is gained from a decision. If risk perception deviates from an objective and fair assessment of true risk (although it may not exist in reality), the perceived benefit of alternative options (e.g. seismic retrofitting) may vary greatly. It is rooted in cognitive limitation of human beings and it is not easy to alleviate or eliminate. A consequence of this is that the public may fail to receive the correct message from risk analysis, and may make suboptimal choices. Politicians, city planners, and experts should be well aware of the public's risk perception, as failure to meet the public demand might result in political turmoil. Abundant evidence of such sub-optimal decisions is available for natural disaster insurance (Kunreuther 1996).

6.3.3 Acceptable risk

Determination of acceptable risk levels entails consideration of health, safety, and welfare of citizens (Paté-Cornell 1994; Hayes 1998; Hall and Wiggins 2000; May 2004a,b). Starr (1969) evaluated the risk of death from various causes and identified two general categories: (i) risk associated with

voluntary activities in which an individual evaluates and adjusts his/her exposure to risk (individual risk criteria); and (ii) risk associated with involuntary activities, where risk levels are specified by regulations from governmental agencies (societal risk criteria). The benchmark for involuntary risk level is 10^{-6} (Paté-Cornell 1994). Setting this type of an acceptable risk threshold is often exacerbated for voluntary and involuntary risk exposures (Hall and Wiggins 2000). The public is typically willing to accept a risk level 1000 times greater for *voluntary* risks than for *involuntary* risks (Starr 1969).

Individual risk

The *individual risk* is defined in terms of annual probability of being harmed or killed due to a hazardous situation (Vrijling *et al.* 1995; Diamantidis 2008). Table 6.2 summarizes the personal risk associated with different activities for developed countries. Indeed, the activities summarized in Table 6.2 can be categorized in terms of voluntary and involuntary risk. Thus, considering the voluntary compared to involuntary activities, the personally acceptable probability of failure P_{fi} can be computed as (Vrijling *et al.* 1995; Diamantidis 2008):

Table 6.2 Fatality accident rate for various activities

Cause of death	During activity ¹ (per 10 ⁸ hours)	Proportion of time (average)	Annual probability (1/year)
Rock climbing	4000	0.005	1/500
Motorcycle accidents	300	0.01	1/3000
Skiing	130	0.01	1/8000
Workers in high-rise building industry	70	0.2	1/700
Deep sea fishing	50	0.2	1/1000
Workers on offshore oil and gas rigs	20	0.2	1/2500
Disease average for 40–44 age group	17	1	1/600
Travel by air	15	0.01	1/70 000
Travel by car	15	0.05	1/13 000
Disease average for 30–40 age group	8	1	1/1200
Coal mining	8	0.2	1/6000
Travel by train	5	0.05	1/40 000
Construction industry	5	0.2	1/10 000
Agriculture (employees)	4	0.2	1/12 000
Accidents in the house	1.5	0.8	1/9000
Travel by local bus	1	0.05	1/200 000
Chemistry industry	1	0.2	1/50 000
California earthquake	0.2	1	1/50 000

¹The number of fatalities per 100 million hours spent on the activity.
Source: Diamantidis (2008).

Table 6.3 The value of the policy factor α_i as a function of voluntariness and benefit

α_i	Voluntariness	Direct benefit	Example
100	Completely voluntary	Direct benefit	Mountaineering
10	Voluntary	Direct benefit	Motorbike riding
1.0	Neutral	Direct benefit	Car driving
0.1	Involuntary	Some benefit	Working in a factory
0.01	Involuntary	No benefit	Working in an LNG (liquid natural gas) plant

Source: Diamantidis (2008).

$$P_{fi} = \frac{\alpha_i \times 10^{-4}}{P_{d|fi}} \quad [6.7]$$

where $P_{d|fi}$ is the probability of being killed in the event of an accident, and α_i is the policy factor. α_i varies with the degree of voluntariness with which an activity i is undertaken and with the benefit perceived (Table 6.3). It ranges from 100, in the case of complete freedom of choice, to 0.01 in the case of an imposed risk without any perceived direct benefit.

Societal risk

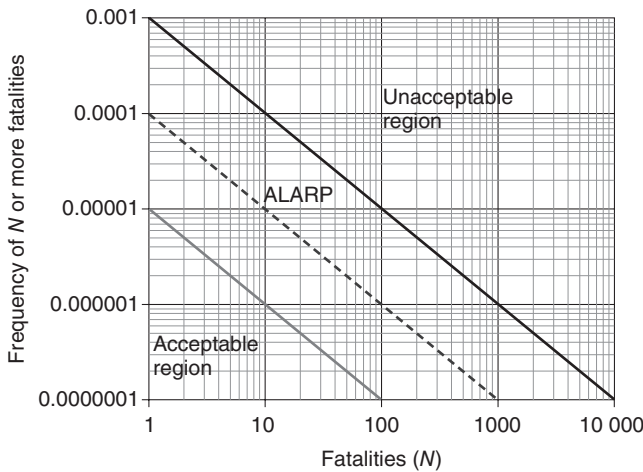
A widely used method for quantifying *societal risk* criteria is an F - N curve (Vrijling *et al.* 1995; Evans and Verlander 1997; Trbojevic 2009), which is a plot of frequency (F) against the number of deaths (N). The F - N curve displays the probability of exceedance as a function of the number of fatalities. A schematic of F - N curves is shown in Fig. 6.3. The F - N curve can be categorized into acceptable, as low as reasonably practicable (ALARP), and unacceptable regions (Melchers 2001).

The societal risk criterion, consistent with the individual risk, can be shown as (Trbojevic 2009):

$$N_{\max} \times IR = \sum f(N) \times N^{\eta_i} \quad [6.8]$$

and $f(N) = F(N) - F(N+1)$

where $N_{\max}$ is the number of people exposed to particular hazards; IR is the maximum tolerable individual risk, i.e. $N_{\max} \times IR$ is the maximum group risk assuming that all people are exposed to the same IR ; $f(N)$ is the frequency of exactly N fatalities; $F(N)$ is the frequency of N or more fatalities; N is the number of fatalities; and η_i is a risk aversion exponent ($= -1$ for no aversion and -2 for the ‘Dutch’ case see Trbojevic 2009 for further details).



6.3 Societal risk criteria.

Life quality index

The life quality index (LQI) is a compound social indicator proposed by Nathwani *et al.* (1997), and can be used to evaluate the societal acceptability of involuntary risk and to make cost-effective risk management decisions related to life-safety. The LQI criterion shares a similar foundation with the willingness-to-pay approach for increased survival (Shepard and Zeckhauser 1984), and has been successfully applied in establishing the acceptable expenditure related to life-saving interventions (Pandey and Nathwani 2004; Rackwitz *et al.* 2005).

The societal LQI, L , can be expressed as function of two measures of public welfare, social wealth G and longevity E (Pandey and Nathwani 2004; Rackwitz *et al.* 2005):

$$L = G^{w/(1-w)} E \quad [6.9]$$

where G is the consumption available in a unit of time; w is the fraction of time allocated for economic activities and typically ranges from 0.12 to 0.20 for developed countries; and E is the age-averaged discounted life expectancy of the population (Rackwitz *et al.* 2005). If the societal LQI is discussed at a national level, G is often considered as a fraction of the gross domestic product (GDP) per year and per capita. Note that the ratio w reflects the national productivity and may change over time. Furthermore, the societal LQI can be interpreted as a multi-attribute utility function calibrated for the general public, since L consists of a concave function of G with respect to $w/(1-w) < 1$ and a linear function of E . In other words, under the definition of the societal LQI, the general public is considered to

be risk averse with respect to the consumption rate, and risk neutral with respect to the discounted life expectancy.

The societal LQI facilitates the comparison of social benefits obtained from different life saving activities/interventions. For instance, public interventions that increase or maintain L are desirable (i.e. $dL \geq 0$ under small changes in G and E , dG and dE). Such criteria can be defined mathematically as follows (Nathwani *et al.* 1997; Lind 2002):

$$\frac{dL}{L} = \frac{w}{1-w} \frac{dG}{G} + \frac{dE}{E} \geq 0. \quad [6.10]$$

By considering that the life risk due to structural failure in major earthquakes is uniformly distributed over age, gender, and economic profile within a nation, Rackwitz *et al.* (2005) showed that the increase of the seismic design level X , which affects the initial construction cost $dC_0(X)$ (positive) and its associated change in mortality $d\mu_{\text{mortality}}$ (negative), is desirable if:

$$\frac{\gamma \exp(\gamma t)}{\exp(\gamma t) - 1} \frac{dC_0(X)}{dX} + N_F G_{\Delta E} \frac{\eta_c \exp(\eta_c t)}{\exp(\eta_c t) - 1} \frac{d\mu_{\text{mortality}}}{dX} \leq 0 \quad [6.11]$$

where t represents the service period of the structure; η_c is the consumption growth rate; N_F is the number of people in an engineered facility; and $G_{\Delta E}$ is numerically equivalent to the societal value of a statistical life. This inequality/criterion indicates that the investment for the intervention is less than or equal to the total gain from the saved lives (in a statistical sense); thus the equality in Equation (6.11) determines a balanced solution from the societal point of view. It is interesting to compare the socially acceptable seismic safety level for structures with the seismic design level dictated by the current seismic provisions in national design codes. Such an investigation for Canada was carried out by Goda and Hong (2006) (see also Section 6.4.3).

6.3.4 Risk communication

Risk communication covers a range of activities, aiming at increasing the public's knowledge and awareness of potential risks. It also facilitates the participation of the public in risk management and policy-making process. This includes, for example, warning labels that provide consumer education about existing hazards, development of publicly accessible databases characterizing hazardous circumstances, and public hearings on risk management issues.

Risk communication emerged as an important element of risk management of technological issues early in the 1970s. At that time, it was realized that the majority of the public was not familiar with the nature of risks and

that risk management decisions could not simply be made by technical experts and public officials. Risk communication is now viewed as being a dialogue among interested parties – risk experts, policy makers, and affected segments of the public.

This is particularly important for issues related to highly uncertain events with potentially catastrophic loss to the society (e.g. natural disasters, nuclear policy, and infectious and unfamiliar diseases). For such risks, scientific/technical assessments among the experts vary widely. Furthermore, risk perception by the public can be considerably far from estimates by the experts. The driving factors of such disparity of opinions include uncertainty, fear, dreadfulness, and irreversibility. Starr and Whipple (1980) argued that although decisions related to public policies should be based on experts' judgments, controversies may arise due to differences in (probabilistic) assessments between the public and the experts, and such reactions may overrule the experts' decisions through the political process. Therefore, significance of risk communication in the context of earthquake risk management should not be underestimated.

6.4 Risk management

The risk management is a process of weighting alternatives (options) and selecting the most appropriate action by integrating the results of risk assessment with engineering data as well as social/economic/political factors to reach an acceptable decision. Generally, risk assessment/analysis process involves objectivity, whereas risk management involves preferences and attitudes which have both objective and subjective elements. Two options that can be combined in an overall risk management strategy are (Paté-Cornell 1996):

1. reducing expected loss by implementing preventive or mitigation measures; and
2. insurance and loss sharing in case of an accident (or a damaging earthquake).

The choice of particular retrofit technique or consideration of insurance will depend on the owner's financial resources and risk tolerance. In the following, the current practice in the risk management through insurance (risk transfer) and multiple criteria decision making in the consideration of seismic retrofitting are discussed.

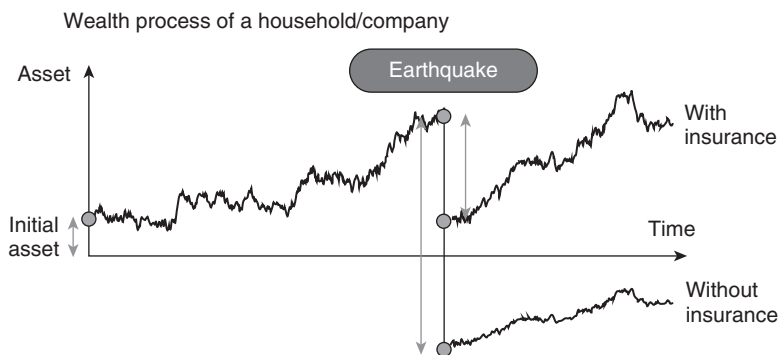
6.4.1 Risk transfer (insurance)

Insurance is a risk transfer instrument for economic/financial consequences and smoothes out fluctuation/variability of a stakeholder's asset caused by

contingencies. A stakeholder pays premium to an insurer, while he/she receives compensation, according to a pre-agreed contract, upon the occurrence of specific loss events. In the context of earthquake insurance, the occurrence of seismic damage cost, exceeding a specified deductible, triggers the pay-out from an insurer. A typical pay-out function includes deductible, cap, and co-insurance factor. The earthquake insurance premium consists of pure premium, which is equivalent to the expected damage cost, and risk premium (plus transaction cost). The risk premium is an overcharge requested by an insurer for undertaking low-probability and high-consequence events, and can be much greater than pure premium. The appreciation of benefit from purchasing earthquake insurance coverage varies significantly, depending on risk attitudes, financial status, personal experience, and many other factors (Palm 1995). Therefore, even when the overall premium is reasonably priced, not so many stakeholders voluntarily purchase earthquake risk coverage.

In earthquake risk management, besides enhancing seismic provisions for new construction and retrofitting existing structures and infrastructure (i.e. hard measures for seismic risk mitigation), insurance serves as a valuable vehicle to mitigate financial risk (i.e. soft measure for seismic risk mitigation). This is particularly important for facilitating post-disaster recovery, because a negative impact is often too grave and drastic; it would take a long time to return to the original state of wealth/asset due to stagnant economic situations in the affected area. Fig. 6.4 illustrates the wealth/asset fluctuation due to a catastrophic seismic risk and potential benefit of insurance.

In the actuarial literature, strengthening/retrofitting of structures and purchase of insurance to mitigate natural hazard risks have been discussed as an optimal strategy combining self-insurance, self-protection, and market



6.4 Fluctuation of wealth process subject to catastrophic earthquake risk.

insurance (Ehrlich and Becker 1972). They distinguished self-insurance and self-protection: the former refers to risk mitigation activities that only affect the severity of adverse events, whereas the latter refers to those that only affect their frequency.

In general, the selection of different seismic design levels affects both severity and frequency of adverse events (Goda and Hong 2006); therefore, the strengthening/retrofitting of structures is interpreted as a mixture of self-insurance and self-protection. Note that the demand and optimal purchase of market insurance are often analyzed based on the normative decision theories, such as the expected utility theory (Mossin 1968; Schlesinger 2000), and occasionally based on the descriptive decision theories, such as the regret theory (Braun and Muermann 2004). The approaches differ in how to account for risk perception and cognitive limitation of decision makers in dealing with low-probability high-consequence risks. These theoretical investigations provide valuable insights on how decision makers will behave/make choices in such situations, and are useful in developing an earthquake risk management strategy/framework.

Another important aspect of earthquake insurance is the risk transfer capacity of insurers who sell coverage to various households and companies. Conventionally, the financial risk to an insurer is coped with through risk pooling/sharing and reinsurance. The former creates a pool of funds from various insurers; the pay-out is made from the fund, when a large loss event is triggered, whereas the latter simply transfers a portion of potential insurance claim loss (typically large loss layer) to a reinsurer who often has a greater capital capability in dealing with a large loss event and does business in wider geographical areas (so risk is diversified spatially). A considerable difficulty arises, when a mega-natural disaster happens, generating spatially correlated and temporally concentrated seismic loss claims. Catastrophic earthquakes impose tremendous financial stress on insurers who underwrite earthquake insurance policies in a seismic region, resulting in possible insolvency. For example, two major earthquakes in 2010, M_w 8.8 Chile and M_w 7.0 Darfield (New Zealand) earthquakes, caused insurance claims of about 8.0 and 4.4 billion U.S. dollars, respectively (Swiss Re 2011), which rank in the top 20 costly natural catastrophes since 1970. Moreover, the two most recent devastating events in Christchurch (New Zealand) and Tohoku (Japan) caused insurance claims of 12 and 20–30 billion U.S. dollars, respectively.

To reduce the financial risk of insurers, various alternative risk transfer methods have been developed in recent years, including government/federal reinsurance and catastrophe bond/security (Kleindorfer and Kunreuther 1999; Dong and Grossi 2005). The government reinsurance relies on its capability of borrowing money over time with practically zero default risk (though current economic crises in Greece and Spain would be

a counter-example for this assumption), achieving temporal smoothing of catastrophic earthquake risk. The catastrophe bond/security takes advantage of significantly large capital size of major stock markets; investors buy securities/bonds that yield high interest if no payment-triggering earthquake occurs, while they lose the principal if such triggering seismic events happen. Although new alternative methods offer more flexibility, further advancement is yet to be achieved to deal with catastrophic earthquake risk, such as the 2011 Tohoku earthquake in Japan.

6.4.2 Prevention and mitigation

Lessons learned from performance of buildings in past earthquakes and research over the last six decades have resulted in improved seismic design code provisions. For example, the National Building Code of Canada (NBCC) (Table 6.4) has evolved over time learning from devastating earthquakes in Canada and other parts of the world (Rainer and Northwood 1979; Heidebrecht 2003). Figure 6.5 shows a plot of monetary loss and fatalities associated with earthquakes from 1980 to 2011, for different geographical/seismic regions. From Fig. 6.5, it can be highlighted that earthquakes in the United States and Japan have caused considerable monetary loss, but the reported fatalities are minimal (except for the 1995 Kobe earthquake and the 2011 Tohoku earthquake). This indeed corroborates the life safety performance limit of the current seismic design code, and the motivation for performance-based earthquake engineering that accounts for other losses. The loss of lives reported from the South American and Asian earthquake is considerably high.

For new structures, the primary means of safeguarding the public from earthquake-induced structural failures is to improve and enforce the current seismic design codes provisions. However, vulnerability of existing buildings, on the other hand, remains a concern because of the use of older design codes and/or poor construction practices/materials at the time of design and construction. The main objectives of seismic risk management for existing buildings coincide with three philosophies of new building design: (i) protect structures from collapse in a future strong earthquake, (ii) keep damage at tolerable levels in earthquakes of moderate intensity, and (iii) eliminate damage in earthquakes corresponding to relatively short return periods (Penelis and Kappos 1997). Various retrofit alternatives enhance different engineering performance measures of buildings, which can be grouped into three categories: (i) system ductility, (ii) stiffness/strength, and (iii) both ductility and stiffness/strength (see Fig. 6.6; Penelis and Kappos 1997; Thermou and Elnashai 2006). Candidate retrofit alternatives can be selected based on the parameters that contribute to building vulnerability; e.g. weak storey, soft storey, and year of construction (used as a surrogate measure of

Table 6.4 Evolution of seismic design forces in the National Building Code of Canada

Year	Base shear formula	Comments
NBC 1941	$V = CW$	NBC = National Building Code; incorporated in an appendix and was based on concepts presented in the 1937 U.S. Uniform Building Code (UBC); V = lateral force; C varied from 0.02 to 0.05; W = total weight (dead load + 25% of the snow load).
NBC 1953	$V = CW$	Incorporated into the main text of 1953 NBC; Inclusion of Canadian seismic zoning map; and recognition of the influence of building flexibility; W = total weight (dead load + 25% of the design snow load); C = horizontal force factor for minimum earthquake load; Zone 1: $C = 0.15/(N + 4.5)$; Zone 2: $C = 0.30/(N + 4.5)$; Zone 3: $C = 0.60/(N + 4.5)$; N = number of storeys.
NBC 1965	$V = RCIFSW$	R = seismic regionalizing factor (values are 0, 1, 2, and 4 for earthquake intensity zones 0, 1, 2, and 3, respectively); C = type of construction factor (=0.75 for moment resisting space frame; =1.25 for non-ductile structures); I = importance factor (1 or 1.3); F = foundation factor; S = structural flexibility factor = $0.25/(N + 9)$; W = total weight (dead load + 25% of the design snow load + design live load for storage area).
NBC 1970	$V = \frac{1}{4}R(KCIFW)$	Seismic zoning map was revised; R , I , F , and W are same as NBC 1965; K = type of construction factor (values range from 0.67 to 1.33 for buildings); C = structural flexibility factor = $0.05/T^{1/3} \leq 0.10$; T = fundamental period of the structure ($0.05h_n/D^{1/2}$ or $0.10N$); h_n = height of the structure in feet; D = dimension of the building in direction parallel to seismic force in feet; N = number of storeys.
NBC 1975	$V = ASKIFW$	I , F , and W are same as NBC 1965; A = assigned horizontal design ground acceleration; S = seismic response factor ($0.5/T^{1/3} \leq 1$); K = numerical coefficient reflecting the influence of the type of construction on the damping, ductility, and (or) energy-absorption capacity of the structures (values range from 0.7 to 2 for buildings).
NBCC 1980	$V = ASKIFW$	No major change.

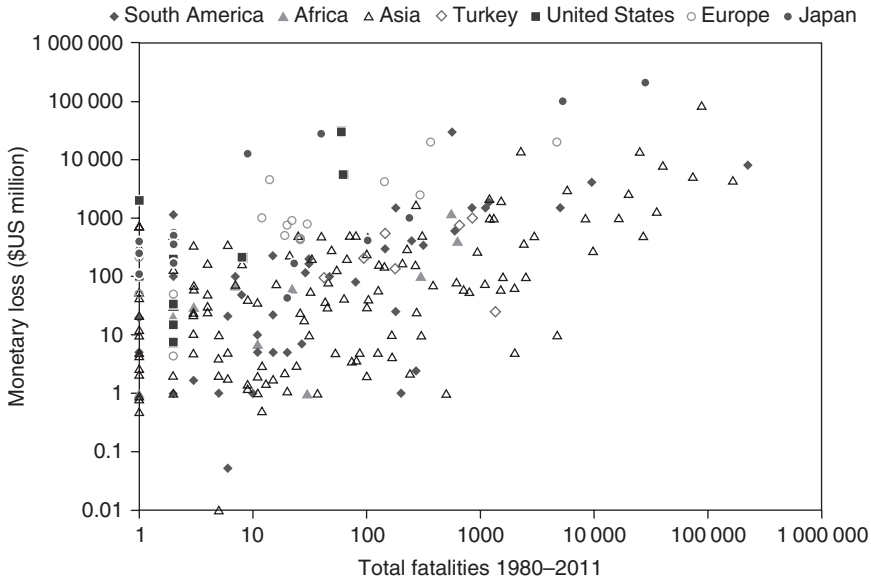
(Continued)

Table 6.4 Continued.

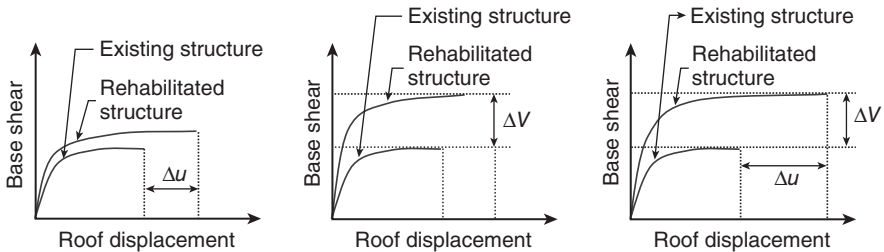
Year	Base shear formula	Comments
NBCC 1985	$V = \nu SKIFW$	New methodology in the calculation of seismic risk; a change in the probability level at which design ground motion is computed; use of both peak ground acceleration and peak ground velocity as ground motion parameter to represent the intensity of shaking; an increase in the number of seismic zones in Canada; K , I , F , and W are same as NBC 1975; ν = zonal velocity ratio; S = new seismic response factor depending on the periods of the structure.
NBCC 1990; 1995	$V = \frac{V_e}{R} U$ $V_e = \nu SIFW$	V_e = elastic lateral seismic force; $U = 0.6$, calibration factor; R = force modification factor (ranging from 1 to 4); ν = zonal velocity ratio; S = seismic response factor, I = importance factor (1, 1.3, 1.5); F = foundation on site factor; W = dead load For the 1995 code, additional R factors are introduced.
NBCC 2005	$V = \frac{S(T_a)M_v I_E W}{R_d R_o}$	$S(T_a)$ = design spectral response acceleration at the fundamental period of vibration; I_E = importance factor (1, 1.3, 1.5); R_d = ductility factor ($1 \leq R_d \leq 5$) and R_o = over strength factor ($1 < R_o < 1.7$); M_v = factor to account for higher mode effects on the base shear.

ductility). For example, if the presence of vertical and plan irregularity is identified, reinforced concrete (RC) jacketing or addition of RC walls can be used as a viable means to mitigate damage (Thermou and Elnashai 2006). On the other hand, seismic demand on the structure can be reduced by considering retrofitting using base isolation (Kelly 1997). Active and passive dampers (Housner *et al.* 1997) are increasingly used to improve the overall damping of structures and to minimize ensuing damage.

Besides consideration of the engineering performance, the best retrofitting technique can be selected by utilizing different decision making tools, e.g. cost–benefit analysis (CBA; Smyth *et al.* 2004; Goda and Hong 2006); life-cycle costing (LCC; Xu *et al.* 2007), and multi-attribute utility (Caterino *et al.* 2009; Tesfamariam *et al.* 2010). The CBA and LCC are analytic procedures to estimate the net economic value of a given alternative, which will be then compared among all viable options. The CBA and LCC require the presumption that the economic factor is the only decision driver (Cowen 1998). However, the decision making process is affected by social, cultural,



6.5 Total fatalities vs monetary loss for world earthquakes 1980–2011.



6.6 Alternative retrofit strategies for seismic upgrading.

political, economic, and engineering parameters (Penelis and Kappos 1997). After the 1985 Mexico earthquake, for example, use of bracing system and addition of structural walls induced a sense of security for building occupants (Penelis and Kappos 1997), and was a desired retrofit technique irrespective of its economic performance.

Risk management poses a challenge for decision makers to select an alternative based on multiple and often conflicting criteria that necessitate the use of decision support aid. In such cases, multiple criteria decision making (MCDM) is a useful tool. For example, parameters that influence desired retrofit selection could be grouped into general criteria and

technical criteria (Penelis and Kappos 1997). The general criteria are: consideration of cost, durability, availability of workmanship, possibility for appropriate quality control, occupancy, aesthetics, preservation of architectural identity, and duration of work. Whereas the technical criteria can be broadly categorized as: enhancement of system ductility, enhancement of stiffness and strength, and enhancement of ductility, stiffness, and strength. In Section 6.4.3, the primer of the CBA and LCC is provided and illustrated with an example. Subsequent to that, an application of MCDM for retrofit selection is illustrated with an example.

6.4.3 CBA and LCC

CBA and LCC are the essential tools for efficient risk management of structures and infrastructure that are subjected to environmental disturbances, such as deterioration and natural hazards (Rosenblueth 1976; Rackwitz 2000; Wen 2001). They can take into account all relevant costs arising during the service life of a structure, such as initial construction cost, maintenance and operation cost, and damage cost due to external hazards. It is important to note that the timing of occurrence of various costs differ; initial cost is incurred at the beginning; the maintenance and operation cost occurs continuously and regularly over the service period; whereas the earthquake damage cost, for instance, only occurs infrequently but can be very costly. For maintenance/operation and earthquake damage costs, adequate discounting to the present time is necessary. Once all relevant costs are taken into account and adequate discounting is applied, CBA and LCC produce a cost estimate of an engineered structure over its lifetime for a given design/retrofitting/intervention alternative. Comparison of such estimates for different alternatives is a rational approach to identify the most economically efficient and viable option.

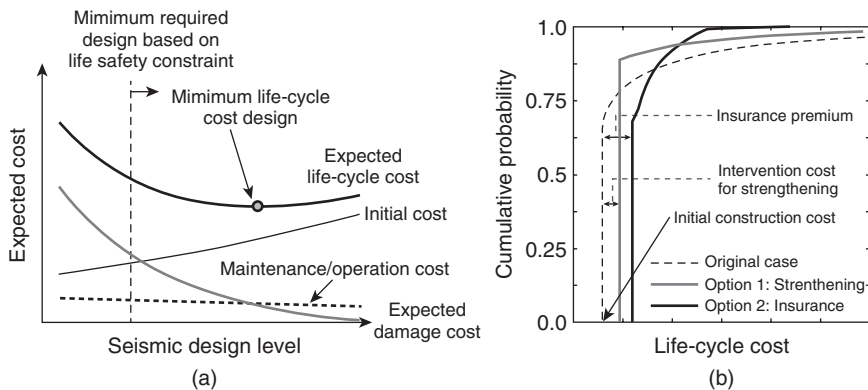
Suppose that a structure is to be designed and constructed for a service in a seismic region. Focusing on earthquake risk, the LCC of the structure consists of initial construction cost, earthquake damage cost, and maintenance/operation cost. The objective function $O(X, t)$ of the structure defined as the benefit at the present value, $B(X, t)$, minus the LCC at the present value can be expressed as (Rosenblueth 1976; Rackwitz 2000; Wen 2001):

$$O(X, t) = B(X, t) - C_0(X) - E \left[\sum_{i=1}^{N(t)} \sum_{j=1}^k C_j \exp(-\gamma t_i) P_{ij}(X, t_i) \right] - \int_0^t C_m(X) \exp(-\gamma \tau) d\tau \quad [6.12]$$

where X is the structural design parameter, such as the design base shear coefficient; $C_0(X)$ is the initial construction cost; $N(t)$ is the number of

events during the service period of t years; C_j is the damage cost for the j th limit state (i.e. damage level); t_i is the occurrence time of the i th event; γ is the discount rate from financial markets; $P_{ij}(X, t)$ is the occurrence probability of the j th limit/damage state for the i th event; and $C_m(X)$ is the maintenance and operation cost per year. The exponent term in the equation is related to discounting to the present value. The equation is formulated based on the assumption that structures are systematically repaired or reconstructed to the original condition upon partial damage or collapse. The maximization of the objective function or minimization of the LCC cost for a set of design/retrofitting options (i.e. different values of X) can identify an economically optimal solution.

To illustrate the concept, a schematic diagram is shown in Fig. 6.7a, including: expected LCC curve, initial cost curve, expected damage cost curve, and maintenance/operation curve. The horizontal axis is taken as a seismic design level, and the vertical axis corresponds to the cost (rather than benefit). In general, the initial construction cost curve is an increasing function of the design parameter, whereas the expected seismic damage cost curve is a decreasing function of the design parameter. The maintenance/operation cost curve is shown as a gently decreasing function of the design level for simplicity. The expected LCC curve has a U shape and attains a minimum value at a particular design parameter value (where the increment of initial cost curve and the decrement of the sum of the expected damage cost and maintenance/operation cost curves are the same). The identified minimum cost option can be considered as optimum (Rosenbluth 1976; Rackwitz 2000; Wen 2001) if economic efficiency is measured in terms of the expected value of (usually highly uncertain) a monetary variable. This decision criterion is a risk neutral approach. Beyond the risk neutral optimality, risk averse and risk taking attitudes can be incorporated into decision making process (Goda and Hong 2006).



6.7 Seismic design level and life-cycle cost.

Another example of CBA and LCC in the context of seismic risk mitigation is shown in Fig. 6.7b. In this figure, three cumulative probability distributions of LCCs for different mitigation options are shown. The base case is the original design as is, while two options are available for seismic risk mitigation: (i) seismic retrofitting (hard measure) and (ii) earthquake insurance (soft measure). It is noted that, looking at the probability distribution curve for the original case, the lower constant part of the distribution corresponds to a situation ‘no earthquake damage’, while the upper part of the curve, having a long right tail, corresponds to ‘earthquake damage occurrence’. By implementing seismic risk mitigation measures, the lower constant part shifts towards right; the difference of the constant part of the curves between the original case and the strengthening or insurance case is the intervention cost for strengthening or insurance premium, respectively. The risk mitigation measures usually have significant impact on the right tail of the LCC distribution. As seen in Fig. 6.7b, both hard and soft measures alter the probability distribution of the LCC, resulting in greater expected cost but less variability and consequence for the worst case. This is the essence of CBA and LCC – desirable change to risk characteristics such that the selected option meets decision makers’ risk appetite and requirements better.

6.4.4 MCDM

MCDM techniques deal with problems where alternatives are predefined and the decision maker ranks available alternatives. MCDM has proved to be a promising and growing field of research since early 1970s and many applications in the fields of engineering, business, and social sciences have been reported. Carlsson and Fullér (1996) classified MCDM methods into four distinct types, namely (i) outranking, (ii) utility theory, (iii) multiple objective programming, and (iv) group decision and negotiation theory.

Information for the MCDM parameters is often obtained from experts. Identification of experts and information solicitation is shown to be a daunting task for earthquake damage evaluation, which can be done through a Delphi procedure (ATC-13 1985). The Delphi procedure consists of formulating questionnaires, obtaining individual answers to the questioners, iterating the questionnaires one or more times where the information feedback between rounds, and aggregating the responses by statistical operations. Often solicitation of information from experts is prone to bias, misrepresentation, and anchoring of individuals’ perceptions of earthquake risk (ATC-13 1985; May 2004a; Celsi *et al.* 2005). The experts should have greater diversity in terms of experience and institutional affiliation. Other methods for expert opinion elicitation is a SSHAC approach (for probabi-

listic seismic hazard analysis (Budnitz *et al.* 1997; 1998), which is linked with nuclear seismic risk assessment) and Cooke's method (Cooke 1991).

Caterino *et al.* (2009), for example, have illustrated MCDM-based retrofit selection using decision-support software TOPSIS, ELECTRE, MAUT, PROMETHEE, etc. These MCDM techniques require generation of relative weights between two criteria. In this chapter, the example provided in Caterino *et al.* (2009) is extended into a decision making framework without consideration of weights (e.g. Laplace criterion, Maximin criterion, Hurwicz criterion, etc.).

The structure considered in the application is a three-storey RC structure built at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Center (JRC) in Ispira, Italy (Caterino *et al.* 2009), which is representative of pre-seismic code constructions in southern Europe. A total of five seismic resistance upgrading options are considered, three of those aiming at seismic capacity enhancement (A_1 , A_2 , A_3), while the last two providing seismic demand reduction (A_4 , A_5). The five retrofitting options are:

1. A_1 : confinement by glass fiber reinforced plastic of columns and joints (increases the building displacement capacity).
2. A_2 : addition of steel braces (provides a global strength and stiffness enhancement).
3. A_3 : concrete jacketing of selected columns (partial but simultaneous enhancement of strength and ductility).
4. A_4 : base isolation of the structure (reduction of the seismic forces).
5. A_5 : installing four viscous dampers at the first story of the building (attenuates the seismic demand).

Each of the five options is evaluated based on eight criteria x_{ij} , where i represents the option used and j represents the criterion (see Table 6.5). Results of the five options with respect to the eight evaluation criteria are provided in Table 6.6 (adopted from Caterino *et al.*, 2009).

Table 6.5 Description of eight evaluation criteria

Criterion	Application
C_1	Installation cost
C_2	Maintenance cost
C_3	Duration of work/disruption of use
C_4	Functional compatibility
C_5	Skilled labor requirement/needed technology level
C_6	Significance of the needed intervention at foundations
C_7	Significant damage risk
C_8	Damage limitation risk

Table 6.6 Decision matrix for the five retrofit alternatives with respect to eight evaluation criteria

Alternative	Evaluation criteria							
	C ₁ (€)	C ₂ (€)	C ₃ (days)	C ₄	C ₅	C ₆	C ₇	C ₈
A ₁	23096	23206	33	0.482	0.374	2.90	0.022	0.281
A ₂	53979	115037	122	0.063	0.104	15.18	0.024	0.002
A ₃	11175	40353	34	0.255	0.044	2.97	0.040	0.171
A ₄	74675	97884	119	0.100	0.374	2.65	0.020	0.000
A ₅	32309	36472	19	0.100	0.104	2.87	0.040	0.263

Table 6.7 Normalized payoffs x_{ij} values for the five retrofit alternatives with respect to evaluation criteria

Alternative	Evaluation criteria							
	C ₁	C ₂	C ₃	C ₄	C ₅	C ₆	C ₇	C ₈
A ₁	0.77	0.86	0.81	0.15	0.32	0.82	0.68	0.33
A ₂	0.47	0.29	0.31	0.89	0.81	0.06	0.65	1.00
A ₃	0.89	0.75	0.81	0.55	0.92	0.82	0.41	0.59
A ₄	0.26	0.40	0.33	0.82	0.32	0.84	0.71	1.00
A ₅	0.68	0.78	0.89	0.82	0.81	0.82	0.41	0.38

The evaluation criteria summarized in Table 6.6 are non-commensurable (e.g. C₁ is defined as cost in € while C₃ is defined as duration in days), and consequently cannot be directly aggregated using MCDM. Therefore, non-commensurate input parameters have to be transformed into commensurate units. Four commonly used transformation functions that can be used for this purpose are: monotonically increasing function, monotonically decreasing function, and non-monotonic function that could be convex or concave function. Selection of these functions depends on the type of input parameters (attributes). For simplicity, the transformation t_{ij} is implemented by normalizing the decision matrix x_{ij} summarized in Table 6.6 using a TOPSIS normalization method; the values are summarized in Table 6.7 (Caterino *et al.* 2008). The transformed values are further converted into a benefit criterion, where an option with the higher number is a better alternative, and summarized in Table 6.7.

To select the desired retrofit alternative, five decision criteria are considered; (i) Laplace criterion, (ii) Maximin criterion, (iii) Maximax criterion, (iv) Minimax regret, and (v) Hurwicz criterion.

Laplace criterion

Laplace criterion is based on the ‘principle of insufficient reason,’ proposed by Jacob Bernoulli. The criterion states that ‘if we are ignorant of the ways an event can occur (and therefore have no reason to believe that one way will occur preferentially compared to another),’ then they should all be treated as equally likely, being assigned the probability $1/n$, where n is the number of states. Thus, accordingly, the ranking for the five alternatives are:

$$A_3(0.72) > A_5(0.70) > A_1(0.59) > A_4(0.58) > A_2(0.56)$$

Maximin criterion

Under the Maximin criterion, the decision maker takes a pessimistic viewpoint. The decision makers assume that the worst that can happen will, and then they take the action with the best (*maximum*) worst case (*minimum*) scenario. Under this assumption, it would be best to choose that action which maximizes the minimum payoff among all possible actions.

$$A_3(0.41) > A_5(0.38) > A_4(0.26) > A_1(0.15) > A_2(0.06)$$

Maximax criterion

Under the Maximax criterion, the decision maker takes an optimistic viewpoint. The decision makers take the action with the best (*maximum*) of the best (*maximum*) case scenario. Under this assumption, it would be best to choose the action which maximizes the maximum payoff among all possible actions.

$$A_4(1.00) > A_2(1.00) > A_3(0.92) > A_5(0.89) > A_1(0.86)$$

Minimax regret criterion

Minimax decision making is based on opportunistic loss. Often, decision makers have taken a certain action only to regret it later when the true state of nature is known; e.g. the payoff was not as great as it could have been if a different action had been taken. The difference between what could have been obtained and what was actually obtained has been defined as ‘regret.’ For example, Scawthorn (2003) expounded this as:

... a plant manager chooses to delay seismic retrofit due to its high cost and judgment that there is an extremely small chance of a major earthquake in the next 5 years. An earthquake occurs soon after this decision, which causes a 30-day interruption of operations. Even though the decision was in fact a wise one, given the decision maker’s

Table 6.8 Regret matrix

Alternative	Evaluation criteria							
	C ₁	C ₂	C ₃	C ₄	C ₅	C ₆	C ₇	C ₈
A ₁	0.12	0.00	0.08	0.74	0.60	0.02	0.03	0.67
A ₂	0.42	0.57	0.58	0.00	0.11	0.77	0.06	0.00
A ₃	0.00	0.11	0.08	0.34	0.00	0.02	0.29	0.41
A ₄	0.63	0.46	0.56	0.07	0.60	0.00	0.00	0.00
A ₅	0.21	0.08	0.00	0.07	0.11	0.01	0.29	0.62

understanding of the costs and risks, the plant manager is criticized for not choosing the alternative that would have yielded the best outcome.

The regret matrix is shown in Table 6.8. For example, for C₁, the maximum value is 0.89 (in Table 6.7), which is associated with A₃. Thus, the regret matrix is computed by subtracting the other values from 0.89. In a case where a cost criterion is considered, where lower number is desirable, the regret matrix will be negative, and thus a Maximin operator should be used. Thus, for each alternative, the maximum values can be computed, and the ranks can be shown to be:

$$A_3(0.41) > A_5(0.62) > A_4(0.63) > A_1(0.74) > A_2(0.77)$$

Decision makers may well decide to take an action which maximizes the minimum regret that they may later experience.

Hurwicz criterion

Often, the decision maker is neither a complete pessimist (maximin criterion) nor optimist (maximax criterion). In this case, the Hurwicz criterion (Hurwicz 1951a,b) furnishes the flexibility to lie anywhere in between, by computing a w -index for each alternative as:

$$w\text{-index of } a_i = wM_i + (1 - w)m_i \quad [6.13]$$

where M_i and m_i are the maximum and minimum payoff or utility for action a_i over all possible states of nature, respectively. The w -index in Hurwicz operators is used to incorporate the attitude of a decision maker in the aggregation process. It characterizes the degree to which the aggregation is like *OR* or *AND* operator. Thus, from Equation (6.13), when $w = 0$, the Hurwicz operator becomes a *minimum* operator and conversely, when $w = 1$, the operator becomes a *maximum* operator. In general, the Hurwicz

Table 6.9 Transformation of linguistic degree of optimism to w -index values

Degree of optimism	w -index	Ranking of alternatives
Conservative	0.1	$A_3(0.46) > A_5(0.43) > A_4(0.33) > A_1(0.22) > A_2(0.16)$
Moderately conservative	0.3	$A_3(0.57) > A_5(0.53) > A_4(0.48) > A_1(0.36) > A_2(0.34)$
Normative	0.5	$A_3(0.67) > A_5(0.63) > A_4(0.63) > A_2(0.53) > A_1(0.50)$
Moderately optimistic	0.7	$A_4(0.78) > A_3(0.77) > A_5(0.74) > A_2(0.72) > A_1(0.64)$
Optimistic	0.9	$A_4(0.93) > A_2(0.90) > A_3(0.87) > A_5(0.84) > A_1(0.79)$

criterion operator plays an important role, if available information is not enough to assign disjoint probabilities to payoffs.

One shortcoming of this method is that it considers only the minimum and maximum values of each alternative, thus, important information may be missed. To overcome this problem, Yager (1988) introduced a new family of aggregation techniques called the ordered weighted averaging (OWA) operators that consider the ordinal position of each criterion. The degree of optimism (Equation (6.13)) in OWA operators are described as *orness*. Tesfamariam *et al.* (2010) have proposed a linguistic transformation function to quantify *orness* values. Each *orness* values are used as a surrogate measure of w -index (Table 6.9), the five options are ranked, and results are summarized in Table 6.9. It can be noted that, for w -index = 0.10, 0.30, 0.50, A_3 and A_2 are the most and least desired retrofit alternative, respectively. It is interesting to note that, with increasing w -index values, rank reversals occur; for example, for w -index = 0.70, 0.90, A_4 and A_1 are the most and least desired retrofit option, respectively.

6.5 Conclusions

Exposure of civil infrastructure systems to multiple hazards necessitates the need for a robust risk analysis tool. In this chapter, an overview of seismic risk assessment and management framework is provided, and prevalent challenges are highlighted. The risk analysis is subject to scarcity of data, lack of knowledge, and subjectivity. Various mathematical techniques and uncertainty quantification techniques available in the literature that deal with complex problem of decision making under various types of uncertainties are discussed. Moreover, a quantifiable measure of acceptable risk is discussed by focusing on different features of individual and societal risks.

Risk management is an integral part of overall seismic risk analyses that are used to promote risk transfer (insurance) and/or prevention and mitigation. Under the prevention and mitigation, ‘best’ retrofit alternative can be

selected through cost–benefit analysis and life-cycle costing. Often, the retrofit selection is subject to multiple and conflicting criteria. Different multiple criteria decision making tools, without the consideration of relative weights are discussed and an illustrative example is provided.

The state-of-the-art review from this chapter highlighted there is a need for robust risk analysis decision making tools that can incorporate different uncertainties. These uncertainties are prevalent from quantification of hazard to decision making for risk management. Failing to quantify these uncertainties, will give a false sense of security with the risk analysis, and under extreme case manifest as catastrophic consequences. The problem is compounded for spatially distributed civil infrastructure systems. Thus, there is an even increasing need to integrate existing uncertainty quantification and aggregation technique for seismic risk analysis of spatially distributed infrastructure.

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Seismic risk analysis using Bayesian belief networks

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Abstract: Seismic risk assessment involves different levels of complex models and requires consideration of uncertainties. The confidence level on seismic risk assessment can be enhanced by considering soft computing techniques that account for different sources of uncertainty. Soft computing is a conglomerate of computing techniques that include fuzzy-based methods, neuro-computing, probabilistic reasoning, genetic algorithms, chaotic systems, belief networks, and learning theory. In this chapter, application and utility of Bayesian belief network (BBN) in liquefaction assessment and seismic risk assessments at regional as well as individual building levels are provided. For the liquefaction, the BBN structure is generated from historical data through different machine learning algorithms. For the risk assessment, a heuristic-based BBN is proposed and an example is provided.

Key words: risk assessment, Bayesian belief network (BBN), liquefaction, decision making.

7.1 Introduction

7.1.1 Problem statement

Reported damage from recent global earthquakes, 1994 Northridge earthquake in the USA, 2004 Sumatra earthquake in Indonesia, and 2008 Sichuan earthquake in China, for example, highlighted the vulnerability of existing buildings and the importance of seismic retrofitting. The building vulnerability is mainly due to older building design codes, poor design/construction practice and poor code enforcement. Most of these buildings are currently operational and need to be further assessed and upgraded to minimize seismic damage and improve life safety. From a city manager's decision making perspective, comprehensive retrofitting/upgrading of all buildings is not economically feasible, and it is desirable to perform risk-based prioritization of deficient buildings (e.g. Tesfamariam and Saatcioglu 2008, 2010; Ellingwood 2001).

A risk-based seismic assessment can be undertaken by integrating site-specific seismic hazard, building vulnerability (likelihood of failure), and

importance/consequence of failure. The seismic risk assessment requires methods that combine human knowledge and experience as well as expert judgment. The challenge of seismic risk assessment and decision making is further compounded due to ubiquitous uncertainty (Wen *et al.* 2003). The level of uncertainty associated with any system is proportional to its complexity, which arises as a result of vaguely known relationships among various entities, and randomness in the mechanisms governing the system (Ross 2004).

7.1.2 Soft computing

The confidence level on seismic risk assessment can be enhanced by considering soft computing techniques that account for different sources of uncertainty (see Chapter 6). Soft computing is a conglomerate of computing techniques that include fuzzy-based methods, neuro-computing, probabilistic reasoning, genetic algorithms, chaotic systems, belief networks, and learning theory (Zadeh 1994). Different classical (probabilistic) and non-classical methods (including possibility theory, fuzzy sets, fuzzy measures, random sets) are used to represent different types of uncertainties. When extensive historical data exist, model-free methods, such as artificial neural networks (ANN), can provide insights into cause–effect relationships and uncertainties through learning from data. In cases where historical data are scarce and/or available information is ambiguous and imprecise, other soft computing techniques, such as fuzzy sets and Bayesian belief network (BBN), can provide an appropriate framework to handle such relationships and uncertainties. BBN has a utility where the physics-based models are not readily available and intuitive knowledge of the expert is used to develop the causal network (e.g. Li *et al.* 2010).

In this chapter, applications of BBN to site-specific seismic hazard (liquefaction), regional damage estimation, and individual building damage estimation are presented. There are increasing applications of the BBN in regional risk assessment (Cockburn and Tesfamariam 2012), seismic hazard assessment (Bensi *et al.* 2009; Bayraktarli *et al.* 2011), risk assessment for bridges (Bensi *et al.* 2009) and buildings (Faizian *et al.* 2004; Tesfamariam *et al.* 2010; Bayraktarli and Faber 2011), loss assessment (Li *et al.* 2010; Schubert and Faber 2011); and optimization and reliability (Nishijima *et al.* 2009; Straub and Der Kiureghian 2010a,b). Weber *et al.* (2010) provided a thorough review of the application of BBN in risk analysis.

7.2 Bayesian belief networks (BBN)

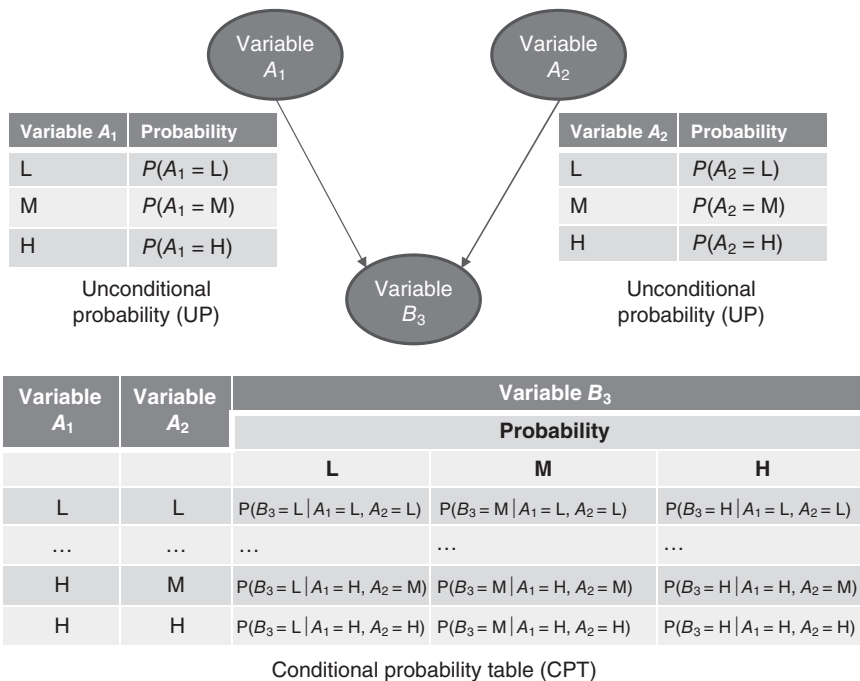
BBN, also known as Bayesian net, causal probabilistic network, Bayesian network or simply belief network, is a graphical model that permits a proba-

bilistic relationship among a set of variables (Pearl 1988). A BBN is represented with a directed acyclic graph (DAG), where nodes represent variables of interest (vertical irregularity, exposure, etc.), and the links between them indicate informational or causal dependencies among the variables (Pearl 1988; Straub and Der Kiureghian 2010a,b). The absence of a link between two variables is an indication of conditional independence between the corresponding variables.

Uncertainties in a BBN model are described through subjective probability (Pearl 1988). As depicted in Fig. 7.1, a BBN is composed of:

- a set of variables (e.g. A_1 , A_2 and B_3) and a set of directed links between the variables;
- a set of mutually exclusive states for each variable (e.g. for A_1 and A_2 the states are {L, M, H}); and,
- an assigned conditional probability for each variable with ‘parents’ (e.g. for B_3).

The relations between the variables in a BBN are expressed in terms of family relationships, where a variable A_1 is said to be the parent of B_3 and B_3 the child of A_1 if the link goes from A_1 to B_3 (Fig. 7.1). The dependence



7.1 Schematic of a Bayesian belief network.

is quantified by conditional probabilities for each node given its parents in the network, or conditional probability tables (CPTs); each variable is assigned a CPT of the variable given its parents. For variables with no parents, the probabilities are reduced to the unconditional probability (UP) (e.g. A_1 and A_2 in Fig. 7.1). The efficacy of a BBN is realized in its flexibility to capture top-down inference, observing the cause (or parent) and inferring the possible effect (or child), and bottom-up inference, observing the effect (child) and inferring the possible cause (parent).

The main concept of a BBN is rooted in the use of Bayes' theorem, in which the relation between two nodes, hypothesis H (parent) and evidence E (child), is represented as:

$$P(H|E) = \frac{p(E|H) \times p(H)}{p(E)} \quad [7.1]$$

where $p(H|E)$ is one's belief for hypothesis H upon observing evidence E , $p(E|H)$ is the likelihood that E is observed if H is true, $p(H)$ is the probability that the hypothesis holds true, and $p(E)$ is the probability that the evidence takes place. $p(H|E)$ is known as *posterior* probability and $p(H)$ is called *prior* probability. In a BBN analysis, for n number of mutually exclusive hypotheses $H_{i(i=1, \dots, n)}$, and a given evidence E , the updated probability is computed by expanding the $p(E)$ in Equation (7.1) as:

$$p(H_j|E) = \frac{p(E|H_j) \times p(H_j)}{\sum_{i=1}^n p(E|H_i) \times p(H_i)} \quad [7.2]$$

Fundamentally, a BBN is used to update probabilities as new information is obtained. The network supports the computation of the probabilities of any subset of variables given evidence about any other subset.

7.2.1 Conditional probabilities

The conditional probabilities shown in Fig. 7.1, which will be used in Equations 7.1 and 7.2, can be obtained through expert knowledge elicitation (Katsis *et al.* 2008; Joseph *et al.* 2010), or training from data (Cooper and Herskovits 1992; Bouckaert *et al.* 2011). Where multiple experts are considered, credibility of each decision maker on the decision can be elicited by considering experience and confidence on the assessment (Kiremidjian 1985; Tesfamariam *et al.* 2010). The credibility factor can be used to reach the overall decision.

The UPs of the basic input parameters are often not known *a priori*. Consequently, equal weights ($1/n$, where n is number of category considered for each basic input) can be assigned using the principle of insufficient reasoning (Tesfamariam and Martín-Pérez 2008). For example, if the states

for A_1 are categorized as *low* (L), *medium* (M), and *high* (H), the UPs will be $P(A_1 = L) = 1/3$, $P(A_1 = M) = 1/3$, and $P(A_1 = H) = 1/3$. However, from the evaluation, with certainty, if the state of A_1 is determined to be H , the UPs will be $P(A_1 = L) = 0$, $P(A_1 = M) = 0$, and $P(A_1 = H) = 1$. Indeed, if there is still uncertainty with the state of A_1 , the appropriate values should be used.

7.2.2 Elicitation of knowledge from data

There are two steps involved in the classification process: learning and inference, where various algorithms are available for structure learning (Bouckaert *et al.* 2011). Once a network structure is identified, the CPT for each variable can be estimated. The BBN B over a set of variables $U = \{x_1, \dots, x_k\}$, $k \geq 1$ is a network structure B_s , which denotes a DAG over U and a set of probability table B_p (Bouckaert *et al.* 2011). In the probability tables, $B_p = \{p[u|pa(u)] | u \in U\}$, $pa(u)$ is the set of parents of u in B_s . A Bayesian network represents a probability distribution $P(U) = \prod_{u \in U} p[u|pa(u)]$.

In a classification task, there are class variable y and attribute variables $x = x_1, \dots, x_n$. A Bayesian network classifier is to map an instance of x to a value of y , i.e. $h: x \rightarrow y$. To use a Bayesian network, all variables must be discrete. Thus, continuous variables in the dataset need to be discretized. The classification is made based on the value $\arg \max_y P(y|x)$. The value $P(y|x)$ is calculated for all class values as follows:

$$P(y|x) = \frac{P(U)}{P(x)} \propto P(U) = \prod_{u \in U} p[u|pa(u)] \quad [7.3]$$

7.2.3 Performance measures

The metrics used for assessing classification results are listed in Table 7.1. Each method is briefly discussed below. Kappa statistics is used to measure the agreement between the true and classified results (Witten and Frank 2005). As illustrated in Fig. 7.2, the kappa statistic can be calculated from the confusion matrix as:

$$\kappa = \frac{P(A) - P(E)}{1 - P(E)} \quad [7.4]$$

The maximum value of kappa is 1, which means a statistically perfect modeling whereas zero means the classification results are different from the actual values. Thus, the higher the kappa value, the better the result.

Mean absolute error is the average of the difference between predicted and actual values in the test, i.e. the average prediction error. The root

Table 7.1 Metrics for the assessment of classification results

Performance measure	Formula
Kappa statistic	$\frac{P(A) - P(E)}{1 - P(E)}$
Mean absolute error	$\frac{ p_1 - a_1 + p_2 - a_2 + \dots + p_n - a_n }{n}$
Root mean squared error	$\sqrt{\frac{(p_1 - a_1)^2 + (p_2 - a_2)^2 + \dots + (p_n - a_n)^2}{n}}$
Relative absolute error	$\frac{ p_1 - a_1 + \dots + p_n - a_n }{ a_1 - \bar{a} + \dots + a_n - \bar{a} }$ where $\bar{a} = \frac{1}{n} \sum_i a_i$
Root relative squared error	$\sqrt{\frac{(p_1 - a_1)^2 + \dots + (p_n - a_n)^2}{(a_1 - \bar{a})^2 + \dots + (a_n - \bar{a})^2}}$

Confusion matrix

classified as	a	b
a = Yes	x_1	x_2
b = No	x_3	x_4

$$\begin{aligned}
 x &= x_1 + x_2 + x_3 + x_4 \\
 p_{11} &= x_1/x, \quad p_{12} = x_2/x, \quad p_{21} = x_3/x, \quad p_{22} = x_4/x \\
 p_{\cdot 1} &= p_{11} + p_{21}, \quad p_{\cdot 2} = p_{12} + p_{22} \\
 p_{1\cdot} &= p_{11} + p_{12}, \quad p_{2\cdot} = p_{21} + p_{22}
 \end{aligned}$$

classified as	a	b	
a = Yes	p_{11}	p_{12}	$p_{1\cdot}$
b = No	p_{21}	p_{22}	$p_{2\cdot}$
	$p_{\cdot 1}$	$p_{\cdot 2}$	1

$$\begin{aligned}
 P(A) &= p_{11} + p_{21} \\
 P(E) &= p_{\cdot 1} p_{1\cdot} + p_{\cdot 2} p_{2\cdot}
 \end{aligned}$$

7.2 Calculation of kappa statistic.

mean-squared error is simply the square root of the mean-squared error. Relative absolute error is the total absolute error made relative to what the error would have been if the prediction simply had been the average of the actual values. Relative squared error is the total squared error made relative to what the error would have been if the prediction had been the average of the absolute value. The square root of the relative squared error is taken to give the same dimensions as the predicted values themselves (Witten *et al.* 2011). For these error measurements, the smaller the value, the better the performance of classification.

In the two-class case, a single prediction has four possible outcomes as shown in Table 7.2. The true positive (TP) and true negative (TN) are correct classification (Witten *et al.* 2011). A false positive (FP) occurs when the output is incorrectly predicted as negative while a false negative (FN)

Table 7.2 Different outcomes of two-class prediction

		Predicted class	
		Yes	No
Actual class	Yes	True positive (TP)	False negative (FN)
	No	False positive (FP)	True negative (TN)

occurs when the outcome is incorrectly classified as negative. Precision is referred to as positive predictive value and recall is referred to as the true positive rate or sensitivity:

$$\begin{aligned}
 \text{true positive rate} &= \frac{TP}{TP + FN} \\
 \text{false positive rate} &= \frac{FP}{FP + TN} \\
 \text{precision} &= \frac{TP}{TP + FP} \\
 \text{recall} &= \frac{TP}{TP + FN}
 \end{aligned}
 \tag{7.5}$$

The F-measure is a measure that combines precision and recall, which is expressed as:

$$\text{F-measure} = 2 \cdot \frac{\text{precision} \cdot \text{recall}}{\text{precision} + \text{recall}}
 \tag{7.6}$$

which has the best value at 1 and the worst value at 0. ROC (receiver operating characteristic) curve is a graphic plot of the sensitivity, or true positive rate against false positive rate for a binary classification. The area under the curve (AUC) is used to summarize the ROC curve. A larger area indicates a better model.

7.3 Application of Bayesian belief networks (BBN) to seismic risk assessment: site-specific hazard assessment

Site-specific seismic hazard (SSH) is an integral component of seismic risk assessment of engineered buildings (see Chapter 1). The SSH encompasses the effect of ground shaking, landslide, and liquefaction. The dominant reported cause of building damage was ground shaking (Bird and Bommer 2004); however, recent earthquakes in 2011 Christchurch, New Zealand (GEER 2011) and historical earthquakes (e.g. Bird *et al.* 2006; Juang *et al.*

2002; Mollamahmutoglu *et al.* 2003), for example, showed that liquefaction and related phenomena can still be responsible for a tremendous amount of building damage. The 1964 Alaska earthquake (March 27, 1964, Anchorage, Alaska, M_w 8.6) and 1964 Niigata earthquake (June 16, 1964, Niigata, Japan, M_w 7.5), for example, have highlighted the dramatic consequence of liquefaction-induced damage (Bird *et al.* 2006). Lateral spreading is the most pervasive type of liquefaction-induced ground failure, which is an indicator of damage severity that ranges from a few centimeters to several meters (Abdoun and Dobry 2002). Lateral spreading induced damage to deep foundation (e.g. piles) is costly and various studies are reported in the literature (Abdoun and Dobry 2002; Takahashi and Takemura 2005; Finn and Fujita 2002; Miura *et al.* 1991). Hence, a reliable liquefaction and lateral spreading risk assessment tool is of utmost importance for loss assessment in seismic risk analysis (Bradley *et al.* 2010). Discernment of liquefaction and lateral spreading vulnerability, however, is a complex and nonlinear procedure that is influenced by model and parameter uncertainty. With increasing availability of *in situ* data, however, soft computing techniques can be used for identification of sites prone to liquefaction and lateral spreading risk.

7.3.1 Application of soft computing for liquefaction

Various empirical and semi-empirical formulations of liquefaction quantification are reported in the literature (Youd and Idriss 2001). Hwang *et al.* (2004) developed a reliability-based method for assessing soil liquefaction potential. Liyanapathirana and Poulos (2002) have developed a numerical model for dynamic soil liquefaction analysis. Idriss and Boulanger (2006) have developed semi-empirical procedures for evaluating liquefaction potential during earthquakes. Cabalar *et al.* (2012) have developed an Adaptive Neuro-Fuzzy Inference System (ANFIS) for evaluating liquefaction. Juang *et al.* (2002) have developed probability-based methods for liquefaction potential evaluation using logistic regression and Bayesian mapping. ANN techniques have also been used to model the liquefaction potential (e.g. Baziar and Nilipour 2003; Goh 1994, 1996, 2002; Tung *et al.* 1993).

7.3.2 Application of soft computing for lateral spreading

To predict the lateral displacement, various empirical (Valsamis *et al.* 2010; Finn and Fujita 2002; Tamate and Towhata 1999) and numerical methods have been proposed. Bartlett and Youd (1992, 1995) have introduced an empirical model using a multi-linear regression model (MLR). The empirical models were revised by Youd *et al.* (2002) due to changes of the

database. Further refinement of the empirical model was presented in Zhang and Zhao (2005). The use of ANN for liquefaction-induced horizontal ground displacement was reported in Boone (1996), Youd and Idriss (2001), and Baziar and Ghorbani (2005). The ANN models demonstrated some advantages over the empirical models in term of the coefficient of determination. Further improvement was documented in Hwang *et al.* (2006), in which the prediction with a neuro-fuzzy system was described.

7.3.3 Example for BBN in liquefaction assessment

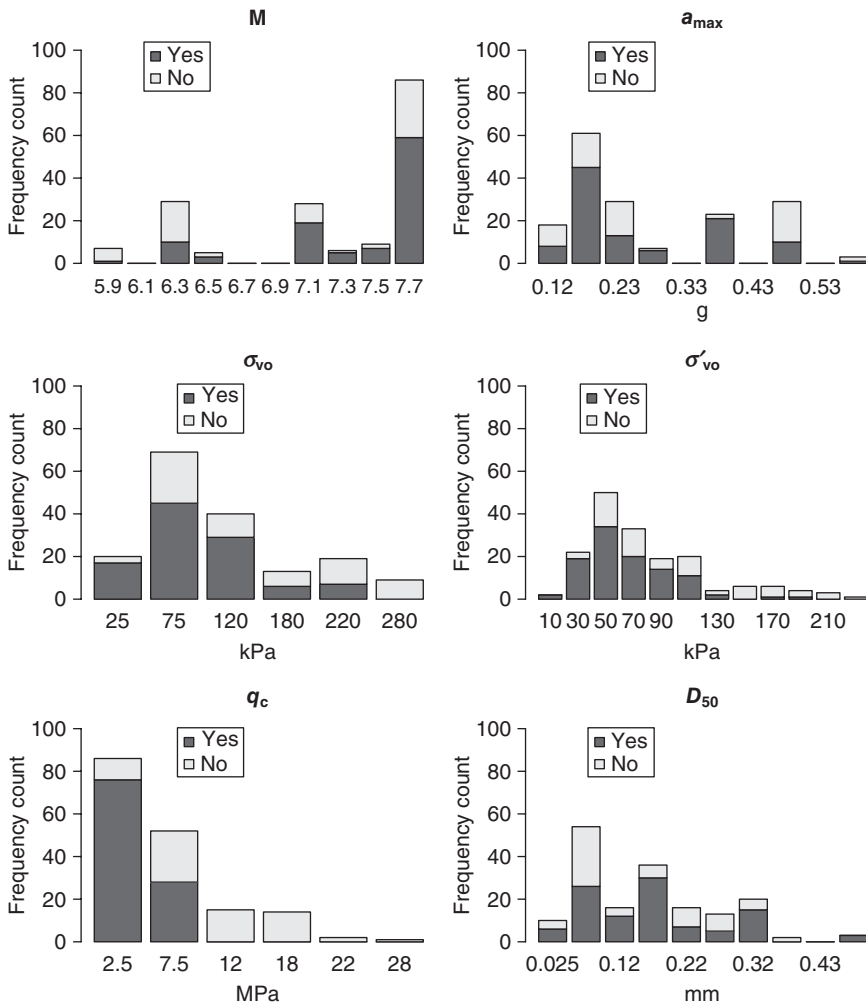
The frequently used model to assess liquefaction potential is a ‘simplified procedure’ (Finn 2002). With the simplified procedure, liquefaction resistance of a soil is expressed as a cyclic resistance ratio (CRR), whereas liquefaction ‘loading’ applied to this soil is expressed as cyclic stress ratio (CSR). The CSR is estimated by considering peak horizontal acceleration $a_{\max}$, total and effective vertical over burden pressure, σ_{vo} and σ'_{vo} , respectively, and stress reduction coefficient r_d (Seed and Idriss 1971):

$$CSR = 0.65 \frac{a_{\max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d \quad [7.7]$$

Because of the difficulty and high cost of obtaining quality undisturbed samples, the CRR is often quantified through simplified *in situ* tests methods, such as standard penetration test (SPT), cone penetration test (CPT), shear wave velocity (V_s), and Becker penetration test (BPT) (Kramer 1996). Advantages and limitations of each of these techniques are discussed in Youd and Idriss (2001), and in general, SPT and CPT are the preferred means of evaluating liquefaction potential (Finn 2002).

To show implementation of BBN, *in situ* CRR quantified using CPT and liquefaction datasets from previous earthquakes are considered (Stark and Olson 1995). The liquefaction is classified as ‘Yes’ and ‘No’, and six input parameters are considered: $a_{\max}$ = peak horizontal acceleration (g), M = moment magnitude, q_c = CPT tip resistance measurement (MPa), D_{50} mean grain size (mm), σ_{vo} (kPa), and σ'_{vo} (kPa). There are total 170 case histories in the dataset, in which 104 are ‘Yes’ and 66 are ‘No’. Figure 7.3 shows the histograms of the six variables. From Fig. 7.3, it is intuitive to see that, with increase in M , $a_{\max}$, and D_{50} , the likelihood of liquefaction increases. On the other hand, with decrease in q_c , σ_{vo} , and σ'_{vo} , the likelihood of liquefaction increases.

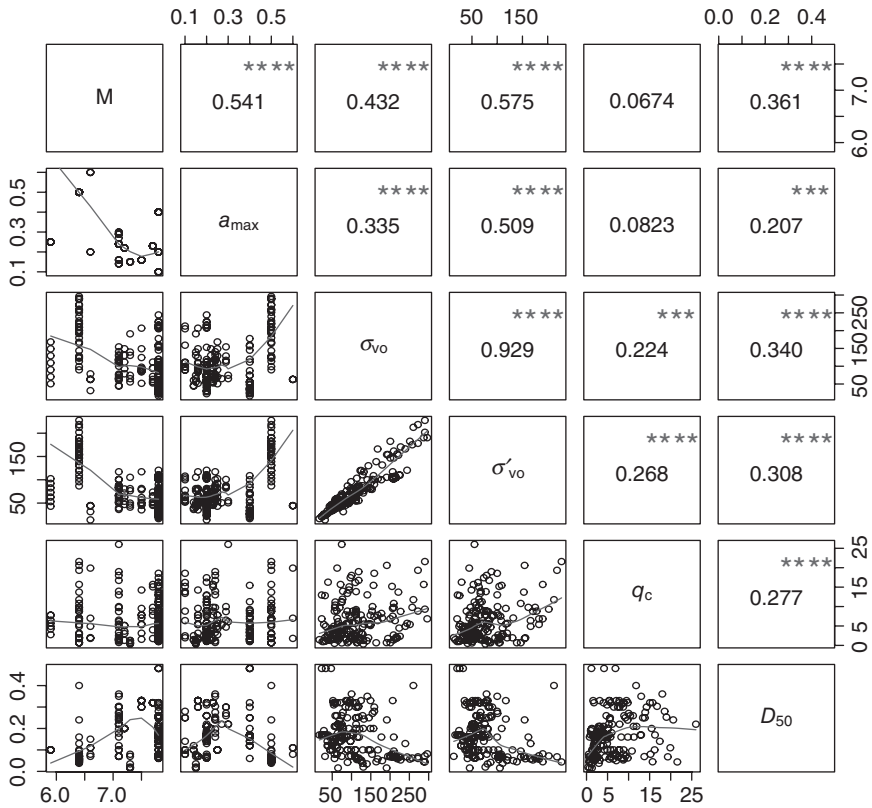
The correlations between the six input parameters are plotted in Fig. 7.4, which shows that, as expected, there is a strong correlation between σ_{vo} and σ'_{vo} , and between M and $a_{\max}$. Indeed, Fig. 7.4 also shows that there are counter intuitive results, where a strong correlation between both σ_{vo} , and σ'_{vo} and both M and $a_{\max}$. This highlights, the importance of understanding



7.3 Histogram of six variables.

the engineering significance of each parameter and correlation in the model development part.

To generate a BBN for liquefaction assessment, the local score metrics are considered for structure learning. The process of learning a network structure is implemented by maximizing the quality measure of the network given the training dataset. The quality measure, i.e. metric, can be based on a Bayesian approach, minimum description length, information and other criteria (Bouckaert *et al.* 2011). The score of the whole network can be decomposed as the sum (or product) of the score of the individual nodes. This will allow for local scoring and local search methods. Four searching



7.4 The correlation matrix of considered variables (liquefaction).

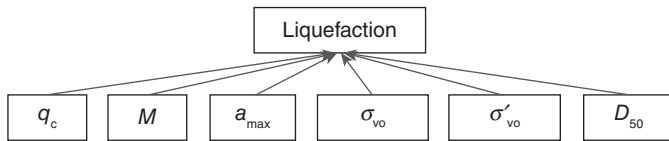
algorithms, K2, hill climbing, tree-augmented naïve Bayes (TAN), and Tabu search (Bouckaert *et al.* 2011), were tested with the available dataset. A brief description of these searching algorithms is provided in Table 7.3. Detailed information is available in the corresponding references.

The BBN relationships are generated through the four search algorithms and results are depicted in Fig. 7.5. The K2 had all input parameters at the same level, whereas the hill climbing search algorithm identified the relationship between M and $a_{\max}$. The TAN and Tabu search algorithms showed counter-intuitive results in terms of the parameter dependence. For example, both show that M is dependent on D_{50} .

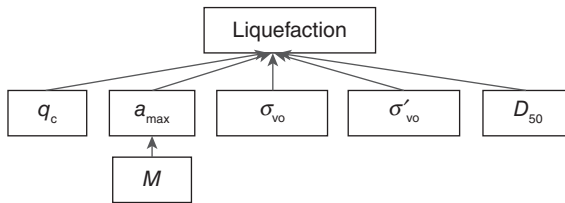
Based on the engineering judgment and known relationship between different input parameters, a fixed structure model is proposed as shown in Fig. 7.6. The conditional probability tables are generated by the specific searching algorithm with the available training data set. The classification results and performance measures of each algorithm are also summarized in Tables 7.4 and 7.5, respectively. As shown in Table 7.4, the BBN model

Table 7.3 Search algorithms

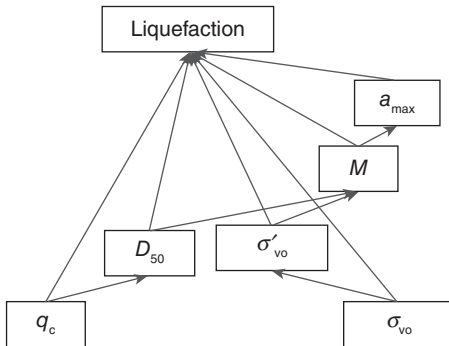
Search algorithm	Description	References
K2	Hill climbing adds arcs with a fixed ordering of variables.	Cooper and Herskovits (1992)
Hill climbing	Hill climbing adds and deletes arcs with no fixed ordering of variables.	Buntine (1996)
TAN	Known as tree augmented naïve Bayes. The tree is formed by calculating the maximum weight spanning tree using Chow and Liu algorithm.	Cheng and Greiner (1999); Friedman et al. (1997); Chow and Liu (1968)
Tabu search	Performs hill climbing until reaching a local optimum. Then steps to the least worst candidate in the neighborhood.	Bouckaert (1995)



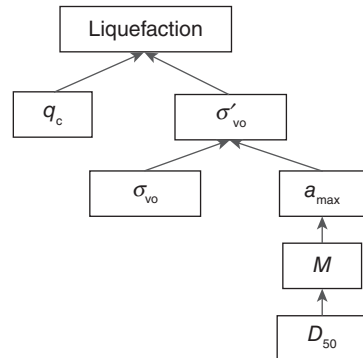
(a) K2



(b) Hill climbing

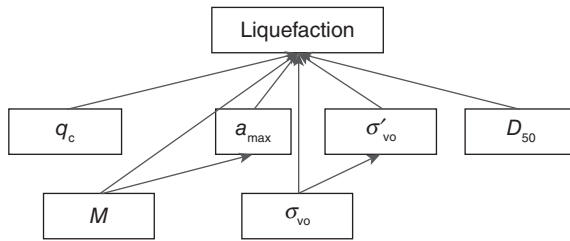


(c) TAN

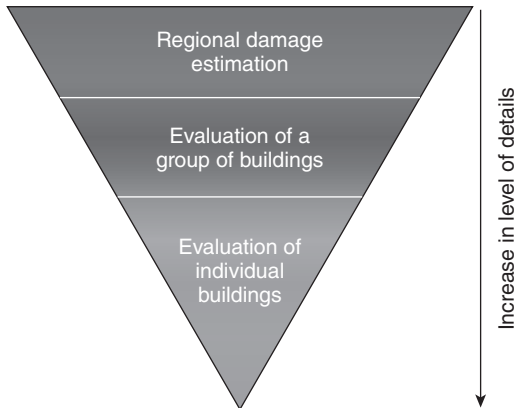


(d) Tabu search

7.5 BBN results through different classification techniques.



7.6 Bayesian network for liquefaction assessment.



7.7 Increase in level of details for regional and individual building damage estimation.

with fixed structure achieved the best classification results in terms of the criteria considered. The compatible algorithm is hill climbing, which achieves similar correct and incorrect classification rates as the model with fixed structure. However, performance of the other criteria is lower.

Table 7.5 gives information on how the BBN models work for the liquefaction status, i.e., 'yes' and 'no'. This table can be used to explore the trade-offs among BBN models over different scenarios. For example, the model with hill climbing algorithm has a larger TP rate than the model with fixed structure for liquefaction 'Yes', but it also has a larger FP rate at the same time.

7.4 Regional damage estimation

Damage estimation of buildings requires different spatial resolutions. The damage estimation can be undertaken for a city, group of buildings or individual buildings. With each assessment type, the level of details and information required increases as shown in Fig. 7.7.

Table 7.4 The classification results for liquefaction

Search algorithm	Correctly classified instances	Incorrectly classified instances	Kappa statistic	Mean absolute error	Root mean squared error	Relative absolute error	Root relative squared error
K2	135 (79.41%)	35 (20.59%)	0.5555	0.2299	0.3772	48.35%	77.37%
Hill climbing	141 (82.94%)	29 (17.06%)	0.6253	0.2289	0.3604	48.13%	73.92%
TAN	137 (80.59%)	33 (19.41%)	0.5785	0.2415	0.3888	50.79%	79.74%
Tabu search	137 (80.59%)	33 (19.41%)	0.5736	0.2656	0.3690	55.85%	75.68%
Fixed-structure result	141 (82.94%)	29 (17.06%)	0.6296	0.2225	0.3549	46.79%	72.80%

Bold type represents 'best' method.

Table 7.5 Performance assessment of different models for liquefaction classification

Search algorithm	Class	TP rate	FP rate	Precision	Recall	F-measure	ROC area
K2	Yes	0.875	0.333	0.805	0.875	0.839	0.884
	No	0.667	0.125	0.772	0.667	0.715	0.884
Hill climbing	Yes	0.933	0.333	0.815	0.933	0.870	0.894
	No	0.667	0.067	0.863	0.667	0.752	0.894
TAN	Yes	0.894	0.333	0.809	0.894	0.849	0.866
	No	0.667	0.106	0.800	0.667	0.727	0.866
Tabu search	Yes	0.913	0.364	0.798	0.913	0.852	0.875
	No	0.636	0.087	0.824	0.636	0.718	0.875
Fixed-structure result	Yes	0.913	0.303	0.826	0.913	0.868	0.887
	No	0.697	0.087	0.836	0.697	0.760	0.887

TP = true positive, FP = false positive, ROC = receive operating characteristic.

Chan *et al.* (1998) developed a general method of estimating earthquake loss by considering gross domestic product (GDP). In comparison with conventional approaches, this method requires less information and enables loss assessment for any region with readily available GDP and population data. Yong *et al.* (2002) utilized this method for the seismic hazard and loss estimation in Central America. Balassanian *et al.* (1999) have developed a similar method by considering building vulnerability, area of vulnerability, and number of inhabitants. Carrenño *et al.* (2007) applied a complex model, with a wide array of base inputs, such as population density and hospital beds, to evaluate the seismic risk for Bogota and Barcelona. This model used a hierarchical structure with the base inputs that encompassed both 'hard' and 'soft' aspects contributing to seismic risk. More recently, Barbat *et al.* (2009) compared such a holistic approach with a more traditional vulnerability index and capacity spectrum, and discovered the potential robustness of such a model and its ability to easily identify the most relevant aspects of the urban seismic risk index. However, these models apply largely to districts within city boundaries. Although this is invaluable information for planners, it cannot quickly highlight which cities within a region require such detailed analysis. The use of a hierarchical structure model to prioritize seismically high-risk cities is gaining popularity (e.g. Roberts *et al.* 2009). Davidson and Shah (1997) developed an earthquake disaster risk index (EDRI) to assess seismic risk between urban cities. The EDRI has been applied in many case studies (e.g. Sica *et al.* 2002; Zobin and Ventura-Ramirez 2004; Cockburn and Tesfamariam 2012).

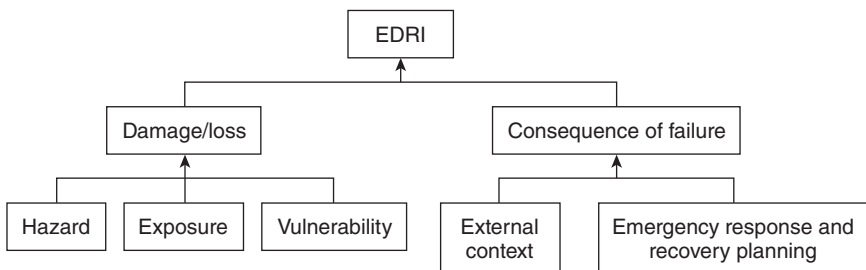
7.4.1 Example of BBN for regional seismic risk assessment

Regional damage estimation requires aggregation of different non-commensurable input parameters. Different aggregation techniques are reported in the literature, e.g. minimum, maximum, average, weighted arithmetic mean (WAM). Aggregation of multiple inputs into a single output can also be viewed that a whole is synthesis of the parts (Ross 2004). In aggregating through a hierarchical structure, there is a potential for loss of information due to exaggeration and eclipsing (Tesfamariam and Sadiq 2008). Exaggeration occurs when all input parameters are of relatively low importance, yet the aggregated score comes out unacceptably high. Eclipsing is the opposite phenomenon, where one or more of the input parameters are of relatively high importance, yet the aggregated score comes out as unacceptably low.

The EDRI quantifies earthquake risk by aggregating hazard (ground shaking and collateral hazards), exposure and vulnerability (physical infrastructure, population, economy, social-political system), external context (economy, transportation, politics, culture), and emergency response and recovery capability (planning, resources, mobility and access) (Fig. 7.8). This method gives the general risk assessment of a city or country, and facilitates decision for resource allocation and further detailed analysis. The key performance indicators used to quantify the hazard, exposure, vulnerability, external context, and emergency and recovery planning are provided in Table 7.6.

Data transformation

Input parameters of the EDRI are furnished in non-commensurable units, e.g., number of hospitals, population size, and seismic hazard. Since units of the performance modifiers are non-commensurable, as a result, transforma-



7.8 Framework for earthquake disaster risk index.

Table 7.6 Summary of input parameters and sample analysis weights

Factor	Variable
Hazard	PGA ¹ w/50-year return period ²
	PGA ¹ w/500-year return period ²
	Percent of urbanized area with soft soil ³
	Percent of urbanized area with high liquefaction susceptibility ²
	Percent of buildings that are wood ²
	Population density ⁴
Exposure	Tsunami potential indicator ⁴
	Population ⁴
	Per capita GDP, constant 1990 US\$ ⁴
	Number of housing units ⁴
	Urbanized land area ⁴
Vulnerability	Population ⁴
	Per capita GDP, constant 1990 US\$ ⁴
	Seismic code indicator ⁵
	City wealth indicator ⁴
	City age indicator ⁴
	Population density ⁴
External context	City development speed indicator ⁴
	Percent of population aged 0–4 or 65+ ⁴
	Economic context indicator ⁴
	Political country context indicator ⁴
	Political world context indicator ⁴
Emergency response and recovery	Planning indicator ⁵
	Per capita GDP, constant 1990 US\$ ⁴
	Ten-year average of annual real growth in per capita GDP ⁴
	Housing vacancy rate ⁴
	Number of hospitals per 100 000 people ⁴
	Number of physicians per 100 000 people ⁴
	Extreme weather indicator ⁶
	Population density ⁴
	City layout indicator ⁵

¹Peak ground acceleration; ²2005 National Building Code; ³Regional Geological Map; ⁴Statistics Canada (Census); ⁵Visual inspection/expert opinion; ⁶National Geophysical Data Center (NGDC).

Source: Davidson and Shah (1997).

tion functions are used to translate the actual values to a common scale to allow aggregation of the data. The transformation technique considers both average $\bar{X}_i$ and standard deviation s_i (Davidson and Shah 1997). Each input parameter is scaled by a factor of two s_i , to ensure that negative values are avoided. For the basic input parameters that are directly related to disaster risk, the transformation function is:

$$X'_{i,j} = \frac{X_{i,j} - (\bar{X}_i - 2s_i)}{s_i} \quad [7.8]$$

where X'_{ij} and X_{ij} are the scaled and unscaled values for indicator i and region j , respectively. Furthermore, for input parameters that are inversely related to disaster risk, the transformation function is:

$$X'_{i,j} = \frac{-X_{i,j} + (\bar{X}_i + 2s_i)}{s_i} \quad [7.9]$$

A salient feature of this transformation method is that the transformed values are in a relative scale and comparisons can be made quite easily.

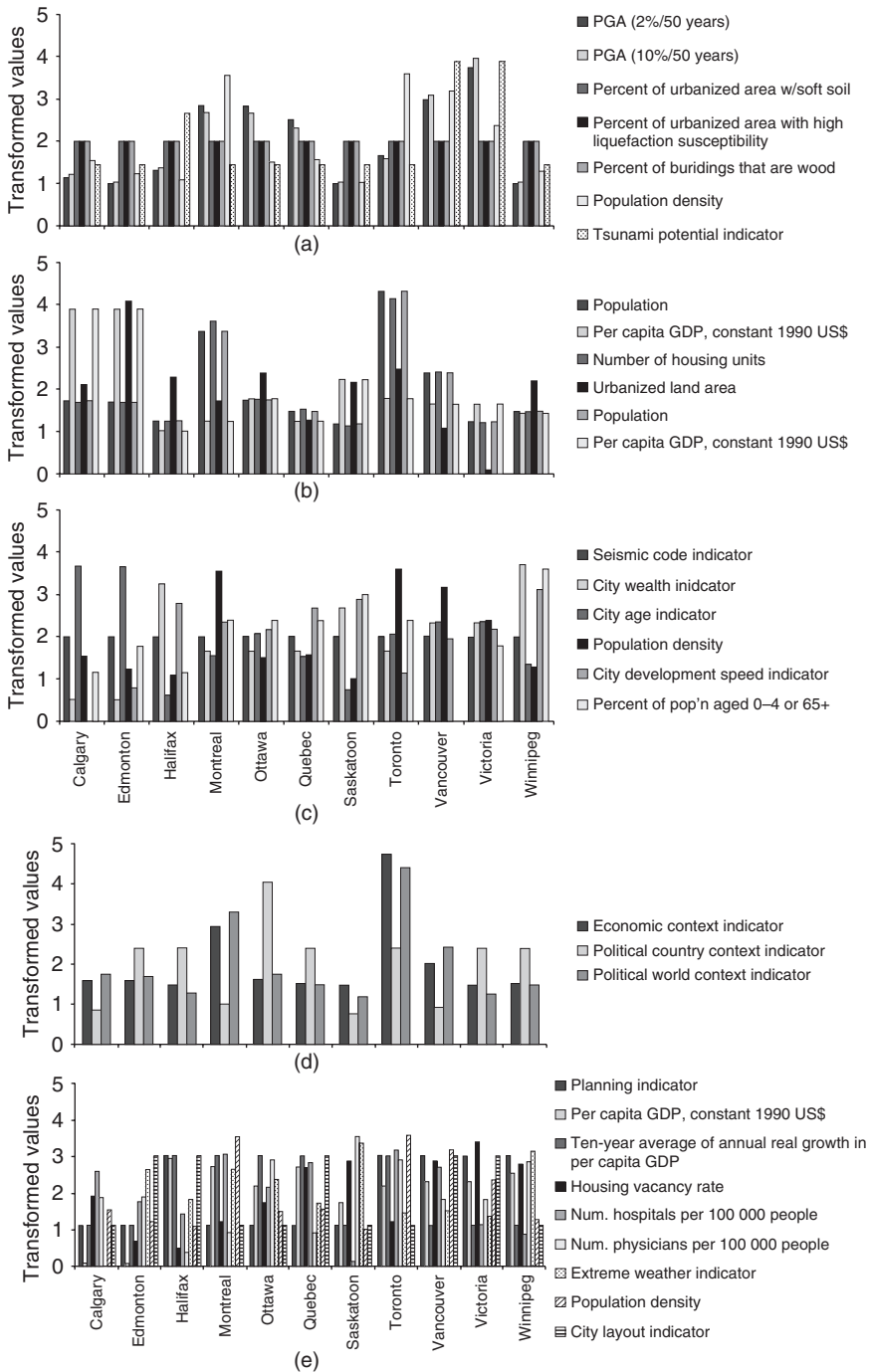
Input parameters

In order to show utility of the proposed model, 11 Canadian cities were considered (see Cockburn and Tesfamariam 2012). These cities are Victoria, Vancouver, Edmonton, Calgary, Saskatoon, Winnipeg, Toronto, Ottawa, Quebec, Montreal, and Halifax. The basic input parameters (Table 7.6) are obtained from different Canadian sources (see footnote of Table 7.6). Subsequently, the values are transformed (see Equations (7.8) and (7.9)) into commensurable units and plotted in Fig. 7.9. Subsequently, the transformed input parameters are used in the proposed BBN model, and the corresponding EDRI values are computed.

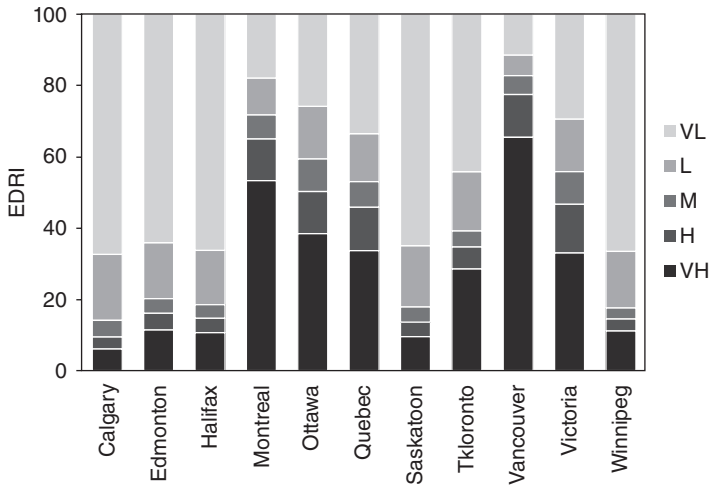
Results and discussion

Using the BBN of the hierarchical structure depicted in Fig. 7.8 and input parameters described in Table 7.6, the EDRI model is developed. The transformed input parameters shown in Fig. 7.9 are used as input parameters to the BBN model. The results of the EDRI obtained through the BBN model are depicted in Fig. 7.10, which shows that, as expected, cities located in high seismic hazard zones (e.g. Vancouver and Montreal) show higher EDRI values, as compared with cities in moderate and low seismicity (e.g. Calgary and Halifax). Furthermore, for similar magnitude of seismicity, the consequence of failure dictates the EDRI values. Vancouver has the highest EDRI value, followed by Victoria and Montreal.

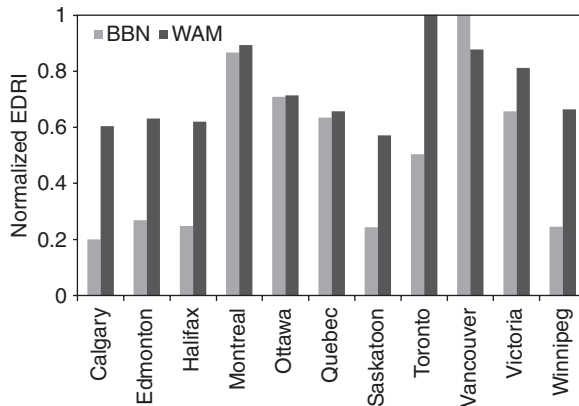
To compare the BBN results with WAM method results (Davidson and Shah 1997), states of EDRI values (EDRI^{VL} , EDRI^{L} , EDRI^{M} , EDRI^{H} , EDRI^{VH} ; superscripts stand for very low, low, medium, high and very high) are transformed into a single index. This is done by multiplying each state, EDRI^{VL} , EDRI^{L} , EDRI^{M} , EDRI^{H} , and EDRI^{VH} , respectively, by the following weights, 0, 0.25, 0.50, 0.75 and 1. Subsequently, the EDRI results of the BBN and WAM are normalized by their corresponding *maximum* values and the results are depicted in Fig. 7.11. The BBN results show that the two cities located in high seismic hazard, Vancouver and Montreal, have the highest EDRI values, whereas, the WAM showed Toronto has the highest



7.9 Transformed basic input parameters: (a) hazard, (b) exposure, (c) vulnerability, (d) external context, (e) emergency, response and recovery.



7.10 Earthquake disaster risk index (EDRI) values for 11 Canadian cities.



7.11 Comparison of WAM and BBN normalized earthquake disaster risk index (EDRI) values for 11 Canadian cities.

EDRI value, though it had a relatively low hazard. As discussed before, this anomaly is due to equal weights (importance) assigned to the five parameters of hazard and consequence of failure, linear aggregation of WAM, and nature of the hierarchical structure. On the other hand, the BBN model provides more realistic and intuitive evaluations of relative risks, and the proposed new hierarchical structure have improved these shortcomings.

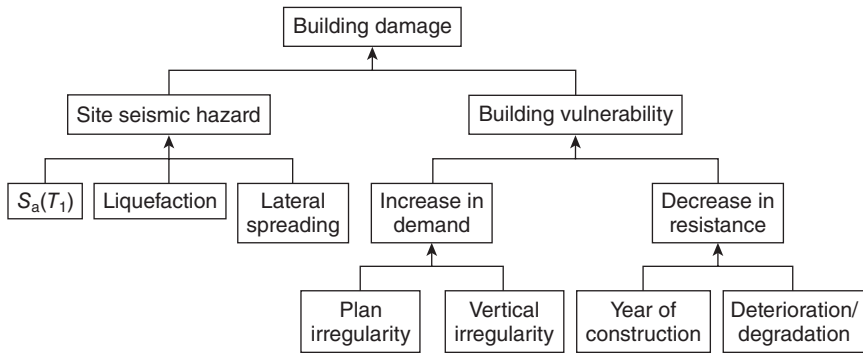
7.5 Vulnerability and damage assessment of individual buildings

7.5.1 Background

The quantification of earthquake-induced building damage requires inputs related to seismic hazard (e.g., spectra acceleration, liquefaction susceptibility) and parameters that contribute to building vulnerability (e.g., building type, vertical and plan irregularity, construction quality). Various techniques are used to assess building vulnerability and resultant seismic loss, however, albeit similar techniques, different taxonomies are reported. The reported techniques can be grouped into (FEMA249 1994; Boissonnade and Shah 1985): empirical/statistical method; heuristic method; and analytical/ mechanistic/ theoretical method. At times, where a lack of information exists, a combination of these methods can be used (Kappos *et al.* 1998).

The mechanistic methods involve building models that consider the dynamic characteristics of a structure, and are commonly used to predict damage for a single structure. The advantage is that they correlate the seismic demand with typical physical parameters used in seismic design. However, the main drawback is the challenge involved in model development and computational efforts (Villaverde 2007). Lack of available data and incomplete knowledge base, for example, necessitate the use of heuristic-based models based on the intuitive knowledge of the engineers. The empirical methods involve gathering and correlating ground motion and damage information. They are based on field observations of damaged buildings during earthquakes. This method gives an average value of the damage ratio for a group of buildings having similar structural systems and materials of construction (Boissonnade and Shah 1985). Consequently, this method is prone to high uncertainty. The empirical method is suitable for regional damage estimation; however, it cannot be used for an individual building.

The complex problem of building damage assessment can be handled through a simple and manageable hierarchical structure (Tesfamariam and Saatcioglu 2008, 2010). The hierarchical structure follows a logical order where the causal relationship for each supporting argument is further subdivided into specific contributors. Figure 7.12 shows a four-level hierarchical structure. *Level 1* of the hierarchy is the overall goal of the analysis, i.e. *building damage*. The building damage is computed by integrating the parameters at *level 2* that are *site seismic hazard* and *building vulnerability*. The site seismic hazard (*level 2*) is quantified through the fundamental period (T_1) of the structure and response spectra. The response spectra are obtained either through a site-specific design response spectrum or existing representative earthquake record. Soil type is used to modify the



7.12 Hierarchical building damage assessment.

corresponding design response spectrum. Finally, using T_1 and corresponding response spectra, spectral acceleration $S_a(T_1)$ is obtained. As part of the site seismic hazard, Fig. 7.12 also shows liquefaction and lateral spreading and can be incorporated as shown in the previous section.

Building vulnerability to ground shaking and associated damage can be grouped into two categories; factors contributing to an increase in seismic demand (e.g. plan irregularity, vertical irregularities); and factors contributing to reduction in ductility and energy absorption capacity (e.g. construction quality, year of construction, structural degradation). In a preliminary assessment of buildings, obtaining and incorporating exhaustive details of those factors are not feasible. In this chapter, building vulnerability parameters considered are (i) building type, (ii) vertical irregularity (VI), (iii) plan irregularity (PI), and (iv) year of construction (YC). In addition, reduction in member capacity can occur to structures that were damaged from previous earthquakes and/or deteriorated (e.g; corrosion) (DD). PI and VI are aggregated to compute increase in demand. Furthermore, YC and DD are aggregated to compute decrease in capacity. Finally, the building type, increase in demand and decrease in capacity are aggregated to quantify building vulnerability. It should be noted that this framework is generic and it can be applied to other regions; for example, Tesfamariam and Saatcioglu (2008) applied it to US buildings, Tesfamariam and Saatcioglu (2010) applied it to Turkish buildings, and Tesfamariam and Wang (2012) applied it to schools and civil buildings in the state of Oregon, US.

The states considered for each input parameter, PI, VI, YC, DD and SSH, are shown in Table 7.7. The intermediate nodes, are computed using CPTs that are generated through expert knowledge and training from past earthquake damage. The discrete states considered for the intermediate nodes are:

Table 7.7 Description of the basic input parameters, states and unconditional probabilities

Variable	State	Value
Plan irregularity (PI)	Yes	–
	No	–
	Unknown	–
Vertical irregularity (VI)	Yes	–
	No	–
	Unknown	–
Year of construction (YC)	Old code	<1940
	Moderate code	1940–1970
	New code	>1970
Deterioration/degradation (DD)	Yes	–
	No	–
	Unknown	–
Site-specific hazard (SSH)	SSH ^{VL}	<0.40
	SSH ^L	0.40–0.57
	SSH ^M	0.57–0.90
	SSH ^H	0.90–1.1
	SSH ^{VH}	>1.1

Table 7.8 CPT for building damage

(SSH, BV)	BD(BD ^{VL} , BD ^L , BD ^M , BD ^H , BD ^{VH})
(SSH ^{VL} , BV ^{VL})	(0.95, 0.5, 0, 0, 0)
(SSH ^{VL} , BV ^L)	(0.90, 0.10, 0, 0, 0)
...	...
(SSH ^M , BV ^{VH})	(0, 0, 0.10, 0.15, 0.75)
...	...
(SSH ^{VH} , BV ^{VH})	(0, 0, 0, 0.5, 0.95)

- Increase in demand, ID: ID^{VL}, ID^L, ID^M, ID^H, and ID^{VH}.
- Decrease in capacity, DC: DC^{VL}, DC^L, DC^M, DC^H, and DC^{VH}.
- Building vulnerability, BV: BV^{VL}, BV^L, BV^M, BV^H, and BV^{VH}.
- Building damage, BD: BD^{VL}, BD^L, BD^M, BD^H, and BD^{VH}.

where the superscript are defined as very low (VL), low (L), medium (M), very high (VH) and extremely high (VVH), respectively. A snapshot of the CPTs for building damages are provided in Table 7.8.

7.5.2 Example

The Northridge Earthquake with M_w 6.7 struck the San Fernando Valley on January 17, 1994. To demonstrate quantification of damage assessment

Table 7.9 Building damage state classifications

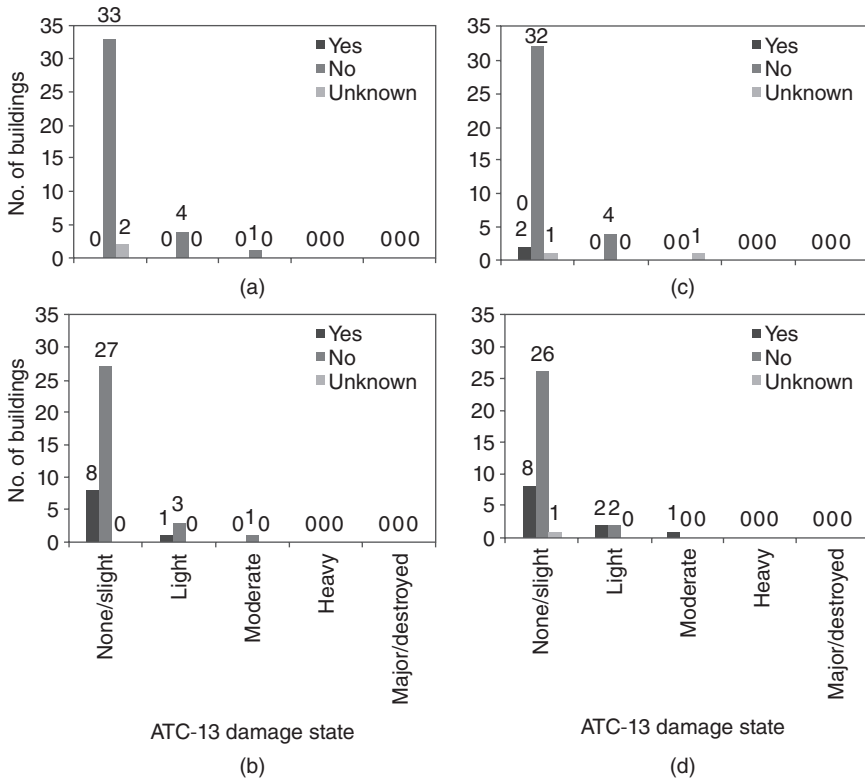
Damage state	Damage factor range (%)	Central damage factor (%)	Description
None	0	0	No damage
Slight	0–1	0.5	Limited localized minor not requiring repair
Light	1–10	5	Significant localized damage of some components generally not requiring repair
Moderate	10–30	20	Significant localized damage of many components warranting repair
Heavy	30–60	45	Extensive damage requiring major repairs
Major	60–100	80	Major widespread damage that may result in the facility being razed, demolished, or repaired
Destroyed	100	100	Total destruction of the majority of the facility

procedure, ATC-38 (ATC 2001), 1994 Northridge earthquake building performance and strong motion data are used. Following ATC-13 (ATC 1985), damage states are categorized into seven distinct states: *none*, *slight*, *light*, *moderate*, *heavy*, *major*, and *destroyed*. The damage states, range of damage factors and descriptions are summarized in Table 7.9.

7.5.3 Data structure

The steel building types are S1 (steel moment frame with stiff/flexible diaphragms), S2 (steel braced frame with stiff/flexible diaphragms), S3 (steel light frame), S4 (steel frame with concrete shear walls), and S5 (steel frame with infill masonry shear walls). In this chapter, no distinction is made on the type of diaphragms, and stiff diaphragms are considered for all buildings. Further, the corresponding building modifiers (summarized in Fig. 7.13) and strong-motion data are obtained from the ATC-38 database. ‘Discontinuous columns’ are used as a surrogate measure of the vertical irregularity. Similarly, ‘other torsional imbalance’ and ‘plan irregularities’ are used as a surrogate measure of plan irregularity.

The spectral acceleration $S_a(T_1)$ is quantified using empirically computed fundamental period T_1 (e.g. Saatcioglu and Humar 2003) and the corresponding response spectrum reported in the ATC-38 database (Fig. 7.14b). Figure 7.14 shows histograms of performance associated with ground failure (lateral ground movement) (Fig. 7.14a), spectral acceleration (Fig. 7.14b), year of construction (Fig. 7.14c), and deterioration of structures (Fig. 7.14d).

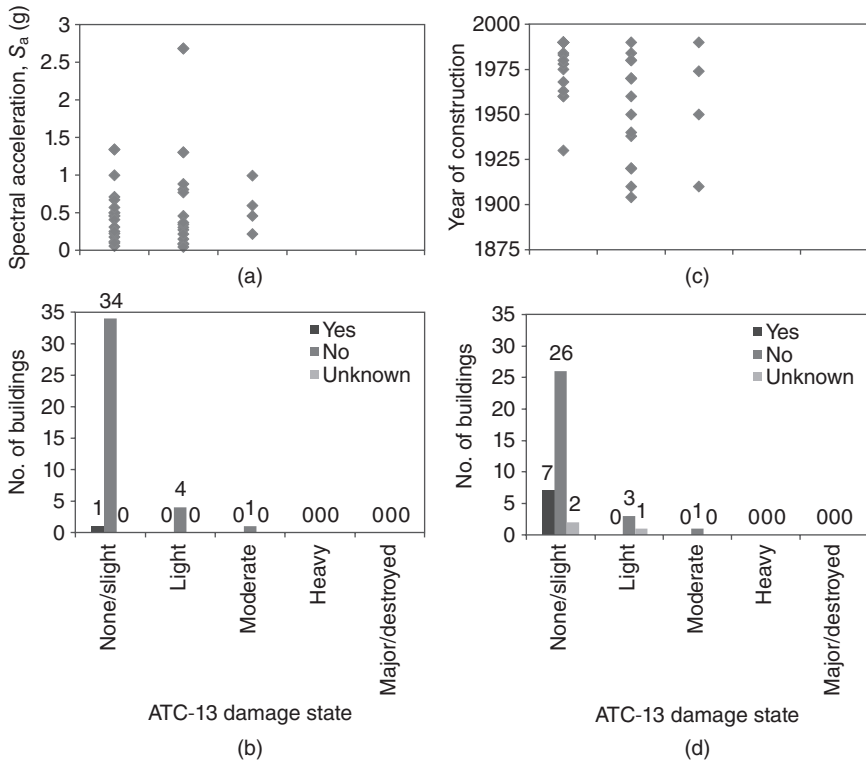


7.13 Performance modifiers for building vulnerability: (a) discontinuous columns; (b) plan setback; (c) torsional imbalance; and (d) plan irregularity.

7.5.4 Conditional probability development

This example was implemented in commercially available Bayesian network software, Netica. Once the hierarchical structure is developed (Fig. 7.12), The Netica¹, offers three alternatives for learning models; regular learning, expected maximization (EM) learning, and gradient learning. The regular learning entails learning by applying Bayesian conditional probability to statistical models built from data loaded into the nodes of the network. For regular learning, the inputs (PI, VI, YC, DD, SSH) should be in the same level, and directly connected to the output (BD). However, as shown in Fig. 7.12, in case where hidden variables are introduced (ID, DC, BV), the EM and/or gradient learning algorithm should be used.

¹Norsys Netica, Advanced Topics: Missing Data and Hidden Variable, Online Tutorial (http://www.norsys.com/tutorials/netica/secD/tut_D1.htm#EMLearning).



7.14 Seismic hazard and performance modifiers for the decrease in demand: (a) spectral acceleration; (b) lateral ground movement; (c) year of construction; (d) deterioration from previous earthquake.

7.5.5 Damage quantification using BBN

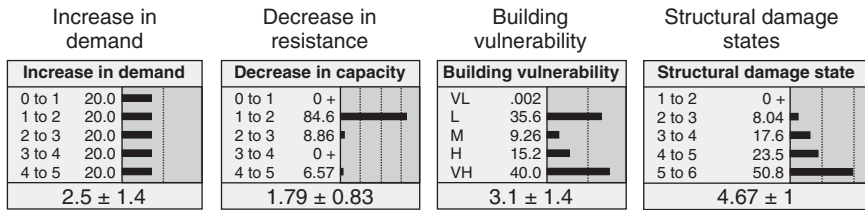
The aggregation through the hierarchical structure is illustrated below. The illustration is carried out for a Building ID = CDMG087-CG-213, with the number of stories $N = 3$. During the 1994 Northridge earthquake, the building was identified as damage state of 'Moderate'. From the *walk down* survey, the building performance modifiers summarized in Table 7.10.

The intermediate step will be computed to obtain the value for building vulnerability. Finally, selecting the values for $SSH = 0.769$ g, the building damage is computed. Snapshots of the BBN results at each step are shown in Fig. 7.15. Results in Fig. 7.15 show that, the probabilities associated with ID, DC, BV and BD are:

- Increase in demand, ID: $(ID^{VL}, ID^L, ID^M, ID^H, \text{ and } ID^{VH}) = (0.20, 0.20, 0.20, 0.20, 0.20)$.

Table 7.10 Building performance modifiers from the walk down survey

Building performance modifiers	Field observation
Structural system (BT)	Steel frame with concrete shear walls (S4)
Site seismic hazard (SSH)	0.769 g
Vertical irregularity (VI)	No
Plan irregularity (PI)	Yes
Deterioration of structure (DS)	No
Year of construction (YC)	1950



7.15 Results of BBN intermediate nodes.

- Decrease in capacity, DC: $(DC^{VL}, DC^L, DC^M, DC^H, \text{ and } DC^{VH}) = (0, 0.84, 0.09, 0, 0.07)$.
- Building vulnerability, BV: $(BV^{VL}, BV^L, BV^M, BV^H, \text{ and } BV^{VH}) = (0, 0.84, 0.09, 0, 0.07)$.
- Building damage, BD: $(BD^{VL}, BD^L, BD^M, BD^H, \text{ and } BD^{VH}) = (0, 0.08, 0.18, 0.23, 0.51)$.

It is interesting to note that the probabilities for ID have an equal mass for all ID values (ID^{VL} , ID^L , ID^M , ID^H , and ID^{VH}). Since the intermediate nodes are calculated from the field observational data, this might be the case where there is a lack of information. In such a case, equal mass is assigned. For the interval-valued damage states, 1-2, 2-3, 3-4, 4-5, 5-6, the corresponding mean values are multiplied by the probabilities, to obtain the mean damage state of 4.67 (Fig. 7.15). The actual damage state from the Northridge earthquake was moderate. The example highlights that, to get the CPT from the field observation, there should be sufficient datasets. Thus training the CPT with incomplete information is not desirable, and expert knowledge should be solicited.

7.6 Conclusions and future trends

Consideration of seismic hazard, vulnerability assessment, and consequence of failure is subject to different levels of uncertainty and model complexity.

In this chapter, a general consideration of soft computing techniques in seismic risk assessment was given. A review on the application of soft computing in seismic risk assessment was provided. Bayesian belief network (BBN) is a graphical model that permits a probabilistic relationship among a set of variables. In this chapter, the application of BBN for liquefaction assessment and seismic risk assessment at regional and individual levels was provided. For the liquefaction model, the BBN structure is generated from historical data. To generate the BBN for liquefaction assessment, the local score metrics are considered for structure learning. The process of learning a network structure is implemented by maximizing the quality measure of the network given the training data set. The score of the whole network is decomposed as the sum (or product) of the score of the individual nodes. This will allow for local scoring and local search methods. Four searching algorithms, K2, hill climbing, TAN, and Tabu search, were tested with the available dataset. For these search algorithms, the hill climbing algorithm furnished an intuitive result. Furthermore, through the domain knowledge base, an additional hierarchical structure was proposed, and CPTs are generated through the learning algorithm. Indeed, this domain-knowledge-based structure had a better prediction capability, and highlights the need for such approach in developing this type of system.

For regional risk assessment, a heuristic-based BBN was proposed. The examples highlighted utility of the BBN in seismic risk assessment. For individual buildings, the CPT was generated through the EM training algorithm. This example highlighted that, since the dataset provided was not sufficient, the training algorithm will miss these points, and can furnish results that are not reliable. From the general discussion and examples, salient points and future works are:

- For distributed assets (e.g. buildings, bridges), the BBN model of individual buildings can be enhanced by considering spatially correlated ground motions (see Chapter 3).
- Consideration and incorporation of time-dependent factors, ageing and deterioration, in the risk assessment. This can be done through dynamic Bayesian network models.
- The need for expert knowledge in generating the preliminary BBN structure before implementing the machine learning.

7.7 References and further reading

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Structural vulnerability analysis of civil infrastructure facing seismic hazards

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Abstract: Managing risks to structures or infrastructure systems requires a thorough analysis of uncertainty. Structural reliability methods are suitable for dealing with parametric uncertainty, but many other aspects of system uncertainty remain unaddressed. These require an analysis of the form of the structural system as well as the structural response to hazards, such as material defects, loading conditions, and accidental damage. An approach to identify structural risks due to failure scenarios, which may remain hidden during a normal response analysis, has been developed. Such an analysis leads to identification of failure scenarios with high vulnerability. Different actions on a structure might contribute differently to a failure scenario; an interval probability theory can be used to combine the evidence associated with different actions. Structural risk is obtained as a function of probability of the identified failure scenarios and their consequences. The approach has been generalized to other infrastructure systems.

Key words: safety, vulnerability, robustness, risk analysis, uncertainty, disproportionate failure.

8.1 Introduction

One of the main objectives of a design process is to ensure the safety and serviceability of the product. Traditionally, the safety of components or systems has been expressed by safety factors as the ratio between capacity and demand (see e.g. Pugsley 1966). Although easy to understand, it does not account for uncertainties in material and loading. Reliability analysis tools (see e.g. Thoft-Christensen and Baker 1982; Melchers 1999) have proved useful to explicitly account for statistical variations in material and demand parameters. However, they do not address all aspects of the system uncertainty. For example, it is common to start with a predefined model of load and to focus on failure scenarios that are most likely to occur. In some cases, this may lead to a false sense of safety if there exist failure scenarios with relatively low likelihood but with large consequences. The realization of these low-probability and high-consequence failure scenarios cannot be ignored for important structures (Taleb 2007; Government Office for Science 2012). In particular, human error and sabotage may contribute

significantly to the occurrence of such major failure scenarios (see Kamienetzky 1991 and Delatte 2009 for several examples, including Hyatt Regency Walkways, Skyline Plaza, Alfred P. Murrah building). Thus the whole problem of dealing with total system uncertainty requires an effective management of structural risks anticipating not just failure scenarios that are most likely to occur but also those which lead to catastrophic consequences. This has motivated the development of an approach, called *structural vulnerability analysis* (Wu *et al.* 1993; Lu *et al.* 1999; Agarwal 2001; Agarwal *et al.* 2001a, 2003). The approach has been generalized to analyse infrastructure systems which can be modelled as a network system (Agarwal *et al.* 2001b, 2011). Once vulnerable failure scenarios have been identified, their likelihood can be assessed by gathering the evidence available for those failure scenarios, thus enabling a measure of risk to a structure or to a system (Agarwal 2005).

The objective of this chapter is to present the vulnerability analysis approach, with examples, to manage structural or system risks. The chapter is organized as follows. Firstly, interpretations of different related terms, e.g. vulnerability, hazards and risk, are set out. Secondly, a theory of vulnerability of structures is introduced. It forms a basis for finding failure scenarios that are later examined for the chance of failure under specific actions and loads. Thirdly, natural and man-made actions having the potential to cause failure scenarios are examined. Structural risk assessment is carried out by considering the consequences of different vulnerable scenarios and the probabilities of actions causing them. Finally, generalization of the vulnerability approach to infrastructure systems is outlined.

8.2 Vulnerability, hazard and risk

Vulnerability captures the idea of susceptibility to damage. The susceptibility arises from the form of the system within a given context. An inappropriate form if damaged may lead to consequences which are disproportionate to the initial damage. For example, the Ronan Point high-rise block of flats in London was a vulnerable structure, because a small domestic gas explosion in one flat caused the whole side of the building to collapse. While there is little variation in the meaning of vulnerability across different disciplines, the way it is measured varies. In earthquake engineering, vulnerability of a component is its susceptibility from an earthquake of a given intensity. An earthquake in California may produce very different consequences from the same strength earthquake in Iran (developing country). In social systems, adaptability and stability to change or damage determine the vulnerability. A probabilistic measure of vulnerability, defined as the ratio of the failure probability of the damaged system to the failure probability of the undamaged system (Lind 1995), can be applied to almost any system.

Whatever the measure is, vulnerability is a property of a system and it concerns consequences in relation to the initiating damage events.

The core idea of *hazard* is potential for danger. It is a state of the system or process with a potential for harm. For example, a wind load is potential for damage to property; extreme events such as hurricanes and earthquakes are a potential for large loss of life and damage to infrastructure. Hazards can be managed by removing them if at all possible, by reducing them, by modifying the system or by remedying them so that the risk is acceptable. Hazards can incubate over a period of time (Turner and Pidgeon 1998). Blockley (1992) has used a balloon analogy to illustrate this. The air pressure is analogous to the proneness to failure of the system. Hazardous events increase the predisposition to failure and the balloon grows in size. Letting out some of the air parallels the effect of risk management decisions that remove some of the predisposing events and mitigate the proneness to failure. An overstretched balloon represents 'an accident waiting to happen'.

The distinction between hazard and vulnerability is a subtle one. Physical scientists view vulnerability in terms of likelihood of occurrence and impact of physical events such as natural hazards. Social scientists tend to view vulnerability as the socio-economic factors that determine people's ability to cope with stress or change (e.g. Boulding 1989; Blaikie *et al.* 1994). The broad distinction, however, is made based on whether vulnerability is comprehended as potential damage to a system or as a state that exists within a system before it encounters a hazard event. The latter view indicates that vulnerability exists independently of external hazards. Social vulnerabilities of this type might include poverty and inequality, access to insurance and housing quality. Material defects or lack of corrosion resistance also fall in the latter category. Whatever the definition or measure, one of the important ways of managing risks is to reduce vulnerability. In an earthquake, potential for catastrophic disasters does depend on vulnerability of the infrastructure and so in that sense, vulnerability is a particular form of hazard – a hazard which is internal to the system. The concept of robustness is directly related to vulnerability. *Robustness* is the ability of a system to fulfil its function in the face of uncertain and adverse conditions such as the loss of a component or abnormal changes in the demands. A system which is vulnerable in any one way cannot be robust. There are several ways to increase the robustness of a structure but it is difficult to quantify it. *Fragility* is another term that has developed a specialist meaning. The word implies easily damaged or broken, but it is often used to describe the probability of a stated level of damage for a specific hazard, e.g. an earthquake.

Risk is commonly defined as the combination of chance and consequences of an event. Kaplan *et al.* (1981) defined risk by a triplet (h, p, c) where h = what can go wrong, p = how likely is that to happen, and c = if it does happen, what are the consequences? Generally, hazard analysis is

concerned with h alone and reliability analysis examines h and p together (e.g. the probability of moment due to an earthquake exceeding the capacity of the beam). Vulnerability analysis identifies h with maximum c so that action can be taken to reduce c – this is particularly important for critical systems. One needs to be careful about the terminology. In disaster reduction literature (see e.g. UN ISDR 2004), risk is calculated as the product of hazard and vulnerability. A useful summary of different specialist uses of risk, hazard and vulnerability (particularly focusing on climate change research) was given in a report by the Tyndall Centre for Climate Change Research (2003). The core idea of risk remains as the chance of some future hazard and the consequences that will be caused if it comes about. When we think of risk, we are thinking of a potentially dangerous situation which might or might not exist in the future with consequences we want to avoid. It is also important to realize that risk has to be understood in context. In the context of earthquake engineering, hazard is the earthquake and vulnerability depends on the design, construction and maintenance of a facility. The risk is the anticipated loss when the hazard impacts on the facility.

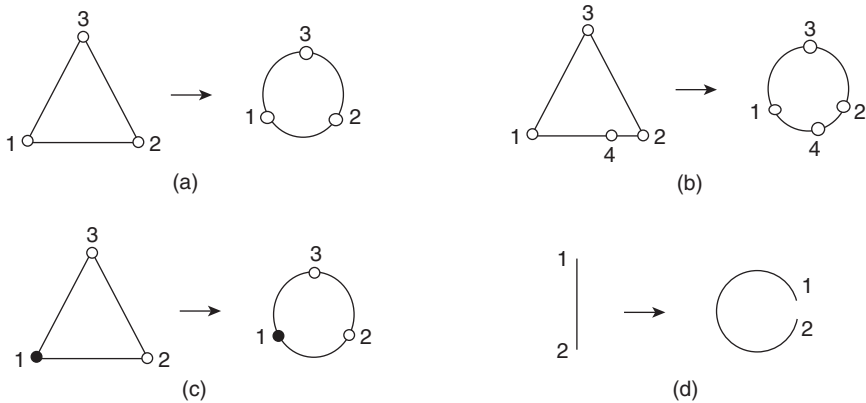
8.3 Identification of vulnerability

A structure is robust if it can withstand arbitrary damage, whereas it is vulnerable if small damage produces disproportionately large consequences. The latter concept has been developed into structural vulnerability analysis (Wu *et al.* 1993; Lu *et al.* 1999; Agarwal *et al.* 2001a, 2003). It is a theory of form and connectivity, and aims at identifying weak links in a structure. In brief, it requires the following three processes:

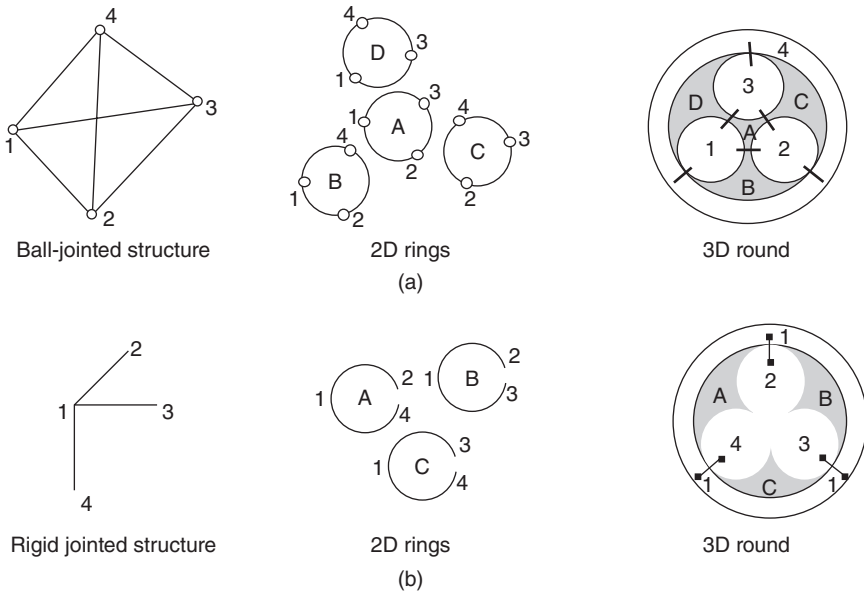
- building a model of the form of a structure using the concept of rings and rounds;
- representing the structure in a hierarchical form through successive clustering; and
- identifying various failure scenarios by a search through the hierarchical model.

8.3.1 Rings and rounds

The function of a structure is to transmit force from one point to another in space without losing its integrity. A consideration of form is essential to that integrity. A structural ring or a structural round is a minimum structural path which can withstand a set of forces in 2D and 3D, respectively. For a pin-jointed structure, three members connected together form a just-stiff ring (Fig. 8.1a). Introduction of a pin (or a cut) into a just-stiff ring turns it into a mechanism (Fig. 8.1b). If a pin joint is replaced by a rigid joint an



8.1 Structural rings: (a) a just-stiff ring; (b) a mechanism; (c) an over-stiff ring; and (d) an open ring.



8.2 Structural rounds: (a) a ball-jointed round and (b) a rigid-jointed round.

over-stiff ring is obtained (Fig. 8.1c). A flexural member can be represented as a ring with a cut (Fig. 8.1d).

A structural round is composed of 2D structural rings described above. It can resist loads from any arbitrary direction in space. Figure 8.2(a) shows the simplest ball-jointed structural round (a tetrahedron) which has four rings. It is called a basic round, because many trusses can be built by adding

more members to it. A structural round which is not constructed using a tetrahedron but is just stable is a higher round. The algorithm to identify these rounds is quite complex and has been described elsewhere (Agarwal *et al.* 2001a). Three members arranged orthogonally at a rigid joint produce a well formed structural round (Fig. 8.2b). This round consists of three open rings. A beam is the simplest space structure forming a structural round. Such a round is called a leaf round.

The quality of the form of a ring or round is a function of the member property, orientation, and connectivity. This quality is defined as the well-formedness Q and can be obtained using the stiffness sub-matrices associated with the joints in a structural ring or round:

$$Q = \frac{1}{N} \sum_{i=1}^N \det(\mathbf{K}_{ii})$$

where $\mathbf{K}_{ii}$ is the stiffness sub-matrix associated with any joint i and N is the total number of joints in the ring/round. This measure is independent of the chosen coordinate system and increases with increasing resistance to loads along the principal directions. Conceptually, principal directions for a structural ring are analogous to principal axes for an I section or an unsymmetric section. Then the measure of well-formedness is the product of principal stiffnesses analogous to second moment of areas about the principal axes. It increases with member stiffness and with joint stiffness. Any reduction of the well-formedness indicates damage to the structure.

8.3.2 Clusters and hierarchy

The purpose of clustering is to successively bring together well-connected members thus leading to a hierarchical representation of the structure. A ring or a round itself is a cluster which can grow by adding more members into it. When a cluster at a higher level is formed, it captures certain attributes from members at lower level as well as new members joining it. The attributes of a structural member (or cluster) used for cluster formation include well-formedness of the structural cluster, minimum damage demand of the structural cluster, nodal connectivity of the structural cluster, and distance from the reference cluster. The well-formedness of a structural cluster is calculated similarly to that for a structural round using all of the joints in the cluster. It is assumed that the damage demand is directly proportional to the loss of the principal stiffness caused by a deteriorating event. Nodal connectivity is the total number of members connecting to the joints in the cluster. For structures, the reference cluster is usually the ground.

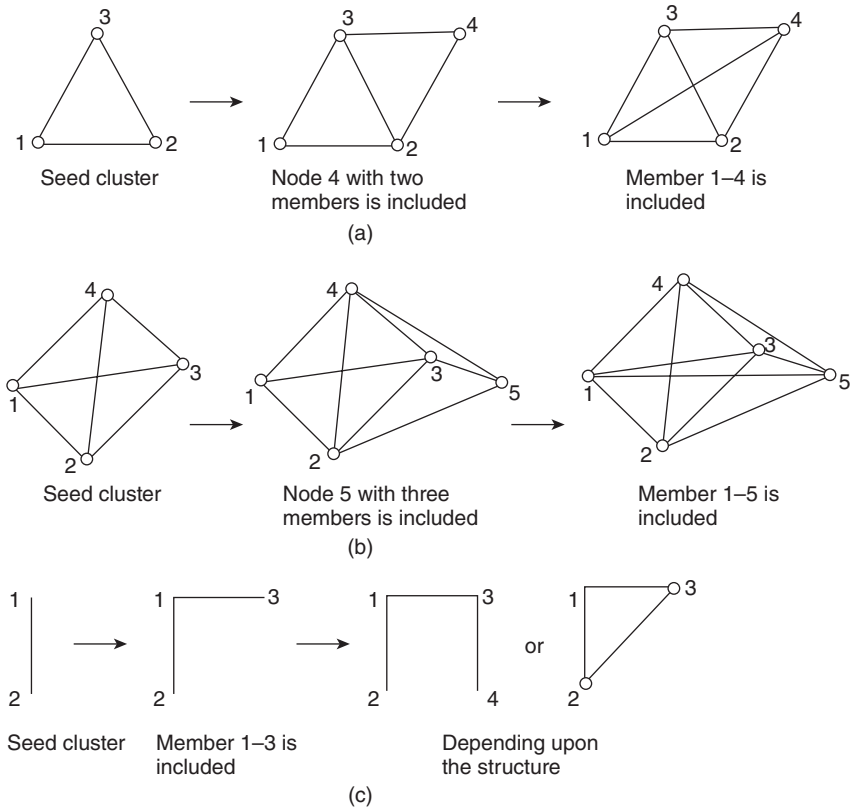
The clustering process starts with the identification of seed clusters. A ring in 2D or a round in 3D with the highest well-formedness is chosen as

the first seed cluster. Then other members are brought in successively. There are three distinct stages in the cluster formation process: Stage I – initial clustering, Stage II – secondary clustering, and Stage III – reference clustering. In Stage I, members are allocated to an existing cluster when doing so increases the well-formedness of the cluster. If two separate members when added to an existing cluster separately produce the same well-formedness, then other criteria are used to decide which of them should be admitted. They are, in order, the maximum of the minimum damage demands, the maximum nodal connectivity, and the maximum distance from the reference cluster. When there are no more members or clusters to be added, which will increase the well-formedness of the cluster, then another cluster is initialized. In Stage II, the value of the well-formedness of the growing cluster will decrease as loosely connected existing clusters are combined. Those clusters are merged together first which result in least reduction of the well-formedness. The criteria to terminate the process is that no further valid structural clusters can be formed except by including the reference cluster. The inclusion of the reference cluster is the final Stage III of cluster formation. The clustering finishes when the whole structure is one complete cluster.

The exact growth of a cluster much depends on the nature of joints in the cluster. In a 3D truss, a tetrahedron can grow in many different ways by adding one or more members at a time. In a completely rigid-jointed structure, a seed cluster grows by adding one neighbouring member or a cluster at a time. Some typical clustering sequences for 2D/3D trusses and frames are shown in Fig. 8.3. All members finally evolve, through different clusters at a finite number of levels of description, into a single cluster which represents the whole structure. Such a hierarchical representation (for an example, see Fig. 8.7) results in continuing selective loss of details up the hierarchy. It is a map showing how the clusters are inter-related.

8.3.3 Deteriorating events and failure scenarios

Deteriorating events affect structural form; they cause failure at the lowest level in the hierarchy but may affect a large part of the hierarchy. The detail and nature of the actions causing the deterioration are not considered at this stage. These may arise, for example, due to excessive loading, natural disaster, accidental force, or human errors. A deteriorating event has an associated concept of damage demand which measures the effort required to trigger that event. A failure scenario may have one or more deteriorating events resulting in a failure mechanism. The consequences of failure events of the same nature vary and largely depend upon how the corresponding member or cluster is configured in the system hierarchy. The references (i.e. supports) play an important role in determining the failure consequence.



8.3 Typical clustering sequences: (a) pin-jointed structure; (b) ball-jointed structure; and (c) rigid-jointed structure.

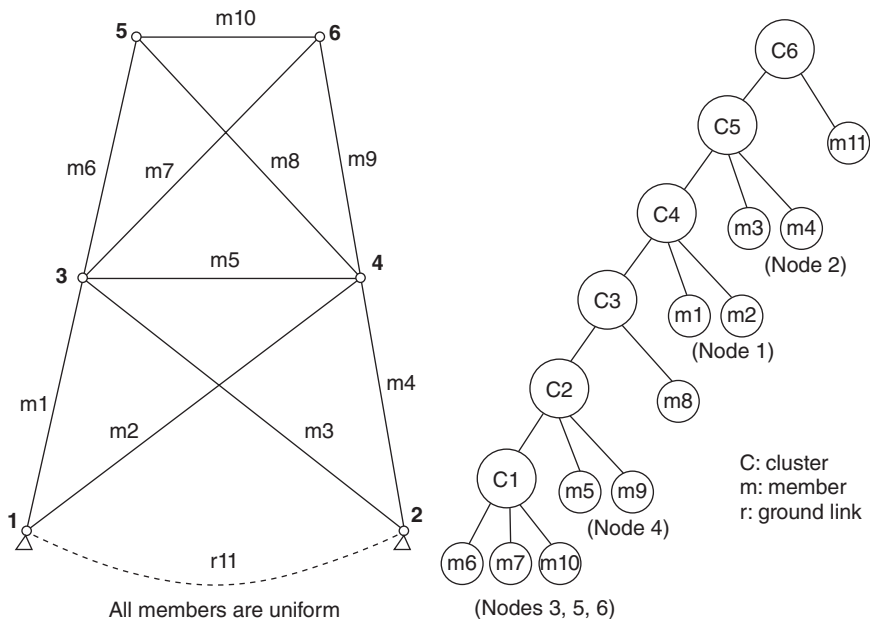
The description of damage scale is related to the change in the quality of form (i.e. well-formedness). The loss in structural well-formedness is calculated as the difference between the structural well-formedness of intact structure and that of a damaged structure. The ratio of this loss in structural well-formedness to the well-formedness of the intact structure defines separateness. The ratio of separateness with respect to damage demand gives vulnerability index.

A *failure scenario* is a set of deteriorating events that reduce the well-formedness of a structure. Failure scenarios are identified by exposing a structure to deteriorating events while traversing down the hierarchy. A deteriorating event could be a cut or formation of a pin. Complete failure of a joint is the same as the failure of all members meeting at that joint. Although a large number of failure scenarios are possible, five types of failure scenarios are important. The *minimum demand scenario* is the easiest way to cause damage to the structure, no matter how small. It is identified

as occurring in the member with the minimum damage demand. The *minimum failure scenario* is the failure scenario that causes the least loss of the form of the structure. It is the failure of the member which joins the cluster last, because it has the least connectivity with the rest of the structure. *Specific failure scenarios* are those where the designer has a particular interest because of the particular conditions of the structure (for example, it is known to be corroded or exposed to a specific threat). The *total failure scenario* is the situation where total collapse is triggered with least effort. The *maximum failure scenario* has the highest vulnerability index which is the ratio of the damage consequence to the effort required to cause that damage. The total failure scenario and the maximum failure scenario are identified through a search down the structural hierarchy. The hierarchy helps in picking up those scenarios which have high vulnerability index values.

8.3.4 Example 1

Figure 8.4 shows a pin-jointed structure with uniform member properties. It could well represent a part of an offshore structure. There are ten structural members (m1 to m10). The reference cluster (i.e. ground) provides another member (r11) with infinite stiffness (called reference member).



8.4 Structure in Example 1 and its hierarchical representation.

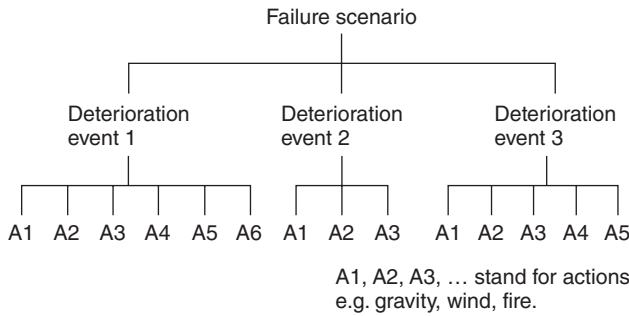
Figure 8.4 also shows the clustering hierarchy. There are six possible rings formed by members [1,2,5], [3,4,5], [5,6,8], [5,7,9], [6,7,10], and [8,9,10]. The best ring (in terms of well-formedness defined earlier) amongst these six rings should form a seed cluster. There are three pairs of rings which have identical attributes. Rings formed by members [6,7,10] or [8,9,10] have the highest well-formedness. Hence clustering can start with these and one of them is chosen at random to form cluster C1. This cluster grows to form a higher level cluster C2 by including neighbouring node 4 and members 5 and 9. Next member 8 is added in to form a new cluster C3 with an increase the well-formedness. Then clusters C4 and C5 are formed by including the neighbouring nodes and members. The choice to include node 1 first is arbitrary, because the members connecting to both nodes 1 and 2 are identical and produce the same attributes when added in to cluster C4. Lastly, the ground is added in to form cluster C6 which is the whole structure.

The clustering hierarchy is used to find failure scenarios. Members 2 and 3 have the minimum damage demand because they are the longest (cross-sectional areas being the same). The minimum failure scenario is to damage member 3 or 4 because they join the growing cluster last. The maximum failure scenario is to pin members 2 and 3 because they produce the maximum consequence with minimum damage demand. The total failure scenario is identical to the maximum failure scenario in this example. Other total failure scenarios are to pin members 3 and 4, or members 1 and 2, but they have less vulnerability index. These are rather obvious results but the example helps to illustrate the analysis procedure.

8.4 Analysis of risk

In vulnerability theory, actions causing damage are not specified and damage may result from one or more possible actions. These may include degradation of material (e.g. corrosion of steel), extreme natural hazards, vehicle impact, intentional damage (e.g. terrorist bombing), fire, and many more. Some actions have the potential to cause damage to many structural members simultaneously. It is possible, at least qualitatively, to assess the likelihood of certain actions.

Each failure scenario with the potential to cause large consequences, as obtained from vulnerability analysis, is considered in turn and an assessment is made about the likelihood. This requires the consideration of all possible actions that may lead to this particular failure scenario. Hence there is a need to gather evidence for each deterioration event in this failure scenario due to each action as illustrated in Fig. 8.5. For ease of explanation, let us focus on the first deterioration event. It may be caused due to any action such as higher live load, strong earthquake load, corrosion, sabotage, etc. Thus evidence needs to be collected about the dependability of each



8.5 Propagation hierarchy for evidence for a failure scenario.

action causing the first damaging event. It is noted that two or more actions may be acting simultaneously thus adding to the complexity of the assessment. Interval Probability Theory (Cui and Blockley 1990) has the potential to account for this dependency and has been used for, for example, earthquake hazards (Sanchez-Silva *et al.* 1996) and corrosion damage (Sarveswaran *et al.* 1998).

In Interval Probability Theory, an interval number is used as the probability measure in order to capture fuzziness and incompleteness. For an event or preposition A , the interval probability is represented as

$$P(A) = [S_n(A), S_p(A)]$$

where $S_n(A)$ and $S_p(A)$ are defined as the lower and upper bounds of the probability of A . $S_n(A)$ represents the extent to which it is certainly believed that A is true, whereas $1 - S_p(A)$ represents the extent to which it is certainly believed that A is false. $S_p(A) - S_n(A)$ is the extent of uncertainty regarding whether A is true or false. Classical probability corresponds to the particular case where $S_p(A) = S_n(A)$.

In the application of Interval Probability Theory for the assessment of structural risks, we are interested in calculating the probability of damage events to members subjected to several actions. It is therefore necessary to determine the union of probabilities for different actions. For failure scenarios with more than one events, union of probabilities of damage events will also be needed. The former can be assumed to be independent for many cases but the latter requires the consideration of dependence between these events in the calculation of the union.

Cui and Blockley (1990) have used a parameter ρ to describe the degree of dependence between two events. For two events A and B with probabilities $P(A)$ and $P(B)$, the degree of dependence is defined as

$$\rho = \frac{P(A \cap B)}{\min[P(A), P(B)]}$$

In general, it is difficult to know precisely the degree of dependence and thus an interval number $[\rho_l, \rho_u]$ may be used. Where probabilities of two events A and B are also written as interval number, the following equations are obtained for the union and intersection of the events (Cui and Blockley 1990):

$$S_n(A \cup B) = S_n(A) + S_n(B) - \rho_u \min(S_n(A), S_n(B))$$

$$S_p(A \cup B) = S_p(A) + S_p(B) - \rho_u \min(S_p(A), S_p(B))$$

$$S_n(A \cap B) = \rho_l \min(S_n(A), S_n(B))$$

$$S_p(A \cap B) = \rho_u \min(S_p(A), S_p(B))$$

The assessment of the likelihood of a failure scenario is made by choosing

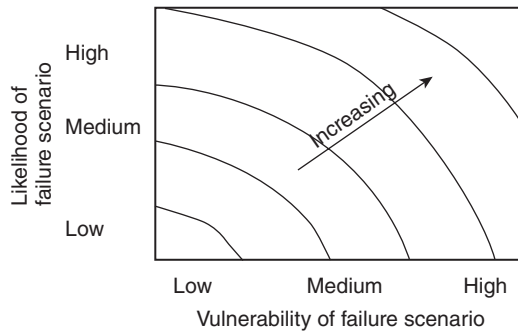
- a measure of evidence that an action is likely to cause damage;
- a measure of confidence in that assessment; and
- a measure of the likelihood (importance) of that action irrespective of the degree of damage it may cause.

The above measures can be either numeric or linguistic. Single point values may be available for certain actions that are known precisely (e.g. dead load). For actions such as earthquake load or snow load, an interval number may be more appropriate considering the uncertainty in assessing the extremes. For actions, such as accidental damage or sabotage, one can at best give linguistic grades for damage. But these linguistic descriptions can be mapped to an interval number using various schemes. Following the scheme in Dester and Blockley (1997), judgement of damage extent, for example, with *little* evidence with *moderate* confidence may be expressed as an interval number of [0.13, 0.45]. The higher the confidence, the smaller is the interval. Below a certain level of confidence, the interval gets much larger. The likelihood (or importance) of an action can also be assessed on a 0 to 5 scale with extremes being *very high* and *very low*.

8.4.1 Risk calculation

The exploitation of vulnerabilities knowingly or unknowingly could cause grave harm to the structure, occupants, and possibly to a much bigger system. Vulnerability together with threat can produce high risks. So long as there is no undesirable action to exploit the vulnerability, it is only a potential risk. However, if a threat is active, the combination of the active threat and the vulnerability creates a real risk to the structure and its users.

Hence, after having identified failure scenarios, it is important to consider the probability of actions and the likelihood of damage occurring. A con-



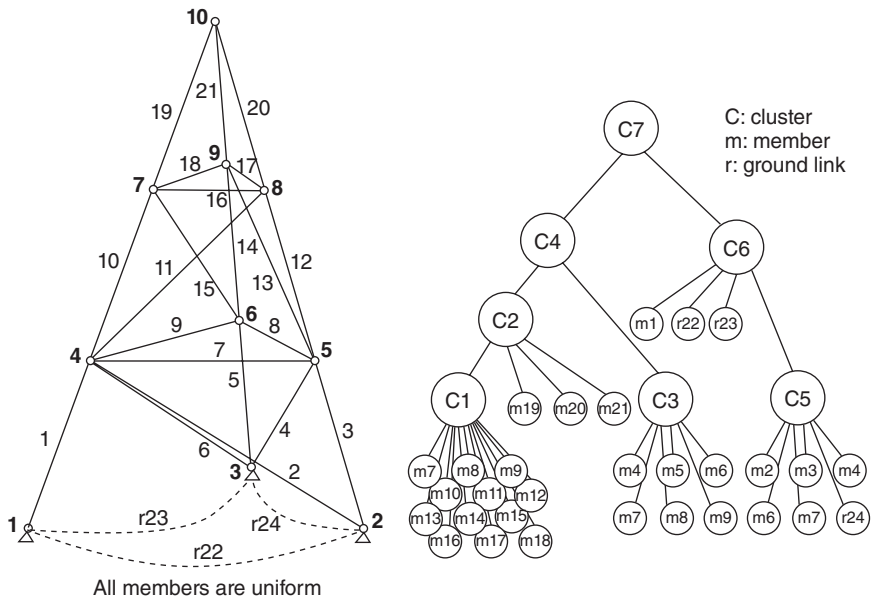
8.6 Structural risk as a function of vulnerability and likelihood of failure scenarios.

sideration of context is also required. For example, for a building to be used as a theatre, an acceptable risk level will be lower than when it to be used as a warehouse.

With the distinction between damage due to an action and consequence of damage, risk assessment requires a slightly different approach. For the latter, it is necessary to define the probability of damage due to an action (or a combination of actions) and then the probability of that damage leading to the consequences. In other words, the latter depends upon the actions present after the damage. For example, if stress levels of structural members were already high before damage, self-weight of the members alone may lead to failure. The former is rather complicated because of many uncertainties and perhaps best captured using an evidence-based approach (for example, Interval Probability Theory discussed above). Figure 8.6 helps to illustrate the risk as a function of vulnerability of failure scenarios and their likelihood.

8.4.2 Example 2

Figure 8.7 shows a slightly more complicated ball-jointed structure than the previous example, but again with uniform member properties. A careful common sense study of this structure reveals that damage to a member in any storey will also damage the upper storey(s) and that damage to member m1, for example, is one of the most obvious choice for total collapse of the structure. However, it is not easy to predict other vulnerable failure scenarios. Figure 8.7 also shows the clustering hierarchy for this example and the results of unzipping are summarized in Table 8.1. The minimum damage demand is the same for members m2, m4, and m6. Each of these three members forms a minimum demand failure scenario. Members m19, m20, and m21 result in the least loss of form and each of them produces a



8.7 Structure in Example 2 and the corresponding clustering hierarchy.

Table 8.1 Failure scenarios for the structure in Example 2

Minimum demand scenario	Minimum failure scenario	Maximum failure scenario	Specific failure scenario (as an example)
Introduce a pin in any of the members m4 (or m2 or m6)	Introduce a pin in member m19 or m20 or m21	Introduce a pin either in member m2 or m6	Introduce a pin in member m10
Separateness 0.99 (for m4)	Separateness 0.2	Separateness 1.0	Separateness 0.8
Vulnerability index 52.3	Vulnerability index 7.5	Vulnerability index 52.8	Vulnerability index 30.2

minimum failure scenario. Notice that damage to any of these does reduce the well-formedness of the structure, but the structure as a whole still stands. Identification of such degradation of structural quality is a useful aspect of vulnerability analysis. The maximum failure scenario is obtained by introducing a pin either in m2 or m6. This causes total collapse of the structure hence it is a total failure scenario. The intuitive and perhaps more obvious candidate is m1. Certainly a pin in m1 will cause total collapse but as the

member is shorter than m_2 or m_6 and therefore has a higher damage demand (or higher buckling load capacity) it is not the one chosen by this analysis. Clearly however were m_1 to be significant in terms of loading or response analysis or for any non-structural reason, then it is a possible failure scenario with high risk.

A qualitative analysis of the likelihood of the failure scenarios in the above example is done and the results are given in Table 8.2. Different actions are considered and a linguistic assessment of damage to members (column c in the table) is made. For the purpose of this illustration, damage assessment of members and evaluation of the likelihood of actions are based on a simplified quantitative analysis and/or engineering judgement. But these results can be improved by carrying out a rigorous quantitative analysis for different actions. Support for the likelihood of a scenario is summarized in column i . Here actions are assumed to be independent of each other though it will not be the case always. Where a scenario has more than one event, careful consideration has to be given to the degree of dependency between them. Vulnerability of different scenarios (column j) and their likelihood (column i) can be plotted in the form of a risk map. Here only three failure scenarios are discussed but in general there would be a large number of failure scenarios and appropriate actions can be taken to mitigate critical failure scenarios.

8.5 Vulnerability of infrastructure networks

Structural vulnerability theory has been generalized into a vulnerability theory applicable to infrastructure systems that can be represented as a graph. Complementary variables, called ‘across variables’ and ‘through variables’, together with equivalents of Kirchhoff’s laws form the basis of the generalization. These variables can be envisaged as cause and effect or driver and change. For structures, they are force and displacement, and the structural rings developed in structural vulnerability analysis are circuits of force and displacement. Analogous variables for water pipes and traffic networks, which also suffer damage during earthquakes, are defined in Agarwal *et al.* (2001b). The across and through variables are related through component equations. The through variables balance across any section through the circuit and the across variables balance around the circuit.

8.5.1 Water pipe networks

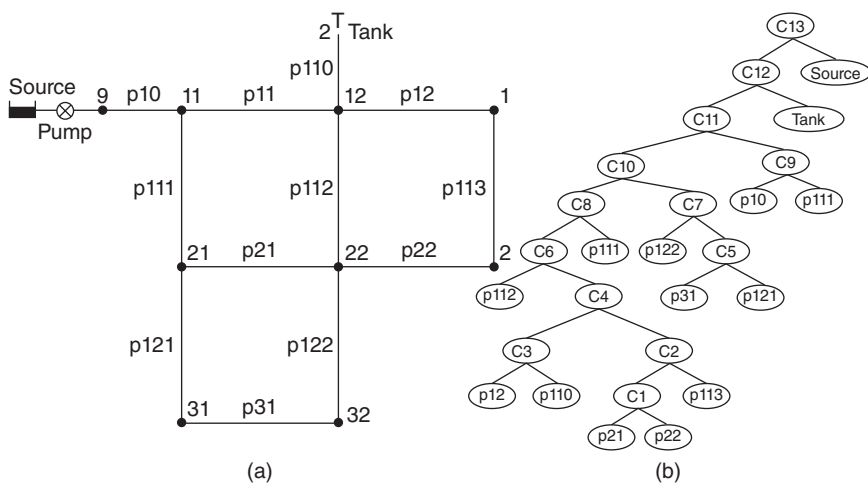
A typical water distribution network is formed with pipes, reservoirs, pumps and control elements. The interconnected pipe network transmits water from reservoir(s) to consumers. However, pipes or other elements are susceptible to failure causing a drop in downstream pressure or performance

Table 8.2 Assessment of structural risk for Example 2

(a)	Action	Linguistic assessment of damage to members	Confidence level in the assessment	Corresponding support interval	Likelihood of the action (on 1–5 scale)	Relative importance of action	Normalized support interval	Accumulated support for likelihood of the scenario	Vulnerability Index	Structural Risk
Max FS: Pin m2 or m6	Gravity load	Very little	High	[0.0, 0.22]	High 4	0.267	[0.0, 0.059]	[0.257, 0.546]	52.8	[13.6, 28.8]
	Wind load	Some	High	[0.35, 0.63]	High 4	0.267	[0.093, 0.168]			
	Fire	Some	Moderate	[0.34, 0.66]	Low 2	0.133	[0.045, 0.088]			
	Explosion	Considerable	Moderate	[0.74, 1.0]	Very low 1	0.067	[0.050, 0.067]			
	Corrosion	Little	Moderate	[0.13, 0.45]	Very low 1	0.067	[0.009, 0.030]			
Min FS: Pin m19 or m20 or m21	Accidental	Some	Low	[0.3, 0.67]	Medium 3	0.200	[0.06, 0.134]	[0.103, 0.370]	7.5	[0.8, 2.8]
	Dead load	Very little	High	[0.0, 0.22]	High 4	0.267	[0.0, 0.059]			
	Wind load	Very little	High	[0.0, 0.22]	High 4	0.267	[0.0, 0.059]			
	Fire	Some	Moderate	[0.34, 0.66]	Low 2	0.133	[0.045, 0.088]			
	Explosion	Some	Moderate	[0.34, 0.66]	Very low 1	0.067	[0.023, 0.044]			
Specific FS: Pin m10	Corrosion	Little	Moderate	[0.13, 0.45]	Very low 1	0.067	[0.009, 0.030]	[0.177, 0.467]	30.2	[5.3, 14.1]
	Accidental	Little	Moderate	[0.13, 0.45]	Medium 3	0.200	[0.026, 0.090]			
	Dead load	Very little	High	[0.0, 0.22]	High 4	0.267	[0.0, 0.059]			
	Wind load	Little	High	[0.15, 0.42]	High 4	0.267	[0.040, 0.112]			
	Fire	Some	Moderate	[0.34, 0.66]	Low 2	0.133	[0.045, 0.088]			
	Explosion	Some	Moderate	[0.34, 0.66]	Very low 1	0.067	[0.023, 0.044]			
	Corrosion	Little	Moderate	[0.13, 0.45]	Very low 1	0.067	[0.009, 0.030]			
	Accidental	Some	Low	[0.3, 0.67]	Medium 3	0.200	[0.060, 0.134]			

of the whole network. Vulnerability analysis of water pipe networks (Pinto *et al.* 2010) follows a similar procedure to that described above for structures. Clusters of connected pipes are formed using head loss as a measure of well-formedness. This relates to the properties of a pipe (e.g. length, diameter and friction factor) and the configuration of the pipe network. Other sources of head loss are also considered. A smaller total head loss indicates a better formed cluster. Where head losses are the same, nodal connectivity and damage demand are used as secondary clustering criteria. Nodal connectivity measures the degree of connection of a cluster with the rest of the network, and damage demand measures the effort needed to introduce a deteriorating event. A deteriorating (i.e. initial failure) event occurs when a pipe bursts or it fails to supply good quality water to a demand point. Currently damage demand is assumed to be proportional to the strength of pipe. Separateness measures the failure consequences and it is obtained as the ratio of total head losses in the damaged network and that of the original network. Vulnerability index measures the disproportionateness of the consequences in relation to the initial damage and is determined as before i.e. the ratio of separateness and damage demand.

Figure 8.8 gives an example water pipe network which is adapted from EPA (2000). A hydraulic analysis was carried out to determine the relative head losses for different pipe segments. Using head losses as the well-formedness measure, damage demand, nodal connectivity, and distance from sources, clustering hierarchy shown in Fig. 8.8b was obtained. A search through this hierarchy suggests a high vulnerability index for the loss of segments p121 and p122.



8.8 (a) An example water pipe network (adapted from EPA (2000)) and (b) its hierarchical representation.

8.5.2 Transport networks

A road network can be represented as a directed, weighted graph. There is traffic demand between any two nodes that drives the flow of traffic along links in some way. Usually the shortest paths between any pair of nodes or the paths with the shortest travel times are identified. However, real networks are far more complex and the consequences of the failure of one or more links need to be analysed with respect to the whole system. It is not uncommon to see ‘knock-on’ effects of the failure of a major road on many neighbouring roads at least in the short term (Goodwin *et al.* 1998; Clegg 2007).

Application of vulnerability concepts to road network (Agarwal *et al.* 2011) is based on a measure of transmittance which depends on capacity speed and length, and orientation is used to calculate the well-formedness. The clustering process starts with the identification of loops with the best well-formedness and higher-level clusters are formed by including the neighbouring loops so as to increase the well-formedness of the cluster. If there is no increase in the well-formedness, a new loop is started and the process continued until the whole network is one cluster. A search through the clustering hierarchy results in the identification of different vulnerable scenarios which can be used for managing risks.

8.6 Advantages of vulnerability analysis

Vulnerability analysis, as presented here, can be used to analyse risks to lifelines and to plan for recovery following an earthquake. An analysis of chance is one part of risk assessment. For low-probability high-consequence events, numerical results based on traditional probability have to be set in a broader picture (van Breugel 1997). The Kobe earthquake in 1995 has brought a qualitative change in the way risk analysis should be done and used. Probability of failure alone is not sufficient to exclude a failure scenario – we must also consider the consequences of such failure scenarios. Vulnerability analysis provides an approach to assess the form of a system so that disproportionate consequences could be minimized.

It has its own place amongst the popular quantitative techniques used for the analysis of failures. For example, cut set analysis gives only those scenarios that are obtained as total failure scenarios in vulnerability analysis. Fault tree analysis starts with a predefined failure whereas vulnerability analysis aims to explore unforeseen but catastrophic failure scenarios. In vulnerability analysis, the effect of a deterioration event becomes apparent through a clustering and unzipping procedure rather than by traversing all the paths as in event tree analysis. In structural reliability analysis, it is necessary to have a system model of loading. In the vulnerability approach,

one starts with vulnerable failure scenarios and looks for the probability of actions that will cause those failure scenarios. Ultimately, both methods lead to an estimation of risk of failure but the latter has the potential to avoid catastrophic failures due to actions that would have otherwise been ignored.

With increasing complexity and interdependence of infrastructures, analysis of risk for any one utility (e.g. water supply system or energy supply network) can no longer be done in isolation. Damage to one infrastructure has the potential to affect other infrastructures. During an earthquake many infrastructures are affected at the same time and generally each one of these at several locations. Thus approaches to analyse the consequences of a hazard irrespective of their chance will be increasingly used.

8.7 Conclusions

Vulnerability analysis is concerned with the form of the system which leads to an identification of vulnerable failure scenarios. Some of these failure scenarios may have the potential to cause disproportionate failure due to one or more actions. Each possible action may contribute to the realization of different failure scenarios. In the approach described here, an assessment of the likelihood of a failure event is initially kept independent of the hazards and then combined with the assessment of the hazards. The consequence of a failure scenario and the support for this scenario give a measure of the risk it poses to the system. Risks due to different failure scenarios can be compared and appropriate measures taken to reduce the risks to an acceptable level. Some failure scenarios may be managed better by improving the form while others might require appropriate measures to limit the likelihood of actions.

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Earthquake risk management of civil infrastructure: integrating soft and hard risks

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Abstract: Risk is an inevitable part of all human activity. Similarly sized earthquakes can have very different impacts in different countries depending on the degree of engineering input into the design and construction of the facilities. In this chapter we will propose an approach based on systems thinking and new systems boundaries. We will identify and characterise three different sources of uncertainty: hard physical system parameter uncertainty, hard system model uncertainty and soft system human uncertainty. We will explore ways in which evidence from previously disparate sources can be managed in an integrated way.

Key words: seismic risk, uncertainty, safety, systems thinking, integration, hard and soft systems.

9.1 Introduction: the inevitability of risk

Risk is an inevitable part of all human activity. It is at the heart of all major engineering questions. How do we know what is safe? How safe is safe enough? Are any specific structures, or groups of structures, particularly vulnerable? What are the chances of another Chernobyl disaster? How do we deal with extreme weather such as hurricanes and flooding and, in particular for this volume, earthquakes? How do we learn from failure so history does not keep repeating itself?

We know from everyday life that things do not always turn out as we want and this is also true in engineering practice. Both everyday life and engineering practice require us to use common sense when solving problems. We have to go through specific processes to get from where-we-are-now to where-we-want-to-be at some time in the future. In doing so we have to decide what we want, what we think we know, how we may achieve what we want, what actually to do and finally what we think might be the consequences. We know if things do not work out then the consequences may range from minor upset to deep and major harm. Quite rightly, what engineers do and the processes they go through are closely scrutinised and can, ultimately, be tested in a court of law as a duty of care.

There is a much greater awareness of the need to deal explicitly with risk in all aspects of life. In many projects, accountants manage the known financial risks well, engineers manage the known technological risks well,

safety specialists manage the known health and safety risks well and quality managers manage the known and recognised processes well. Nevertheless within specialisms a range of techniques may be used to assess different aspects of risk which are difficult to integrate. For example, structural engineers not only have new powerful computational tools but there are also many new and different challenges such as terrorist attack and the impact of climate change. The net result is that even in successful projects and companies, major problems seem to arise in the gaps between specialisms resulting in unknown and unintended complications, such as cost and time overruns and consequent quality problems. Sometimes even processes for which no one is directly responsible happen and hence are not properly controlled. Then there are surprises from previously unknown, unexpected and unforeseen events.

In this chapter we will address the problem of how we can better facilitate well-balanced decisions to manage all risks to minimise unintended harmful consequences. Risk is particularly pertinent to people living in active seismic zones. At all too frequent intervals, TV pictures of earthquake damage to buildings, bridges and other infrastructures are beamed around the world. Similarly sized earthquakes can have very different impacts in different countries depending on the degree of engineering input into the design and construction of the facilities. For example, in 2010 similarly sized earthquakes in Haiti and New Zealand produced very different consequences. However in early 2011 Christchurch in New Zealand was hit by a less severe earthquake in terms of released energy but one that caused more loss of life and damage. In March 2011 a large earthquake, consequent tsunami and damage to a nuclear power station, caused considerable loss of life and damage to large parts of the infrastructure of one of the most prepared nations in the world: Japan. It is crucial therefore to identify structures, groups of structural types and whole regions or countries where vulnerability to earthquakes is unacceptably high.

A key difficulty is how decision makers integrate information from many disparate sources, including different worldviews with different theoretical or calculational models, to manage risks. Risks are also communicated to a wide variety of people in many different and potentially confusing ways. For example rather vague descriptions such as moderate, high or very high may refer to the return period or the level of damage, whereas evidently statistical descriptors, such as 1 in 100 years, may leave us unclear about the level of damage.

9.2 Managing technical risks to structures

Advice to practical engineers about the risks to structures in seismic zones usually focuses on five technical areas (e.g. Booth and Key, 2006; Gibbs,

2003a, b), structural layout and form, ductility, stiffness, materials and foundations.

9.2.1 Layout and form

Experience of the performance of buildings in past earthquakes tells us that certain common-sense ideas are important. For example the elements of a structure should be well tied together with well-defined continuous load paths with no irregularities. Buildings that are symmetrical perform better than those that are unsymmetrical. Weak storeys, perhaps required by architects for open floor areas in a building, are a very common reason for collapses. Buildings where the centre of mass and stiffness are not coincident do not perform well. Plan shapes should be compact. Extensions are prone to vibrate separately so wide separations are required (~50 mm per storey) to allow parts to vibrate separately. Likewise adjacent buildings must be adequately separated. The distribution of mass within a building is important so that high elevations of heavy plant and machinery at the top of a building should be avoided. Structures should be structurally redundant so that there is back up if a member fails but this has to be done in a manner that recognises potential vulnerabilities of form (Agarwal, 2013). Moment resisting frames with rigid beam column connections can provide good ductility and redundancy but if poorly designed (e.g. with weak storeys) can fail catastrophically. Particular attention needs to be paid to the high stress concentrations around joints. Steel welded joints can be vulnerable and concrete joints often have congested reinforcement which requires careful steel fixing skills and good concreting to ensure proper compaction of the concrete. Shear walls provide strength and stiffness at low cost and are a good solution for buildings up to 20 storeys but in taller buildings will probably need to be combined with other techniques to get sufficient overall stability. Special systems to manage the performance of a building during an earthquake include (a) passive devices to change the period of a structure or to dampen vibrations, (b) seismic isolation such as rubber bearings, and (c) active computer control systems to modify behaviour during an event.

9.2.2 Ductility

Structures that are brittle fail suddenly – something that no one wants. Ductile structures absorb energy so that they fail more gradually. Given the huge uncertainties both in predicting earthquakes and in calculating response, then the provision of ductility is one of the most effective ways of preventing loss of life due to earthquakes. Of course, this implies that in an extreme earthquake some damage to the structure may be inevitable.

One of the most common methods for achieving desirable ductility is through capacity design where strong columns are used with weaker ductile beams. The columns need to have sufficient ductility but could buckle in a brittle manner – capacity design is used to ensure that the possibly brittle elements are strong enough to withstand the full capacity of the ductile elements. Suitable detailing is important to ensure that a structure designed to yield can achieve the large strains required.

9.2.3 Stiffness

Although a structure needs to be ductile, the deflections must be limited to reduce damage to non-structural elements, such as false ceilings, wall partitions, cladding and internal furniture (heavy book cases can cause injury). This means that the structure must be adequately stiff with due consideration given to the possibility of $P-\Delta$ effects in which large deflections induce extra bending in columns.

9.2.4 Foundations

General rules for foundations are difficult because local circumstances vary. A major issue is to ensure that foundations can take different forces that may be imposed during an earthquake. For example, some structures may induce uplift forces, piles may need to take lateral forces, piles may be unsupported over a long length and may buckle if soil liquefaction happens (Bhattacharya and Madabhushi, 2008). Raking piles add to the lateral stiffness of pile groups but have been found, by experience, to be prone to failure in an earthquake.

9.2.5 Materials

Steel has a high strength-to-mass ratio and is ductile. However, buckling behaviour is not ductile and connections can be problematic. Timber is strong and light. Concrete beams and columns can be brittle so proper design and detailing are required using capacity design and adequate confinement reinforcement. Masonry is not ductile and so must be designed for large reserves of strength. An early report on the damage due to the 2011 Christchurch earthquake (Ingham *et al.*, 2011) showed that many of the damaged buildings were of unreinforced masonry.

9.3 Reliability theory for the analysis of uncertainty and risk

Managing the practical risks has to be contrasted with theoretical approaches. Reliability theory is a mathematical framework for the analysis of uncer-

tainty and risk in engineering. It is used in practical performance-based engineering, PBE (Ellingwood, 2008). Unfortunately, reliability theory is partial, since it covers only the same uncertainties as simple safety factors – albeit more coherently and rigorously – but omits many of the risks that have been found to result in actual failures. It also does not directly address any of the practical factors described in the previous section. The idea of PBE is wider than reliability theory – it is ‘to provide owners and designers with the capability to select alternative performance goals for the design of different buildings’ (Hamburger, 1996). Ellingwood (2008) writes that PBE is ‘evolving to enable new building technologies . . . to better meet heightened public expectations’. The aim is to extend consideration and hence to cover all risks. PBE as currently formulated in PEER (2010) does enable a better consideration of the levels of expected performance under earthquake loading, ranging from ‘minor damage’ to ‘incipient collapse’, and the idea that essential buildings such as hospitals remain functional even under an extreme event. However, it still is inadequate for an integrated treatment of risk. The problem is that PBE excludes many of the important processes for managing engineering risks. As Hamburger (1996) states, ‘perhaps the most significant barrier to the adoption of performance-based earthquake resistant design is the lack of control exercised by structural engineers over the building design process’. He pointed out that the role of the structural engineer is typically limited to designing and detailing structural components of a building. However, the problem is even deeper than that as we shall see.

Within the limited paradigm of PBE, the role of reliability theory is even more limited. We can illustrate this with an example. Seismic fragility is defined as the probability of occurrence of a stated level of damage due to a given intensity of ground motion. This is apparently a sufficient statement of a technical problem – that of capturing the chance of a level of damage from a given seismic intensity. Unfortunately, there are two important qualifications that derive from restrictions that are axiomatic to the theory that make it insufficient as a general treatment of seismic risk. The first restriction is that it is only possible to represent important uncertainty in the quality of the theoretical model of the physical system by a series of random parameters. This is simply too crude. Not only may the scientific model of the behaviour of the structural components be lacking under the effects of an earthquake but also the vulnerabilities and subtleties of structural form are too important to be treated so simply. For example the use of non-structural partial in-fill panels in a frame building can change the response characteristics of a structure. Also as Agarwal and England (2008) and Nafday (2010) pointed out lack of structural robustness is a property of the form of a structure where the ‘well-formedness’ or conditioning of connectedness is critical. Ghosn *et al.* (2010) have discussed the complexities

of dealing with structural redundancies. These examples show that it is simply not good enough to equate, and hence model, all uncertainties as a variability of a number of random variables. Uncertainty is more than mere random variability.

The second restriction is that all of the risks due to soft issues (such as human error and organisational problems which we will discuss in detail later) are simply not included. FEMA (2009, 2011) and PEER (2010) define systems that are entirely technical and seem not to acknowledge that human and organisational factors have any part to play in why infrastructure systems fail in an earthquake. It could be argued that this may not matter if these softer uncertainties are independent of the technical uncertainties, because they are dealt with elsewhere – by project management and other decision makers. Unfortunately, many of the important hard and soft risks are not independent. Many studies (Blockley, 1980; Reason, 1990; Turner and Pidgeon 1998) have shown that many risks lie at the interface between the technical and human and do directly influence technical outcomes and the total risk. A report by IAEA (2001) shows that operational and financial performance is closely related to safety. The report calls for an integrated approach. One only has to compare the seismic performance of buildings in Haiti with that in more developed countries to understand that political, social and human factors are crucially important in managing all risks to infrastructure. This means that current systems boundary in PBE must be widened.

Within reliability theory and PBE, fragility is the conditional probability that the structural capacity C in a stated limit state fails to resist the structural demand D given the seismic intensity SI .

$$p(C < D/SI = x) \quad [9.1]$$

The three performance levels used in PBE (Ellingwood *et al.*, 2004) are immediate occupancy (*IO*), life safety (*LS*), and collapse prevention (*CP*). Immediate occupancy is the limit below which the structure can be occupied safely without significant repair. Life safety is a level above which there is *significant* damage but a substantial margin remains against collapse. Collapse prevention is the level above which either the strength of structural members or connections is lost or there are *significant* permanent deformations.

The problems of partiality become apparent as we unpack the sources of uncertainty in this formulation. Some authors use the word aleatoric to describe the uncertainties due to randomness. The word aleatoric means ‘dependent on chance or accidental events or other contingencies’. It is unclear how this is different from randomness in a stochastic process and so in this chapter we will consider aleatory, random and stochastic processes as equivalent. The other source of uncertainty is often described as epis-

temic. The word epistemic means ‘pertaining to knowledge or the conditions for acquiring it’. Again it is unclear why such a term is useful since at a philosophical level all uncertainties are due to limitations in what we know or the conditions for acquiring or understanding. In this sense all uncertainty is knowledge-based. For example, random uncertainty is epistemic since randomness is the lack of pattern or specific order in data and must therefore depend on our understanding and consequent models of what we mean by order. Some (but not all) of the epistemic uncertainties seem to arise from assumptions made in the modelling of a system, the methods of analysis and limitations of data. They may be reduced by using more resources to perform more detailed analysis and obtaining more data but there will always be a residual uncertainty because we can never have certainty – there is always a gap between what we think we know and what we do, resulting in the possibility of unintended and unwanted consequences. It is this residual uncertainty which is the real challenge to risk managers since it can sometimes lead to unintended, unforeseen, and unwanted disastrous consequences, such as 9/11, Chernobyl, the near collapse of the world’s banking system in 2008 and the damage to nuclear reactors by the tsunami after the 2011 earthquake in Japan. Social scientists, such as Turner and Pidgeon (1998), have argued that the preconditions to major disasters can incubate or develop in a way so that it may be possible in some instances to identify before a final disastrous event. They argue that we need to develop methods for identifying those preconditions with sufficient dependability to make such politically difficult and potentially expensive decisions to avoid the even greater costs and consequences of a disaster. This is a point to which we will return later.

9.4 Seismic vulnerability

Before we continue to unpack the sources of uncertainty in managing seismic risks and how we might develop a general theory to deal with them, let us look at another technical approach to the problem of evaluating the vulnerability of our urban heritage to earthquakes. Vulnerability is ‘susceptibility to harm’ and, like risk, is an idea central to the planning of retrofits (Bernardini and Lagomarsino, 2008). The focus of vulnerability analysis is the extent to which the form of a structure increases the risk of damage when exposed to a hazard – something only partially considered in reliability theory (Agarwal, 2013; Agarwal *et al.*, 2001, 2003; Nafday, 2010). A hazard is defined as a ‘state of affairs’ with a potential for harm and the dominant one, as far as we are concerned in this chapter, is an earthquake.

Monumental buildings, such as churches and cathedrals in Italy, are embedded in a complex of classical urban environment of traditional

buildings in narrow streets mixed with modern developments. Bernadini and Lagomarsino (2008) classified such buildings as: palaces, churches, monasteries/convents, mosques, towers, obelisks, theatres, castles, triumphal arches, and arch bridges. Clearly such classifications are imprecise so that the task of dealing with these structures does not (as they pointed out) readily fit within the precise scope of reliability theory. The structure of a palace, church or monastery may indeed be quite similar, because each does not 'know' it is what it is supposed to be. As far as 'mother nature' is concerned, they are buildings that may be quite similar except in the way they are adapted for a particular human purpose. Another obvious reason for the lack of fit into a precise theory is the sheer paucity of data and dependable models of any single structure. Statistical data from damage assessments after earthquakes are available and contribute to planning at regional and national levels. However, any predictions from such data concerning individual buildings will be at best approximate. Bernardini and Lagomarsino (2008) therefore have suggested a methodology based on the European macroseismic scale (EMS) in which the classes of vulnerability summarise the observed damage to types of building subjected to past earthquakes.

However estimations of the local intensity of an earthquake from recorded damage are clearly affected by the building type. The EMS 98 (1998) scale used by Bernadini and Lagomarsino (2008) defines the overall damage to buildings, both masonry and reinforced concrete (RC) buildings, using a scale of six qualitative judgements taking into account the severity of damage to structural and non-structural elements. Grade 1 damage is defined as negligible to slight, grade 2 is moderate, grade 3 is substantial to heavy, grade 4 is very heavy and grade 5 is destruction. They then used qualitative rules to summarise the vulnerability of a building not by the type but by its membership in a defined class of vulnerability ranging from A (the most vulnerable) to F (the least vulnerable) to cover the full range of the expected behaviours from traditional non-engineered structures to structures designed for seismic protection. These behaviours are expressed as probabilities of different degrees of damage (six levels) for different values of macroseismic intensity (four values) in damage probability matrices (DPM).

Since these definitions are not precise, Bernardini and Lagomarsino (2008) derived imprecise interval probabilities to estimate bounded intervals of particular functions of the damage. They recognised that the assumption in classical probability theory – that every member of the total sample space is known and can be allocated a probability measure – needs to be relaxed. However, they also note that the widespread distribution of instruments providing direct records of the strong motions due to earthquakes may seem to have reduced the significance of this kind of macroseismic scale and observational approach. They recognised that the HAZUS pro-

cedure developed by FEMA (2011) is a simplified structural mechanics approach, based on fragility curves for archetypical structures. They acknowledged its use to evaluate the probability of reaching or exceeding specific damage limit states from a given ground motion for a variety of building types, such as wooden light frame, steel moment frame, concrete shear wall, and unreinforced masonry. But then they knew that fragility curves well fitted to the seismic behaviour of idealistic types of modern construction types cannot easily be extended to a more general context – especially in Italy (or more generally in Europe and other continents). They agreed that a correlation between structural and macroseismic approaches may be possible and could be used for reciprocal calibration of the models and the calculation of vulnerability curves. However, this can only be done if all of the known uncertainties are considered together with recognition that there may be uncertainties which, as history tells us, as yet are not known to us. This simple recognition inevitably leads us to a new approach as we shall describe later.

Bernardini and Lagomarsino (2008) defined a mean damage grade using two parameters, the vulnerability index V and the ductility index Q . Clearly, a vulnerability index assigned to a monument by a typological classification represents an average value, which does not take into account the distinctiveness of a single building and does not allow the most vulnerable structures among buildings of the same type to be singled out. To refine the vulnerability assessment, a survey is required to collect details of other parameters, such as state of maintenance, quality of materials, structural regularity (in plan and in elevation), size and slenderness of relevant structural elements, interaction with adjacent structures, presence of retrofitting interventions and site morphology. These are all attributes that often cannot be defined with the level of precision required for a reliability calculation.

This kind of approach to the vulnerability of the form of a structure has to be compared with methodologies which attempt to model any susceptibility to cascading failures (Agarwal and England, 2008; England *et al.*, 2007; Starossek, 2007; Tesfamariam and Saatcioglu, 2010). Here vulnerability analysis is aimed at identifying situations where a relatively small amount of damage to one part of a structure can lead to the structure ‘unzipping’ to total, or near total, collapse. This type of analysis is important, because standard reliability theory cannot capture the uncertainties and complexities in the structural model of the form and behaviour of structures, such as the effects of adjacent buildings, mezzanines, weak and soft storeys, geometrical irregularities, vertical discontinuities as well as many other factors. Structures have also to respond to unknown unexpected seismic effects (Blockley and Godfrey 2000; Nafday, 2010) which by definition cannot be included in a probabilistic sample space – even with Bayesian updating.

9.5 Uncertainty: fuzziness, incompleteness and randomness (FIR)

In order to reveal the uncertainties within the theories of reliability and seismic vulnerability as described in the previous sections, then we need to consider the classification of uncertainty very carefully. It is not sufficient to state that all uncertainty can be modelled as randomness. In previous work, the author has classified uncertainty with three *orthogonal* characteristics FIR – fuzziness, incompleteness and randomness. It was argued that other characteristics of uncertainty, such as ambiguity, ambivalence, indeterminacy, unpredictability, and conflict emerge from interactions among these basic three attributes (Blockley, 2010).

Fuzziness is imprecision or vagueness of definition. It is implicit in assumptions about levels of performance, such as *IO*, *LS* and *CP*. It is explicit in the treatment of seismic vulnerability by Bernardini and Lagomarsino (2008), as their analysis is based on the use of fuzzy and random sets. Randomness, as discussed earlier is the lack of a specific pattern in some data and is explicit in probability and reliability theory.

Incompleteness refers to things we do not know – it is perhaps the most neglected characteristics of uncertainty. Some theorists even deny it exists. It is not recognised in the axioms of classical probability theory, because the probabilities of events or statement in the sample space must sum to unity, i.e. everything in the sample space is totally known and identified. This requirement of totality is dropped in fuzzy sets and its derivative, random sets, as used by Bernardini (2005). However, the axiom that replaces it is also restrictive, since it requires total dependency between sets – in effect they are nested (Blockley, 1985). Unfortunately, although incompleteness is explicit in random set theory, it is not sufficiently stressed as a major source of uncertainty. There is a need, in any methodology, explicitly to decouple measures for and against the truth or dependability of an event or logical statement and hence to allow a state of ‘don’t know’. Interval probability theory, interpreted as Italian flags, as described by Blockley and Godfrey (2000) and Blockley (2008) was designed to facilitate a direct handling of incompleteness.

9.6 Systems thinking

Systems thinking (Blockley and Godfrey, 2000) has been used to identify and characterise the three different sources of FIR in risk management previously classified as random parameter, system model and human (Blockley, 1980). They are *hard system* parameter uncertainty, *hard system* model uncertainty, and *soft system* uncertainty. Our purpose will be to argue not just for a need to integrate theories of reliability and vulnerability for

improved risk management but also to integrate all evidence from whatever sources, i.e. hard physical risks with soft 'people' risks.

One of the first questions addressed by systems analysts is where to draw the boundary of a system. A vivid example of this was the Japanese earthquake in March 2011 with a consequent tsunami that inflicted enormous loss of life and damage to infrastructure including fire and explosions with some release of radioactivity from a damaged nuclear power station. One cannot draw a system boundary on the grounds of expediency or available expertise. In practice, one must deal appropriately with the totality of the purpose of a system. It is not acceptable to constrain it arbitrarily as so often can be the case in scientific research – where selective inattention may be required to make the problem tractable (Blockley and Godfrey, 2000). Such luxury is not available to practitioners – seismic risk management of structures must include human and organisational issues, since all hard systems are embedded in soft systems. The consequences of the Japanese earthquake are profoundly social and financial. Of course it can be argued that soft system risks are indeed handled by good project management. Nevertheless, hard system and soft system risks are not independent, and this interdependence has to be part of a general treatment of risk management. For example, an EEFIT investigation into the 1994 Northridge earthquake in California reported that non-structural elements may change the structural response of a building and reduce the ductility capacity. The possible buckling of piles in a liquefied soil (as mentioned earlier) is not currently included in the relevant codes of practice. Baker and Cornell (2006) reported that two definitions of spectral acceleration have been commonly used by analysts and that the distinction between the definitions is not always made clear. Earth scientists typically use the geometric mean of the spectral accelerations of the two horizontal components of ground motion as the intensity measure for *hazard* analysis, while structural engineers often use spectral acceleration of a single horizontal component as the intensity measure for *response* analysis. They state that because of this, a systematic error has been introduced into the results from many risk analyses, typically resulting in unconservative conclusions. They quote results for an example site and structure located in Los Angeles where an error resulted in a 12% underestimation of the spectral acceleration value corresponding to a 2% probability of exceedance in 50 years, and a 10% underestimation of the structure's maximum interstorey drift ratio corresponding to a 2% probability of exceedance in 50 years. However, the authors do not refer to the fact that many actual earthquakes have vertical components of acceleration as well – something that practising engineers cannot ignore in their risk estimates.

We have to widen our systems boundaries from the narrowly technical considerations that dominate engineering theory to encompass all of the

issues addressed by practitioners. We have to include the realities of human and organisational issues and relationships with wider society, which are utterly important in engineering practice and dominate the causes of engineering failures, accidents and disasters (Blockley, 1980, 2010; Reason, 1990; Turner and Pidgeon, 1998). Grundy (2011) has proposed a formal introduction of a disaster limit state which goes beyond an ultimate limit state to consider risk reduction strategies that are not normally considered to be engineering but a wider part of disaster management.

We will use the term *hard system* for the inanimate physical things, such as beams, columns, bridges, dams and reservoirs. We will use the term *soft system* where people and social systems are involved. Hard systems are physical systems that are commonly said to be ‘objective’ in that they are supposed to be independent of the observer and hence the same for all of us. Reliability analyses are almost always technical and only include uncertainties that can be modelled as random or stochastic variations in *parameters* of demand and capacity or load and strength of hard systems – we will call that *hard system parameter uncertainty*. As such, they are sophisticated and more coherent versions of much simpler safety factors.

Analyses of behaviour and performance are based on the physical science of engineering including models that are tested in laboratory under precisely defined conditions (e.g. knife edge supports) with precisely known parameters (load and material properties). They are also tested in practice through the successful performance of the millions of applications that give us confidence that they are dependable. Unfortunately the conditions under which these structures in practice operate are rarely precisely known and they are rarely required to perform at levels close to their limit states. Nevertheless, as long as we are aware of the contexts in which these models work, they are very dependable and the basis for many successful structures. Of course failures (which are fortunately rare) are important learning points, since we know then that our methods have been demonstrably wrong or falsified. Any differences between theoretical and actual behaviour, even when we know the parameters quite precisely, contribute to *hard systems model uncertainty*. These kinds of uncertainties relate to any differences in behaviour between theoretical predictions and actual behaviour. Laboratory tests reveal some (usually small) differences even though test conditions and parameters are precisely controlled. However, the major uncertainties arise due to differences in modelling assumptions and the structure as-built. For example, we may assume simple supports but in practice the supports may be designed and built such that they carry some bending moments (for example, welded end plates on steel beams). Unfortunately measurements of actual ‘as-built’ structural behaviour in operation are rare, although developments in better and cheaper instrumentation systems are leading to more tests and studies on full-scale structures. The

real uncertainties become manifest, when structures fail and enquiries reveal inadequacies. For example, inadequacy of our understanding of the behaviour of steel box girder bridges was demonstrated by several failures in the 1970s (Blockley, 1980).

Attempts to estimate hard systems model uncertainty, using one or more stochastic parameters or extra random variables as in fragility analysis, are inadequate for all but the simplest of cases. One or more random parameters cannot capture the uncertainties in qualitatively different models or modes of behaviour. We know that the underlying reasons for the majority of failures are complex. For example human beings make errors – departures from intended practice for all sorts of reasons. These and other social, economic and political factors (including corruption) contribute to significant *soft system uncertainty*. Soft systems are, as the name implies, systems which are hard to define – the edges are unclear. Generally soft systems are governed by the behaviour of people, which is so complex as to be hard to define. The consequence is that the emphasis in soft systems is not on prediction but rather on managing a process to achieve desired outcomes.

Traditionally, predicting and managing risks in hard and soft systems have been treated quite separately – the former being a matter of engineering physical science and the latter of engineering management and disaster risk reduction. Management techniques, such as quality assurance, value management, lean construction and business re-engineering, are promoted quite independently of hard systems risk and reliability theories. However, all of this information and all of these ideas and techniques need to be brought together and integrated because they are interdependent. They all contribute to the total but are all partial and some may even conflict. We need to explore ways in which evidence from these previously disparate sources can be managed together for the greater good of the whole system.

One way of achieving this kind of integration is through system optimisation. Unfortunately such procedures are also bound to be partial and can also lead to structures that are vulnerable in unintended ways. A simple example is the design of a steel I-beam in bending. If one adopts a partial approach that considers only beam bending, then one will conclude that the deeper the beam the better. Of course in that context such a conclusion is sound. However, if one widens the context and considers the possibility of lateral torsional buckling then matters become quite different, because longitudinal and torsional slenderness become important and what was previously an optimum solution – a very deep beam – may become a failure due to lateral-torsional buckling of a slender flange. Another example is the relationship between the weight of a structure and its cost. If weight is minimised to give an optimum solution, it is quite possible that the costs of building the structure will rise, because more cutting and more complex jointing is required. Optimising a design may result in good solutions in one

specific context but sub-optimal in others. It is possible that a structure may become vulnerable (as in the case of an I-beam) and further a highly interconnected system or structure may become prone to unzipping or cascading failure. Of course one can argue that we simply require an optimum over all possible contexts – but because we now know that complex systems contain unknown unknowns and hence future unknown contexts in which an optimum may fail dramatically. We need to aim for robustness rather than optimisation.

Integrating disparate sources of information to improve the management of risk can also bring direct positive benefits for opportunities to gain synergy, robustness and resilience. Grundy (2011) has described how a headmaster in Japan successfully took special measures to retrofit his school building and to prepare, educate and rehearse his children and staff. Robustness is strength in all possible modes of failure of a system including the people. Robust systems are sturdy and hardy; they endure and are tolerant of damage. A system is robust if it resists damage and, if damaged, resists the propagation of that damage. Progressive collapse or unzipping occurs when, after some initial damage, further damage progresses, without extra inputs, through many limiting conditions and damage states of increasing scope, breadth and consequences. Vulnerability is susceptibility to initial damage. It is the ease with which damage propagates. A vulnerable system may suffer progressive collapse where small damage leads to disproportionate levels of damage. Vulnerability is sufficient but not necessary for lack of robustness. For example, lack of robustness is also caused by undue sensitivity to details such as design assumptions, the accuracy of manufacturing or construction methods, the quality of details such as connections and possible perturbations such as ground movements. Resilience is the ability to recover from damage. It is being capable of returning to an original state after change. For example, linear elastic strain energy is resilient because it is recoverable – there is no permanent change. Hollnagel *et al.* (2006) have proposed the idea of ‘resilience engineering’ to provide a new paradigm for considering the totality of requirements for a system and to capture a way of thinking about safety which enhances the ability at all levels of organisations to be robust yet flexible and to use resources proactively to manage processes to success. They distinguished between resilience and adaptability, which is a capability of being able to adjust to different conditions and still be fit for purpose. Systems that are highly interconnected may not be robust, resilient, or adaptable. Notice that these definitions do not involve risk. The 2011 earthquake in Japan illustrates vividly the interconnections between seismic risk, risks to individual structures and parts of the infrastructure, tsunamis and consequent damage to whole regional areas, damage to nuclear power plants and radiation leaks, social consequences to the spirit of a whole nation, and impact on the nation’s economy and possible impacts

on the future safe development of nuclear power world wide and its role in dealing with climate change.

It is apparent that there are many ways in which these desirable characteristics may be delivered using a variety of theories and techniques. What is needed in a practical engineering project is a way of bringing the results from many considerations together to form an integrated body of evidence upon which to make a decision. In order to do this we need to rethink what we do. We need to move from a purely reductionist scientific view of engineering to one in which reductionism is part of a wider systems thinking holistic view. Three ideas are basic to this change and have been described in some detail (Blockley and Godfrey, 2000). They are: (1) to see every part of a system as both a part and a whole, (2) to focus on emergent properties arising from connectivity, and (3) to see every part of the system as a process. Here we will highlight the role of process (Blockley, 2010). Processes are the way things behave in hard systems and what people do in soft systems. All designed hard systems have a function which is a role in a process. For example, a beam in a structure has the function of carrying loads from floor slab. A dam has the function of holding back reservoir water. The steel and concrete of which the beam and the dam are made do not 'know' they have that function – they have no intentionality. The function is ascribed to a hard system by the people who own it, conceive it, design it, build it and use it. We are also the ones who decide when the hard system has failed according to our criteria of failure. Clearly some functions are obvious – others are less clear and unintended. For example, a bridge designed to carry road traffic was almost certainly not designed to be used as a shelter by homeless people. In one case the repair cost of concrete damaged by the fires lit by homeless people to keep warm under a bridge, was substantial. In summary, all designed hard system processes are embedded in one or more soft system processes.

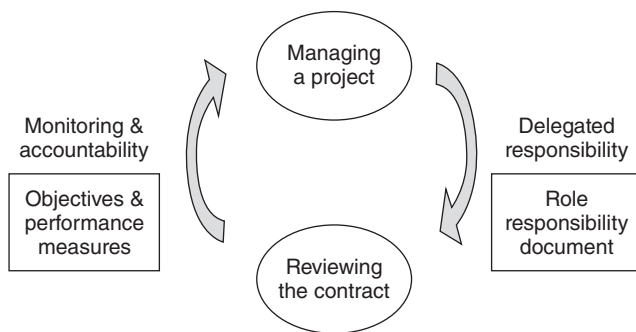
9.7 Process models and project progress maps (PPM)

We will use a process model as the integrating structure on which everything else is built (Blockley and Godfrey, 2000). Frameworks and tools such as IAEA (2001) are, at root, decision processes that identify options, consider alternatives, choose from available options and implement/monitor solutions. There are many views of what constitutes a process. Most people use the word to distinguish the form of the 'doing of an activity' from the content which is the output (e.g. a product). Here we use a new way of thinking about process which does not separate form and content (Blockley, 2010). This concept of what constitutes a 'new' process (to distinguish it from more traditional views of process) is much richer than an input transformed into an output, a recipe, a Gantt chart, a network or a flow chart,

although it can be simplified down to each of these if required. In this methodology, each process is seen as a holon, i.e. it is both a whole and a part at one and the same time. The relationships between processes are captured in a process map and then each process is used as a ‘peg’ on which we can attach all other data and information. The process map therefore sets out the basic structure of a project and all of the data associated with it. The traditional separation of process from product is changed, because by this view a process can be the ‘doing of an activity’ or the ‘performing of a physical system’ – the structuring of the information is the same for both. Of course a product is the output of a process and it is useful to keep the distinction because it defines what the client perceives he/she is buying. However, it is important to understand, from this wider perspective, that the client is paying for all of the processes and because products do things and exist through time, then products are also processes.

In order to use this idea, the map of the interconnections has to show how processes relate to each other. This is the purpose of a project progress map (PPM). It enables us to integrate various data, such as risk registers, structural calculations including finite element analysis, and project progress measures. As stated above, the processes in the PPM form a central spine or skeletal structure on which we attach all data and attributes.

The process holons are arranged hierarchically and vertically in layers of definition – but this must not be mistaken for rigid ordering of power or subordination as would be typical of a military or oligarchic organisation. It is, however, an ordering of scope and responsibility (Fig. 9.1) that have to be addressed even in the most heterarchical or responsibly autonomous organisations and societies – from the processes with widest scope at the top to the processes with the narrowest scope at the bottom. We start by describing the whole system as one process – this is the ‘top’ process and the top layer. Obviously to achieve success of this top process many sub-



9.1 Responsibility and accountability between layers of a process model.

processes have to be successful and these are the second layer. Success in all of the processes of the lower layer should be jointly necessary and sufficient for success of the layer above. Then for the sub-processes to be successful some sub-sub-processes must be jointly necessary and sufficient for the success of the sub-process. This continues down through each layer to a level of detail as appropriate for the particular system. Horizontal interactions between processes at the same layer can be modelled separately and as needed using appropriate tools and techniques. For example, time relationships can be included using a standard critical path network analysis; structural calculations can be performed using finite element packages; and risk registers can be kept using proprietary software. These horizontal relationships between processes are constraints on the attributes of each process.

The attributes of each process have to be monitored to keep it on a path to eventual success. Success is defined by a set of stated objectives that may well change. Every process should have a process owner, i.e. a person who is responsible for steering that particular process towards success. Parameters and performance indicators of the process are monitored to keep them within required bounds and to provide evidence of trends that need remedial action. Thus at any given time during a process, we have evidence from past and present performance and we can use predictive analyses about the future. The words 'who, what, where, when, why and how' are formally used to capture and record information for appropriate sharing with other authorised people. In brief, question 'why' is the potential that drives the flow of change in 'who, what, where, when' through transformations 'how' (Blockley, 2010). In this way, each and every process is steered to success based on up-to-date integrated information.

9.8 Measuring evidence of performance

A key idea of the methodology is that we want to find evidence that any given process is moving towards success and there is no build up of difficulties that might bring about failure. We also want to locate opportunities for improvement. The evidence will come from many sources of various types, and there is a need to collect it, digest it, interpret it, learn from it and make decisions using it for example by developing systems, such as MARIUN (Blockley and Godfrey, 2000). MARIUN incorporates not only technical hard system evidence such as provided by PBE and other techniques but also monitors soft system evidence about human and organisational issues.

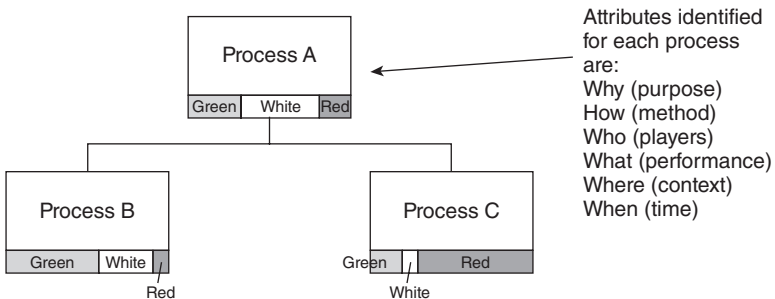
In order to have a measure of evidence that can be used in soft as well as hard problems, we need to decouple the evidence in favour and the evidence against the proposition that a process is heading for success because they may well be in conflict. Of course where there is significant

conflict, we need to identify it and manage it. We must use traditional measures of the physical sciences, for example as state variables, as appropriate but also incorporate ‘soft’ measures (Blockley and Godfrey, 2005) such as voting or individual judgements. We can do this by capturing on a scale [0, 1] the degree of any evidence in favour as well as evidence against. We colour the degree of evidence that a process is heading for success in green as shown in Fig. 9.2. Evidence against is also assessed on a scale [0, 1] and is coloured in red starting from 1 and working back to zero. The difference in the middle is white and represents incompleteness – the extent to which we do not know. The three colours together make the Italian flag.

An all-green flag means that there is complete evidence for and no evidence against (no red). An all-red flag means that there is complete evidence against and no evidence for (no green). An all-white flag means there is no green evidence for and no red evidence against and so we really ‘do not know’ or indeed have no view. Figure 9.3 shows a simple model of a process with two sub-process holons which are both necessary for the success of the top process. The players who own each process associate an Italian flag with that process. The flag represents their view, based on evidence, that the process will be successful. Clearly there is something wrong in Fig. 9.3, since the flags are obviously inconsistent. This means that when the process owners realise this inconsistency, they can discuss the reasons



9.2 An Italian flag.



9.3 A simple map of three processes. Note: The Italian flag of evidence for Process C indicates that it is very likely to fail. Unfortunately the owner of Process A has an Italian Flag that indicates that Process A is likely to be successful. If Process C is necessary for Process A then the two process owners need to sort out the reasons for their differing perceptions.

for it and decide on adjustments or on what needs to be done to improve the chances of total success.

Until recently, the flags were used entirely qualitatively as described in Blockley and Godfrey (2000). A new algorithm developed for the purpose of propagating the flags mathematically has been described recently (Blockley, 2008; Marashi, 2006). However certain constraints and assumptions need to be appreciated before the method is used. The new algorithm is implemented in two 'sister' pieces of software called PERIMETA (Performance through Intelligent Management) and JUSTISE (Joined Up Systems Thinking for Integrating Synergy in Engineering). An Italian flag is defined mathematically as an interval probability – so the probability of an event or proposition E has a lower bound El and an upper bound Eu . Therefore $p(E)$ is an interval number as follows:

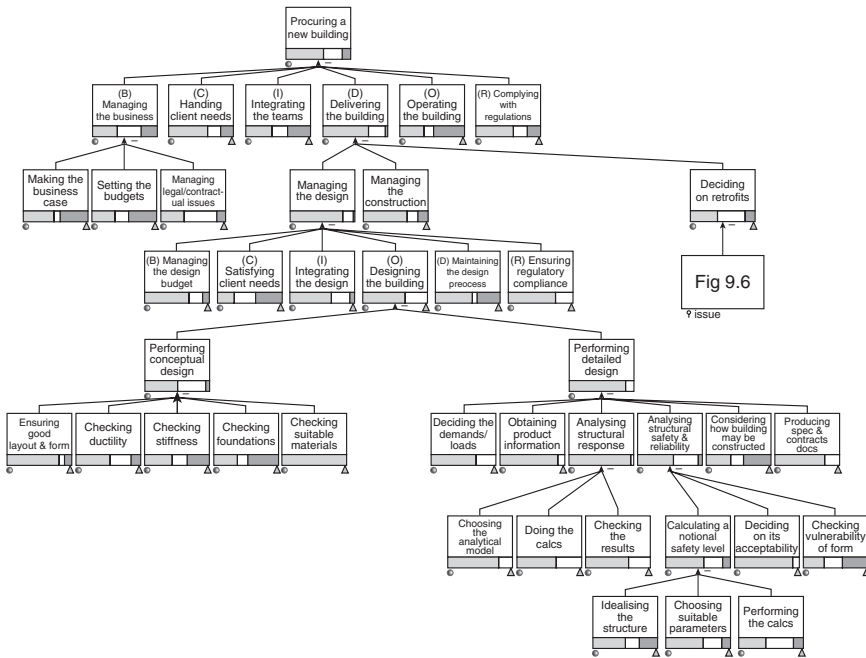
$$p(E) = [El, Eu] = [g, (1-r)] \quad [9.2]$$

Here g represents the green part of the flag colouring an interval on the scale $[0, 1]$ from 0 to g and is a measure of the probability of the set E . Likewise r is the red part of the flag from $(1-r)$ to 1 and is a measure of the probability of the set $-E$. The white part of the flag is therefore $w = 1 - g - r$ and is a measure of the probability of the unknown events in the set (E or $-E$). The white interval enables us to deal with incomplete and possibly inconsistent information which we can interpret as 'don't know'.

9.9 A structural example: procuring a new building

Assume we are concerned with risks to the procurement of a new building. Since this new building is to be in a seismically active zone, we also want to consider possible future retrofits. Following the process methodology described by Blockley and Godfrey (2000), the first task is to identify a process model as shown in Fig. 9.4. In doing so, one has to draw a *system boundary* – so we must include not just direct hard systems technical issues but also soft systems social and political issues that may affect our building. We use the BCIOD+R (Business, Customer, Integrating, Operations, Delivery + Regulatory) framework, as set out by Blockley and Godfrey (2000). Note that, due to space restrictions, the model shown in the figure is incomplete – the intention is that the hierarchy is developed sufficiently to illustrate the methodology.

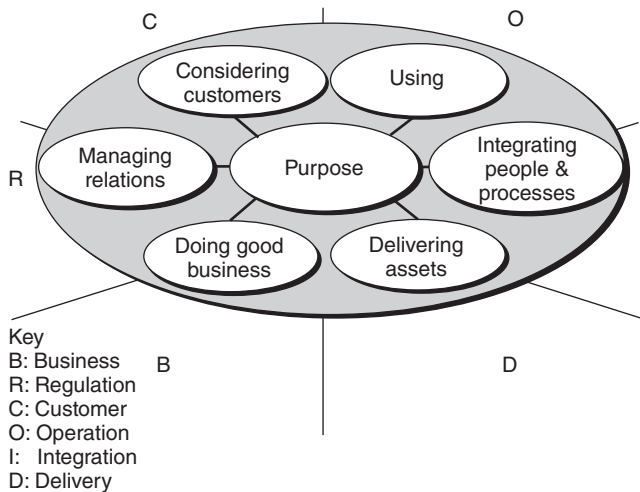
Some Italian flags of evidence are also shown in the figure – remember that they are dynamic as they change during the life of the project. The input flags at the bottom of the tree can either be input directly by the relevant process owner or derived from a performance indicator using a value function as described by Hall *et al.* (2004). The flags for the higher process are calculated using the pair-wise algorithm described earlier. This



9.4 Some processes for procuring a building.

includes the very top process which is 'Procuring a new building'. The methodology facilitates the integration of information of many types from measurements to expert judgement. It is important to stress that the system is dynamic, i.e. it is constantly changing. Although we have not yet implemented a system on a project management internet or intranet, it is anticipated that the model could be available as read only; all players in the project with write access for process owners can change the data concerning their own processes as and when required. In this way, all players can see the effects of changes on the project in detail and as a whole. The system enables the sharing of information – say through a hand-held device available on site. The whole system then acts as a decision tool to inform decisions about priority actions.

Note that Fig. 9.4 includes all processes – both soft and hard. In other words, the model is not restricted to the physical building but includes all relevant information of all kinds. The pedigree of the information clearly varies enormously and that is catered for through the input data. Thus if a difficult judgement which is necessarily imprecise is put alongside information from a precise measurement or calculation then this can be allowed

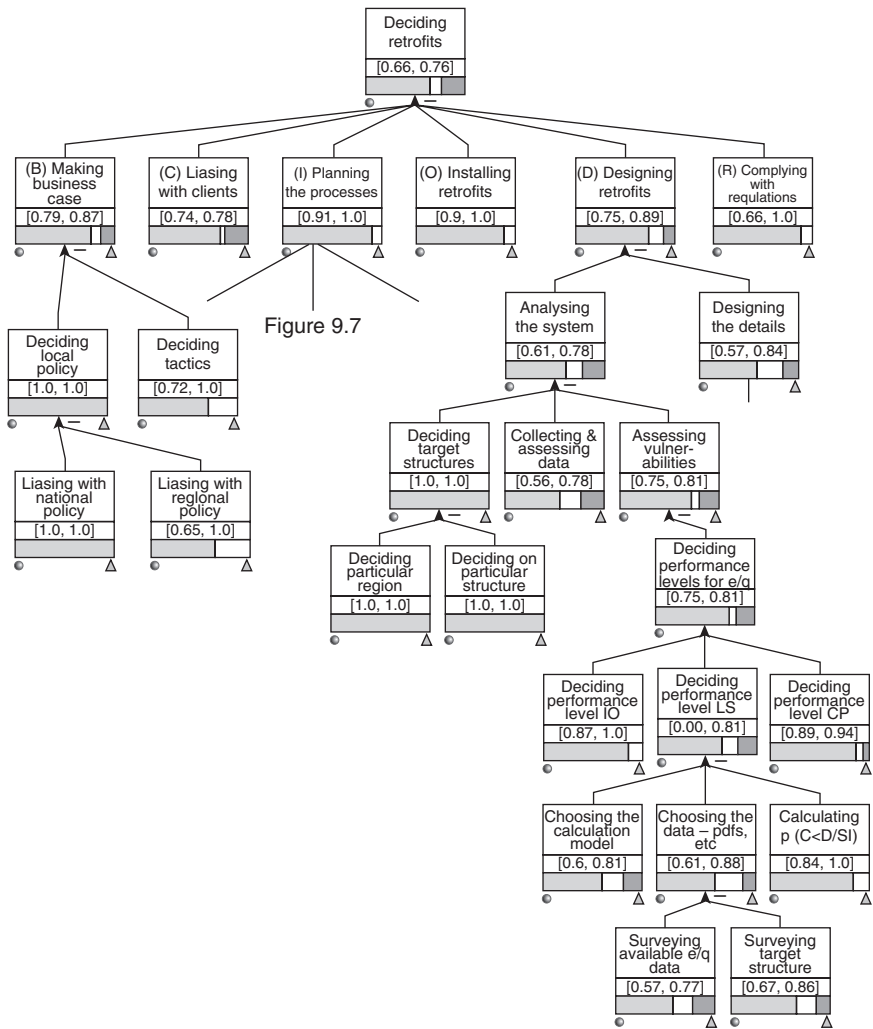


9.5 Interdependence of BCIOD+R processes.

for by a suitable adjustment of the relationships between the measurement and the flag. The computer system JUSTISE also allows notes to be written and links to be made to any relevant documents so that process owners and players as well as stakeholders with access permission can immediately find and see any changes. Note also that the BCIOD+R processes are interdependent (Fig. 9.5) in a manner that has to be captured in the model.

Figure 9.4 refers to Fig. 9.6 which covers some of the processes for retrofitting. Then Fig. 9.6 refers to Fig. 9.7 which again shows some of the processes defined in MARIUN to monitor both hard and soft system processes. These include dealing with unresolved issues, controlling the retrofitting projects, and monitoring its performance right down to the detailed level of checking analytical and design processes. Figure 9.8 shows relationships between some detailed processes in designing a group of beams in a structure.

The choice and design of the processes in all of these models are crucial to the success of working this way and must be performed by project teams and appointed process owners. So how would such a model be used on a project intranet? All participants in a project would have write access to their 'piece of the action' for which they are responsible and accountable. This is not onerous, since after an initial input of contact details and other relatively stable information, the only changes may be values of key performance indicators (KPIs) and/or judgements about the Italian flags – but of course all information can and does change. Project teams would be able to scan over all of the process flags to which they have been given

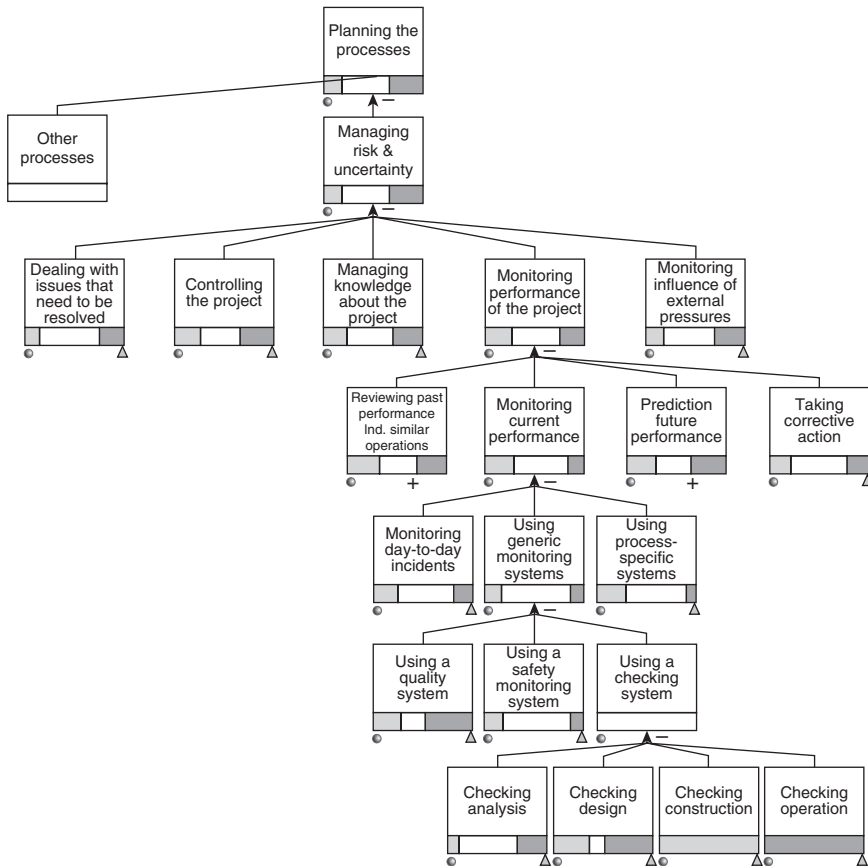


9.6 Some processes for deciding retrofits.

permission to access. Such scans will enable the spotting of possibly previously hidden interactions that could cause problems. In this way decisions are taken by everyone involved in full awareness of the impact of other decision makers on total project success.

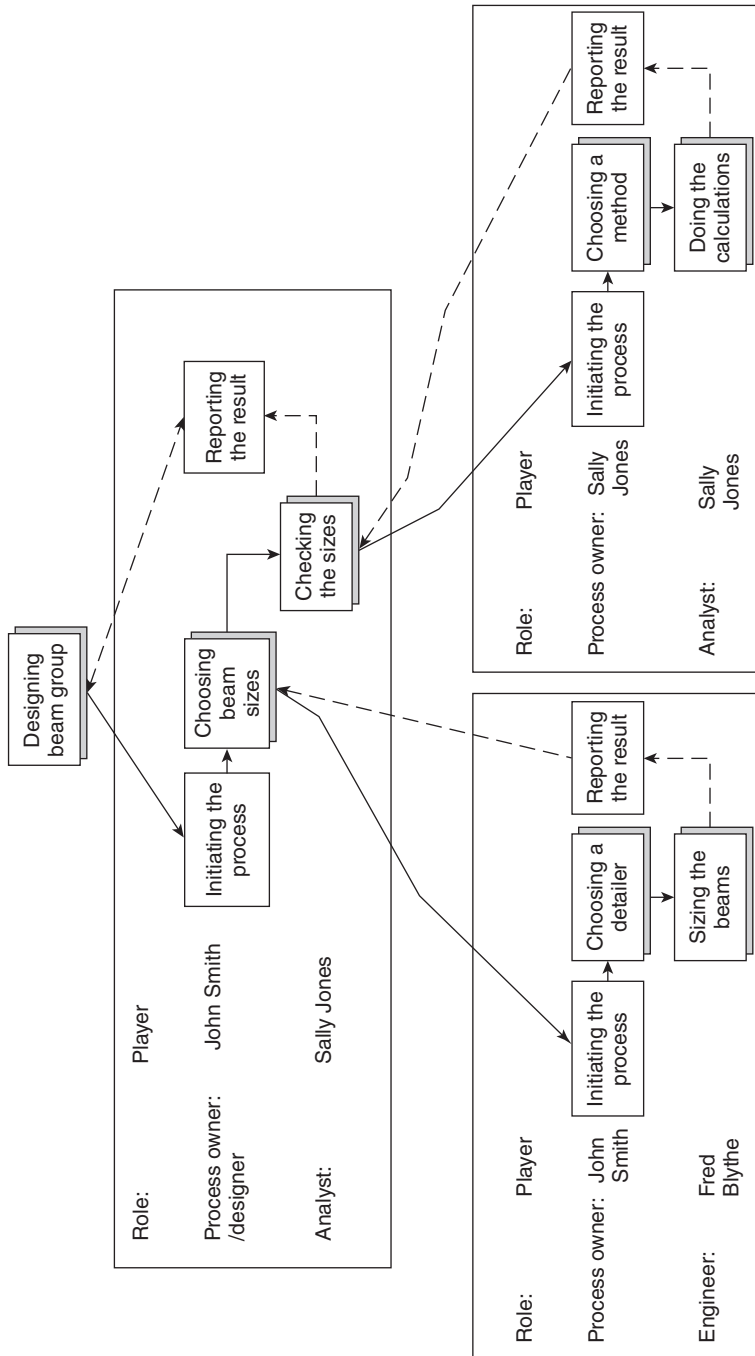
9.10 Conclusions

- There is a real need to integrate the way we manage different risks based on evidence from different types of information.



9.7 Some processes from MARIUN.

- Systems thinking enables us to widen our system boundaries to include all risks relevant to the safety of a structure.
- Process maps can be used to connect data, information, people, and functions, and provide routes by which we can manage changes more effectively.
- Presently, there is a clear distinction between hard and soft system risks, and they are treated as though they are independent. Case histories demonstrate that problems often occur at their interface. Hard systems are embedded in soft systems and they can be integrated by modelling them both as an interacting sets of process holons.
- Italian flags based on interval probability theory are a practical way of representing the dependability of evidence in a process model. They can be qualitative – or with careful understanding of the underlying mathematics, they can also be used quantitatively.



9.8 Dome detailed design processes.

- The Italian flag can be used as part of a social process to improve our understanding of, and judgements associated with, incompleteness of knowledge.
- No one existing methodology is necessarily rejected by the approach suggested here. Merely, it is stressed that all methodologies bring partial evidence to the whole and the emphasis here is on a way in which this evidence can be brought together and better integrated for synergy.

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A capability approach for seismic risk analysis and management

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Abstract: Current approaches for seismic risk analysis conceptualize the consequences of seismic hazards too narrowly. While they are appropriate for risk analysis and management of individual structures, they fail to capture the actual impact of seismic hazards on society. The limitations of current approaches are discussed along with a list of desiderata for a framework better suited for societal seismic risk analysis and management. This chapter proposes a capability approach that meets such desiderata, where capabilities refer to what individuals are still able to achieve or do in the aftermath of a seismic event. In the proposed framework, individual capabilities are used to capture the net societal impact, including both potential benefits and opportunities. The proposed capability approach provides the information needed for rational decisions on the acceptability and tolerability of risks, informed and fair public policies, optimal allocation of resource, enhancing the quality of life, and promoting a sustainable development.

Key words: capability approach, risk analysis, risk management.

10.1 Introduction

Earthquakes and tsunamis present societal risks. Such risks are different from individual risks (i.e., the risk that an individual may take alone; Murphy and Gardoni 2011a), because of the complex processes that result in the possible consequences of a given natural event and their probabilities of occurrence. Moreover, entire communities might be affected by such risks. As a result, risk analysis and management for natural hazards need to consider a number of factors that are not typical for individual risks.

As Peter May writes, '[s]eismic safety is a matter of public welfare involving potential loss of life or injury, disruption of communities, and costs to governments for addressing earthquake losses and recovery' (May 2001). Earthquakes are one of the natural hazards communities might face. Strategies to promote safety share the same general goal: protecting the public by minimizing the societal impact of hazards. This goal informs the three major objectives of public policy and resource allocation strategies: (1) reducing the potential losses from hazards; (2) increasing awareness of and

information about hazards in vulnerable communities; and (3) improving our understanding of hazards and the efficacy of mitigation (Murphy and Gardoni 2007).

Seismic safety of a community is achieved (or fails to be achieved) depending on the choices of individual members of a community, members of organizations, corporations and professions, and policy makers. Common policy strategies for dealing with hazards include setting federal money aside for risk reduction and disaster aid, regulating public and private sector activity through business regulations (e.g., guidelines for insurance companies and construction firms), building codes, land use and zoning ordinances (Murphy and Gardoni 2007). Engineering codes play an important role in promoting seismic safety, because they outline minimum seismic performance standards (May 2001).

Informing the decisions of policy makers, organizations, and individuals regarding seismic safety is a conception of what makes a risk acceptable. Standards for what constitutes acceptable levels of risk for members of a community, for example, informs judgment about what makes a building or structure (or requirement) safe enough. However, the proper way to specify standards for acceptable risk is extremely challenging.

Given that this is fundamentally a value judgment, this would suggest that the public, via a process of collective decision-making, determines such levels. At the same time, it is important for standards to be formulated on the basis of a comprehensive understanding of the risks involved and that information is not always accessible to the public. Moreover, the public is at times indifferent to risks that they face; this is especially true in the context of seismic risk (Murphy and Gardoni 2006, 2008; Gardoni and Murphy 2009). This suggests that experts determine standards or at least play a role.

A further complication is that there are multiple considerations that influence our judgment about the acceptability of risks. For example, equity in distribution affects the acceptability of a risk (Murphy and Gardoni 2008). Equity considerations are relevant for the distribution of the potential losses from a hazard, the distribution of resources to mitigate such potential losses, and the benefits to be gained by accepting a certain level of risk. In addition, there is a strong desire to reduce or minimize losses from hazardous events, which suggests that the standard for acceptable risk should be quite demanding. On the other hand, it is generally recognized that there is a desire to maximize the benefits that may accrue to communities if they are willing to accept certain risks (May 2001). Indeed, some scholars point out that a refusal to accept certain risks may close off possibilities for new technologies or opportunities (Wildavsky 1988). Considering earthquake safety specifically, May (2001) writes, 'the least risky choice would be not to build a given structure in a seismic-prone area. That, of

course, precludes any benefits from activities within the structure. Thus, the decision to build a structure itself entails some risk that can be tolerated because of the benefits of the building or facility.'

Judging a risk to be unacceptable typically implies that action should be taken by a community to mitigate such risks. However, mitigating or guarding against certain losses requires resources, which could be put to other uses if not devoted to such mitigation. In addition, communities often are vulnerable to multiple hazards. Thus, determining the level of risk that is acceptable (or target levels of safety in reverse) in part involves deciding whether mitigation action should be triggered if a risk is judged to be unacceptable (or the relationship between standards for mitigation action and acceptable risk) (Murphy and Gardoni 2007, 2008; Gardoni and Murphy 2008).

Finally, determining target levels of performance and acceptable levels of risk requires understanding the possible negative consequences of hazardous scenarios, or losses due to a seismic event, and benefits from accepting certain risks (Lindell and Prater 2003; Cutter *et al.* 2003; Murphy and Gardoni 2007; Gardoni and Murphy 2008). Identifying and measuring losses involves value judgment about which kind(s) of potential losses should be considered and what metric should be used to assess such losses. Potential losses must be defined in a way that captures the indirect and broader ramifications that hazards can have for communities (May 2001).

There are a number of widely recognized challenges to developing a satisfactory metric for gauging potential consequences of hazardous scenarios (for a comprehensive summary of these challenges see Murphy and Gardoni, 2006, and Gardoni and Murphy, 2009). Losses can be conceptualized in terms of lost capital ('including repair and replacement costs for structural and nonstructural components, buildings content loss, business inventory loss') or income ('business interruption, lost wages') in an annualized manner (May 2001). The choice of metric is important in part because it can influence how risks of various kinds are ranked. In particular, the way in which value should be assigned to human life, and whether, for example, to assign different values to different life expectancies or voluntary exposure to risk (Stern and Fineberg 1996, pp. 52–53) is the subject of ongoing dispute. When market values are used to assess losses, valuation of non-market goods, such as sustainability, becomes more complicated (Gardoni and Murphy 2008).

These considerations suggest a number of desiderata that an approach for seismic risk analysis and management ought to satisfy. Such desiderata are discussed in the next section. Next, the chapter presents a capability approach for seismic risk analysis and management. Finally, the chapter discusses the benefits of a capability approach over traditional approaches available in the literature.

10.2 Desiderata for a framework for seismic risk analysis and management

First, an approach should provide a common metric for assessing different kinds of consequences that capture direct as well as indirect impacts of hazards on communities. Second, it must provide a way of assessing the distribution of consequences across a community. Third, it should inspire confidence in the public, both with respect to the methodology used and the conclusions generated. Fourth, it should provide theoretical resources for prioritizing which hazard to address and which mitigation strategies to pursue.

There are a number of indexes that have been or could be used to assess the impact of events based on the consequences for the quality of life of individuals (e.g., Nathwani *et al.* 1997; Rackwitz 2002, 2006; Cutter *et al.* 2003; Pandey and Nathwani 2004; Ditlevsen and Friis-Hansen 2007). The Life Quality Index (LQI) (Nathwani *et al.* 1997; Rackwitz 2002, 2006), the Life Quality Time Allocation Index (LQTAI) (Ditlevsen and Friis-Hansen 2007), and the Societal Willingness to Pay (SWTP) (Pratt and Zeckhauser 1996; Pandey and Nathwani 2004) are examples of such indexes. Quality of life is typically assessed based on two factors: (1) life expectancy (which provides information about the length of life in good health) and (2) gross domestic product (GDP).

While these indexes share an emphasis on well-being and the quality of life of individuals with the capability approach (which will be described in the next section), they have two important limitations (Gardoni and Murphy 2009). First, they adopt a narrow conception of the quality of life, focusing strictly on length of life and the economic resources available to individuals as indicators of the quality of life. However, other factors, such as the opportunity to be mobile, educated and employed, intuitively seem to influence the quality of life an individual enjoys. Moreover, the GDP of a community does not always tell us about the quality of life members of a community enjoy, and in particular whether they have these additional opportunities. Economic resources are not sufficient to guarantee many important opportunities. For example, the opportunity for education depends on a variety of factors, such as the vicinity of schools; the availability of skilled instructors; whether a school charges tuition, and at what level; as well as the distribution of resources within a community.

Advocates of one of the above indexes may respond by trying to expand the factors considered when evaluating the quality of life. However, and this is the second limitation, such indexes do not provide a principled way to combine different consequences risks might have for the quality of life into a composite index, nor standards for determining which additional dimensions might be considered.

10.3 A capability approach for seismic risk analysis and management

The risks that earthquakes pose to communities have important implications for engineering (in terms of mitigation actions and construction practices). This section first provides an overview of how risk is conceptualized and measured by a capability approach along with a conceptual outline of the practical implementation of a capability approach for risk analysis. Additional details on the mathematical formulation of a capability approach can be found in Gardoni and Murphy (2009, 2010) and Murphy and Gardoni (2010). We then consider how standards for acceptable risk are formulated in this approach. Finally, we discuss the public policy guidelines supported by a capability approach.

10.3.1 Risk determination and measurement

Determining the risks due to natural events requires calculating the probability of occurrence and potential consequences of hazardous scenarios. In the capability approach, the potential impact of a hazardous scenario is defined in terms of the effect on individuals' capabilities.

Capabilities refer to the genuine opportunities open to individuals to achieve valuable states and activities, or functionings (Sen 1999a,b; Nussbaum 2001). Being nourished, being educated, and being mobile are examples of functionings. The well-being or quality of life of individuals is a function of the capabilities they enjoy. Whether or not an individual has a genuine opportunity to achieve a functioning depends on two factors: his or her resources and what he or she can do with those resources. Resources are understood to broadly include internal assets (such as skills) and external goods (such as income). What an individual can do or become with a given set of resources is profoundly influenced by the physical infrastructure and social and political context within which he or she acts. To illustrate the influence of these two factors, consider mobility in the aftermath of a natural event. Whether an individual has a genuine opportunity to be mobile depends on his or her resources, such as the possession of a bicycle as well as venues where to ride the bike. Mobility also depends on whether he or she can use a bike as a form of transportation. The presence of roads or paths for bikes is important in this regard, and these are often damaged or destroyed by natural events.

In practice, only a limited number of capabilities, or genuine opportunities, are open to an individual at any time (Robeyns 2006, p. 354; Murphy and Gardoni 2010). This is because of the ways that diverse capabilities interact. That is, choosing to pursue one opportunity, such as being educated, may enhance other capabilities, such as being employed. Conversely,

the pursuit of certain functioning may foreclose others. With limited economic resources, devoting resources to secure shelter may limit the extent to which an individual can be adequately nourished. This example also highlights the importance of security or stability in what an individual is able to do or become (Wolff and de-Shalit 2007). For example, when limited economic resources are available through temporary employment, the security with which an individual can be sheltered may be undermined. In this case, the capability of an individual to be sheltered is diminished.

Choosing appropriate capabilities for risk determination and measurement

A risk analysis cannot take into consideration the impact of a hazardous scenario on every capability of an individual. Choices must be made regarding which capabilities, and so which dimensions of well-being, will be taken into account. There are five general criteria that should guide the selection of capabilities (Gardoni and Murphy 2009). First, a selected capability should be *relevant*, in the sense that it is a capability that a given hazard typically affects. A selected capability should be *important*, in the sense that it refers to a state or activity that are significant enough to influence recovery and mitigation policies. Third, a selected capability should be *influenceable*; that is, we can alter the consequences of a given hazard on that capability through mitigation action. Fourth, a capability should *provide information* that is not captured by those other capabilities. Fifth, the *fewest possible capabilities* should be selected, consistent with providing a comprehensive picture of the impact of a hazard on a community. This enhances the practical implementability of the capability approach.

Choosing appropriate indicators

Once capabilities are selected, they must then be quantified (Gardoni and Murphy 2010), or in the case of risk analysis the change in each capability must be predicted. Indicators provide information about whether, and to what extent, a given functioning (e.g. being adequately nourished) is achieved. For example, one possible indicator for being adequately nourished is average caloric intake. A possible indicator for being educated is the percentage of children enrolled in school. However, indicators only capture the actual achievement while capabilities refer to the potential ability to achieve if an individual decides to do so (Robeyns 2006; Wolff and de-Shalit 2007). Murphy and Gardoni (2010) developed a peer method to gauge the capabilities of individuals by looking at the achieved functionings of other individuals in the same societal group. This is based on the assumption that even if an individual chooses not to achieve a functioning

he or she could achieve the functionings that his or her peer chooses to achieve.

Adequate indicator(s) for a given capability must satisfy two criteria (Gardoni and Murphy 2009). First, an indicator must be representative, or such that it tracks the level of a given capability in practice. Appropriate indicators for a given capability may be to some extent hazard-specific. Second, it should be reasonable in the sense that there is an intuitively plausible relationship between the indicator and the capability at issue. This second criterion is important for risk communication and effective policy-making for natural hazards. If there is no intuitively plausible relationship between a given indicator and its corresponding capability, this makes it more difficult to communicate the results from a risk analysis to the general public and to generate trust in the results.

Current engineering practice offers a rich set of potential indicators. The information engineers currently use to assess the consequences of disasters for communities, such as the number of individuals killed or injured, seems to provide plausible data about how a community was affected. The capability approach urges us to consider what kind of information the data is providing, specifically in terms of what it is telling us about how specifically the lives and well-being of individuals are affected.

There is a second way that current engineering practice can inform a capability approach to risk. Engineering predictive models as well as past data from actual disasters can inform the prediction of the likely value of indicators (Gardoni and Murphy 2009). Predictions of the value of indicators must account for uncertainties surrounding the magnitude and time of occurrence of a given earthquake (or hazard more generally) and what the impact of a given earthquake on a specific indicator will be. Confidence intervals can be used to specify the probability that the actual value of an indicator will be within a given range. The mean can be used to specify the expected value of a given indicator. A probability distribution can provide a picture of the likelihood of each potential outcome, and so a picture of the differential impact that hazards of different magnitudes might have.

Finally, it is important to note that it may be necessary to change the indicators that are used to assess a particular capability, depending on the period of time a given risk analysis is considering when determining the potential impact of a hazard (Gardoni and Murphy 2009). Consider the capability of being sheltered. The number of individuals homeless as a result of a hazard might serve as an indicator for the initial impact of the hazard. However, that indicator might not provide an accurate picture of this capability over time. In the long term, few members of a community might be literally homeless. However, housing for victims of a hazard might be largely temporary, subsidized by the government for a specific period of time. The temporary nature of such housing would render the capability

for shelter insecure. Thus, an indicator that provided information about whether a residence was permanent could be more accurate. The process of normalizing indicators, which we discuss below, provides a way of combining information from different dimensions of well-being into a composite account.

Scaling and combining indicators

Each indicator provides information about a distinct dimension of well-being. To be able to combine diverse kinds of information together to form a composite picture of the overall well-being of individuals, the process of normalization (or scaling) is needed (Gardoni and Murphy 2009). In this process, dimensionless indices called indicator indices (II) that range from 0 to 1 are created for each indicator. These IIs are based on a comparison of the value of an indicator against reasonable maximum and minimum values for that indicator. A value of 0 indicates that a hazard will have no impact on a given capability. A value of 1 signifies that a hazard will have the maximum foreseeable impact. Finally, the IIs for diverse dimensions for a given individual are combined with a procedure described in Gardoni and Murphy (2009) and Murphy and Gardoni (2010), providing a composite picture of the impact a hazard may have on well-being creating a hazard index (HI) that also ranges between 0 and 1.

Putting the HI in context: hazard impact index (HII)

For a given value of HI, single individuals in a larger society will be impacted less on average than single individuals in a smaller society (Gardoni and Murphy 2009). This is because more people would share the impact and there would likely be more collective resources for the recovery. For example, what an HI means for China is different from that for El Salvador. Therefore, the societal impact captured by HI needs to be converted into an impact per capita by dividing HI by the size of the population affected by the hazard, creating the hazard impact index, HII. Additional details on the procedure for creating the HII can be found in Gardoni and Murphy (2009).

Finally, as noted in the earlier section of this chapter, information about the distribution of risks across a population is important for policy and decision makers. To ascertain the likely distribution of adverse consequences, a process of disaggregation can be used similar to that used by the United Nations in assessing the level of development of sub-groups within a community (Jahan 2003). In disaggregation, the HII for each sub-group is computed and can then be compared against standards for acceptable risk.

10.3.2 Risk acceptability and tolerability

In a capability approach, judgment about the acceptability of a risk due to a natural hazard such as an earthquake is based on a comparison of the likely impact on capabilities with two thresholds (Murphy and Gardoni 2008). The first is a threshold for acceptable risk. This threshold identifies the level of capabilities it is acceptable in principle for individuals to have in the aftermath of a hazard. Put another way, the threshold for acceptable risk identifies a level below which the capabilities of individuals should not fall. Because there is uncertainty surrounding what impact a given hazard will have upon capabilities, it is necessary to specify the probability that capabilities will be above the acceptable threshold.

The second is a threshold for tolerable risk. This threshold is lower than the acceptable threshold. Specifying a tolerable threshold is a response to the recognition that it may not be reasonable to demand that communities guarantee the acceptable level of capabilities even in the immediate aftermath of an earthquake. Communities might permissibly fall below the acceptable threshold in the emergency phase, provided this is a temporary and reversible situation. At the same time, there should be a limit to the impact of capabilities that communities permit even at that time. The tolerable threshold responds to both of these concerns. It identifies a level of capabilities below which it is never permissible to go, even if this level is temporary and reversible.

To illustrate the basic idea behind these two thresholds, consider the capability for shelter. Having a genuine opportunity to be sheltered in a secure manner depends upon the presence of a materially adequate, given the environmental conditions of a particular locale, and stable dwelling. It may be permissible for a community to allow a certain probability of a specified level of damage to dwellings that, if experienced, would make it necessary for some individuals to find alternate dwelling. The potential loss of the use of a stable dwelling would mean that the capability for shelter is to some extent insecure. Available alternate dwelling, if for example temporarily provided by the government, would not satisfy the acceptable level of shelter. However, such temporary housing may be tolerable, provided it is temporary and reversible.

The process of developing and applying these thresholds can respond to the concerns about involving the public in the process of evaluating risks, while acknowledging the concern about the limits to the public's knowledge and understanding of the technical dimensions of risks (Murphy and Gardoni 2008). Acceptable and tolerable thresholds articulate certain value judgments about the kind of opportunity a community is committed to ensuring its members enjoy. Because of this, it is appropriate for the public to play a role in determining what these thresholds are. Acceptable and

tolerable levels of risk may be decided through a public decision making process. The question that guides such decision making is what is it we as a community want to ensure that all members have an opportunity to achieve, even if disaster hits? This question is one that the public can understand and not be indifferent towards.

The thresholds can also be sensitive to distributional concerns. By using a method of disaggregation discussed in the previous section, we can compare the anticipated impact on sub-groups within a community against the thresholds of acceptable and tolerable risk, to ensure that no group of citizens is vulnerable to unacceptable risk.

10.3.3 Public policy and resource allocation

In a capability approach, the purpose of mitigation action for natural hazards is to protect and promote the capabilities of individuals by reducing the potential impact of natural hazards on them (Murphy and Gardoni 2007, 2011b, 2012). Given this aim, policy makers require a way of identifying when mitigation action is required and, when required, determining which of the available policy alternatives should be pursued. Below we describe the basis on which these decisions would be made.

Mitigation action is potentially required only when a risk is judged unacceptable or intolerable (Murphy and Gardoni 2007). In this case, the basic level of capabilities that should be available to individuals is compromised. There may be a number of risks that fail to satisfy the acceptable threshold. Generally, risks that fall below the intolerable threshold should be prioritized as targets for mitigation action. Once such risks are above the intolerable threshold, the focus should turn to risks that are tolerable but unacceptable.

There may be a number of different policies that could be pursued to mitigate a given risk. Thus, different policies must be evaluated and ranked. In a capability approach, the range of options that should be viewed as *viable* is identified. Viable options are likely to reduce risks to the tolerable or acceptable threshold by reducing either the probability of occurrence of a hazard or its expected impact. In the case of earthquakes, engineering mitigation strategies will focus on reducing the expected impact by modifying building codes, retrofitting structures or altering zoning laws.

More than one policy might be viable. In this case a principled basis for selecting among viable options is required. In a capability approach, mitigation strategies should be prioritized based on their likely affectability, the expected dollar per unit change in the HII (Murphy and Gardoni 2007), and available funding. The quantification of the reduction in the expected impact is determined by the likely reduction in the decrease (or increase) in a given capability as gauged by the corresponding indicator. Once the

expected dollar per unit change of viable policies is determined, a sensitivity analysis can further aid understanding of which policy(-ies) will most significantly affect the predicted impact.

10.4 Benefits of a capability approach

There are a number of benefits to using a capability approach when dealing with questions of seismic safety and risk. In this section, we provide an overview of some of these general benefits, focusing on those related to the desiderata for an adequate account discussed in the previous section (for a comprehensive overview of these benefits, see Gardoni and Murphy, 2009).

First, a capability approach conceptualizes consequences in a way that captures the broader consequences of a hazard for a community and measures these consequences using a consistent metric. The capability approach identifies those dimensions of well-being that are characteristically impacted by natural hazards as the consequences risk analysis should consider. By framing well-being in terms of capabilities, this approach can capture the broader and indirect consequences of hazards, because of the impact of these consequences on the genuine opportunities of individuals. The more comprehensive picture of the societal impact of hazards is needed for accurate policy and decision-making regarding natural hazards.

Moreover, the metric used to assess the societal impact of natural hazards does not rely upon controversial judgment about the monetary value of a human life, nor does it require us to assign a market value to all consequences. Instead, the consequences are defined on the basis of distinct indicators.

Second, a capability approach explicitly considers the fairness of the distribution of risks and of the benefits of mitigation efforts. A method of disaggregation can be used to determine the differential impact of hazards and/or mitigation efforts on sub-groups within a population. Third, it can address a concern for environmental and sustainability in the long run. Permanent environmental damage reduces many capabilities of individuals.

Fourth, the capability approach can facilitate communication about risks and mitigation policies to the public and among diverse groups of policy makers, in the process generating confidence among the public in the methodology and results from the approach. The basic idea of capabilities is intuitively understandable, and the indicators used to measure capabilities intuitively plausible. The approach is transparent in value judgment that underpins it, in particular the general concern with protecting and promoting individual capabilities. It is also transparent in the standards used to judge the acceptability of a risk and the basis for making mitigation decisions. Fifth, and relatedly, the capability-based approach provides a space for the public to inform policies regarding seismic safety and risk, while at the same time leaving a distinct role for risk experts in policy implementation.

10.5 Conclusions

Seismic safety is an important concern for communities. Managing seismic risks requires comprehensive understanding of the risks that seismic events pose and standards for evaluating when it is necessary to take mitigation action and what action should be pursued. In this chapter a capability approach to seismic risk analysis and management was presented. In a capability approach, consequences of hazardous scenarios for communities are conceptualized and measured in terms of changes in capabilities. Standards for risk acceptability are defined with reference to the level of capabilities communities are committed to sustain and protect. Mitigation action is selected on a basis of the impact a given strategy will have upon capabilities. This approach balances a concern for including the public in decision-making about risk, while at the same time finding a role for experts in mitigation policy. Because the idea of capabilities is easily communicable to the public, the capability approach can generate confidence in the public in the methods used to evaluate and respond to seismic risks.

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Resilience-based design (RBD) modelling of civil infrastructure to assess seismic hazards

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Abstract: The chapter addresses a new design methodology for civil engineering structures called ‘resilience-based design (RBD)’, which can be considered as the next generation of performance-based design (PBD). Currently, engineers approach a structure as if it stands alone, without considering the interaction with the community. However, instead it should be considered as an integrated part of the design process. Indeed, the building should not be considered alone, but as a group of buildings using the ‘portfolio approach’ which would allow regional loss analysis. The goal of RBD is to make communities as ‘resilient’ as possible, developing technologies and actions that allows each structure and/or community to regain its function as promptly as possible. The fundamental concepts of community resilience are analysed and a common reference framework based on the acronyms called ‘PEOPLES’ is established. Emphasis is given to hazard intensity measures, engineering demand parameters, while a new performance matrix is proposed to define the performance limit thresholds for RBD. Some applications are presented incorporating both performance and resilience objectives in order to illustrate the feasibility of the proposed method.

Key words: resilience, resilience-based design (RBD), performance-based design (PBD), reliability, limit thresholds, community.

11.1 Introduction

Past earthquakes such as San Fernando (1971) (Jennings, 1971), Loma Prieta (1989) (Benuska, 1990), Northridge (1994) (Hall *et al.*, 1996) and Kobe (1995) (Comartin *et al.*, 1995) are often remembered for particular structural problems, because the scientific community mainly focuses its attention on these aspects, while recent earthquakes draw the attention of the researchers mainly to the effects on communities (2004 Indian Ocean Tsunami (Iwan, 2006), 2008 Wenchuan (Lin *et al.*, 2008), 2010 Haiti (Des-Roches, 2011), 2011 Christchurch (Deam *et al.*, 2011), and 2011 Tohoku earthquake (Takewaki, 2011)) rather than structures. Natural disasters in general can cause widespread moderate to severe damage that may diminish the ability of a community to respond. Community in this chapter is defined as a social unit larger than a household that shares common values

and has social cohesion. Widespread damage can have a substantial long-lasting social, economic and cultural impact on the well-being and vitality of a city and nation. Some recent examples are the earthquakes in the three continents:

- 2009 L'Aquila earthquake in Italy (Europe);
- 2011 Christchurch earthquake in New Zealand (Oceania);
- 2011 Tohoku earthquake in Japan (Asia).

The 2011 Tohoku Earthquake caused \$319 billion in direct damage and \$619 billion costs of recovery and other indirect losses, but mostly it showed that post-earthquake evaluation needs improvements in order to be more systematic, accurate and quicker trying to allocate resources and personnel involved in the recovery process efficiently. However a magnitude 9.0 event is very large and it is not possible to avoid chaos in the response. Overall the military forces were very effective in rescue activities, as were the emergency authorities.

The 2011 Christchurch earthquake (EERI, 2011; Muir-Wood, 2012) in New Zealand will be remembered for the effect of very close main shocks and numerous aftershocks. New unmapped faults were found during these events which is a quite common characteristic of every earthquake. The presence of all these uncertainties has shown the current limitations of the Uniform Hazard Spectrum (UHS) which is built using known seismogenic earthquake sources defined in terms of location, geometry and characteristics which are time and spatially independent. A M_w 6.3 earthquake aftershock was registered near Christchurch, New Zealand, on Monday, February 21 (UTC), 2011, being triggered 6 months after a M_w 7.0 earthquake struck the South Island of New Zealand. Obviously these two events cannot be considered as time and spatially independent, therefore recent developments (Yeo and Cornell, 2009) are able to incorporate aftershocks effects in probabilistic seismic hazard analysis (PSHA), but these methodologies have not been incorporated in the assessment of the UHS.

The 2009 L'Aquila earthquake in Italy showed a well-organized emergency response in the short term, but the long-term reconstruction phase is showing poor performance of decision makers and politicians who are also an essential part of the recovery process. It also pointed out the necessity to rehabilitate old masonry buildings using traditional retrofit techniques which performed pretty well when properly designed and installed, reducing building vulnerability (Cimellaro *et al.*, 2010a).

After all these recent extreme events, the international community has become aware that resilience is the key to describe earthquake engineering performance. Disaster resilience is often divided between technological units and social systems (Bruneau *et al.*, 2003). On a small scale, when considering critical infrastructure, the focus is mainly on technological aspects. On

a greater scale, when considering an entire community, the focus is broadened to include the interplay of multiple systems – human, environmental, and others – which together add up to ensure the functioning of a society.

Various studies have been carried out, with the goal of practically evaluating the concept of resilience and identifying the main units of measurement of it. Miles and Chang (2006) presented a comprehensive conceptual model of recovery, which establishes the relationships among a community's household business, lifeline networks and neighbourhoods. Davidson and Cagnan (2004) developed a model of the post-earthquake restoration processes for an electric power system. Chang and Shinozuka (2004) contributed to disaster resilience by discussing a quantitative measure of resilience based on the case study of the Memphis water system. Bruneau and Reinhorn (2007) for the first time related probability functions, fragilities and resilience in a single integrated approach for acute care facilities. While this literature survey is by no mean comprehensive, it is presented here to highlight several distinct techniques, and set the stage for future developments in this work.

Resilience (R) in this chapter is defined as the capability to sustain a level of functionality or performance for a given building, bridge, lifeline networks or community, over a period defined as the control time T_{LC} that is usually decided by owners or society (usually is the life cycle, lifespan of the system, etc.). The chapter presents the concept and the mathematical formulation of resilience-based design (RBD) using a layered approach which is based on the combination of seven aspects of community resilience. A performance evaluation matrix for defining the performance limit threshold of RBD is also presented.

Preparedness and resilience are essential concepts and they are also spreading in the engineering community. The following sections of this chapter provide general information on the state-of-the-practice in performance-based earthquake engineering (PBEE) and the framework for on-going and future developments of performance-based design (PBD) towards RBD. The main aspects of the RBD framework that might prove useful in the development of resilience-based guidelines are identified.

11.2 Development of performance-based design (PBD)

The main equation of PBD is based on the total probability theorem disaggregating the problem into several interim probabilistic models (namely seismic hazard, demand, capacity and loss models) and it is defined as follows (Cornell and Krawinkler, 2000; Krawinkler and Miranda, 2004):

$$\underbrace{\lambda(dv)}_{\text{Seismic risk}} = \underbrace{\iint G(dv|dm)}_{\text{Loss analysis}} \underbrace{dG(dm|edp)}_{\text{Damage analysis}} \underbrace{dG(edp|im)}_{\text{Response analysis}} \underbrace{d\lambda(im)}_{\text{PSHA}} \quad [11.1]$$

where im = intensity measure (e.g. spectral acceleration, duration, etc.); dm = damage measure (e.g. physical condition and consequences/ramifications); edp = engineering demand parameters (e.g. peak/residual drift ratio, acceleration, local indices, etc.); dv = decision variable (e.g. loss, functionality, downtime, casualties, etc.); $\lambda(dv)$ = mean annual frequency of occurrence of a decision variable dv ; $G(a|b)$ is the conditional probability of exceedance $a > a_0$ given b . Each component of Equation (11.1) needs to be determined probabilistically. Currently, the method has been implemented in ATC-58 (2011) and ATC-63 (FEMA_P-695 (2011)), and also in other countries. For instance, in China, PBD has been added to the new version of the code 'Seismic Design for Building Structures' (GB50011-2010), to design tall buildings and innovative systems, while the majority of buildings could be designed with traditional response spectrum analysis.

There are several aspects that are not covered by the current PBD:

- portfolio assessment;
- community assessment.

The concept of PBD can be applied to describe the behaviour of a single building or structure, but the performance of an individual structure is governed not only by its own performance, but interacts extensively with the performance of other entities within the same community. A clear example of these interdependencies between the building and the community is a hospital, which will not be able to perform without electricity and water even if the structure has no structural damage. Another example of the limitations of PBD was demonstrated in the 2009 L'Aquila earthquake, during which the small town of Castelnovo was completely destroyed, except a single housing unit that was standing after the earthquake and suffered minor damage (Cimellaro *et al.*, 2010a). According to PBD the building would satisfy the requirements, but in RBD, the housing unit would not be operational, because it is not able to interact with other entities inside the same community (e.g. a housing unit with minor structural damages without water, electric power and roads connecting it to the rest of the world can satisfy the PBD requirements, but it will not satisfy the RBD requirements). The reason for these limitations is because PBEE in its original formulation was never intended to be applied at a community level; rather, it was developed to permit engineers to better match the building performance to expectations of the owner or architect, which usually require more than the fundamental goal of life safety.

11.3 Towards resilience-based design (RBD)

Nowadays, designers and engineers approach a structure as if it stands alone, without considering the interaction with the community, which

instead should be considered as an integrated part of the design process. There is now a new fundamental way of looking at all problems. The building is not considered alone, but as a *group of buildings* using the ‘portfolio approach’ which will allow regional loss analysis. This concept is borrowed from the financial industry, where modern portfolio theory (MPT) was developed in the 1950s through the early 1970s and was considered an important advance in the mathematical modelling of finance (Elton and Gruber, 1997). MPT is defined as a theory of investment which attempts to minimize risk for a given level of expected return, by carefully choosing the proportions of various assets. MPT is a mathematical formulation of the concept of *diversification* in investing, with the aim of selecting a collection of investment assets that has collectively lower risk than any individual assets. Analogously, the concept of diversification can be applied in the field of disaster resilience, where the *diversification* in retrofit of different buildings in a given region can increase resilience collectively more than any individual retrofits. In other words, RBD attempts to find a method that addresses the concern of minimizing the risk of low-probability–high-consequence (LPHC) extreme events that can potentially result in multiple losses and fatalities.

MPT models an *asset’s return* as a normally distributed function, defines risk as the standard deviation of return, and models a portfolio as a weighted combination of assets so that the return of a portfolio is the weighted combination of the assets’ returns. Analogously in RBD, the *weight aggregated losses* of the *building portfolio* in a community is equivalent to the portfolio asset’s return in MPT, and the *risk of losses* at the community level is the *aggregated conditional probability of exceeding certain damage D* of the building portfolio. Furthermore the *conditional probability of exceeding certain damage* of the building portfolio is a weight combination of the *conditional probabilities of exceeding certain damage* for individual units. In other words, at the community level, analytically the global vulnerability of a community P_{COM} is a weight combination of conditional probabilities associated to a j th unit and it is given by the following equation:

$$P_{\text{COM}}(D/H) = \sum_j w_j P_j(D/H) \quad [11.2]$$

where D = damage state; H = hazard level; $P_j(\)$ = is the probability of exceeding a certain damage level associated with the j th unit; and w_j = weight parameter.

The risk at the community level $Risk_{\text{COM}}$ is a weight combination of aggregated conditional probabilities:

$$Risk_{\text{COM}} = \sum_H \sum_D P_{\text{COM}}(L/D) P_{\text{COM}}(D/H) \lambda_H \quad [11.3]$$

where L = exposure level of a given community; λ_H = earthquake hazard level of a given community. For instance, in RBD group of buildings having

different vulnerabilities $P_f(D/H)$ (because they belong to different structure topologies, they are retrofitted differently, etc.), might have reduced losses at the community level with respect to group of buildings retrofitted with the same strategy, therefore having the same vulnerability level.

Another advantage of RBD with respect to PBD is that it considers the community as a complex system where losses as well as the recovery process are coupled dimensions involving several parameters that are not only engineering parameters, such as drift and accelerations, but also other parameters, such as socio-economic gender, age of the population, etc. In addition, the recovery process becomes an integral part of the RBD process, so it should be planned upfront.

11.3.1 RBD in structures

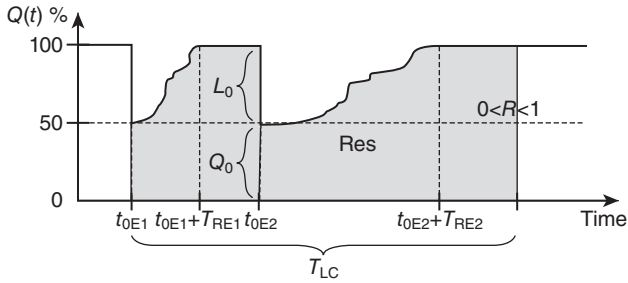
In the last decade, earthquake engineers have given more attention to deformations during their analysis, and life safety, while less attention has been given to socio-economic parameters. Nowadays, attention is shifting toward the necessity to develop a damage-free structure using risk assessment tools which should develop more robust structures against uncertainties. Shorter recovery processes are possible at the building level if the structural damage is minimal; otherwise it might take months to recover. One of the options in order to achieve more resilient structures in face of an earthquake is for example providing them with advanced technologies such as self-centring capabilities with minimum residual deformations which will allow a faster recovery process (e.g. Christopoulos and Filiatrault, 2006).

11.3.2 Definition of resilience

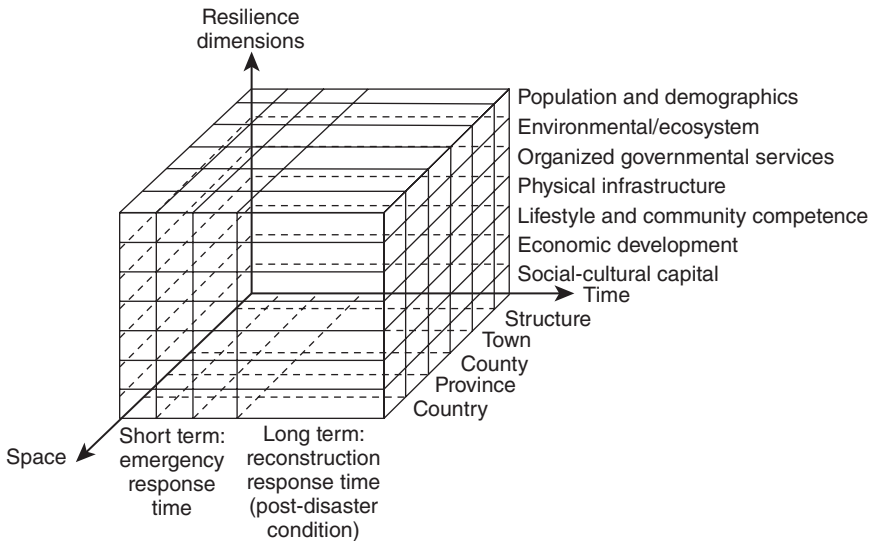
While the general definition of resilience is provided in the introduction of this chapter, the index is illustrated graphically in Fig. 11.1 as the normalized shaded area underneath the functionality function of a system $Q(t)$. Figure 11.1 depicts this as a non-stationary stochastic process and each ensemble is a piecewise continuous function. Analytically, resilience is defined as

$$R(\vec{r}) = \int_{t_{OE}}^{t_{OE} + T_{LC}} Q_{TOT}(t)/T_{LC} dt \quad [11.4]$$

where $Q_{TOT}(t)$ is the global functionality of the region considered; T_{LC} is the control time of the period of interest; $\vec{r}$ is a position vector defining the position P in the selected region where the resilience index is evaluated (Cimellaro *et al.*, 2009, 2010a,c). The community functionality is the combination of all functionalities related to different facilities, lifelines, etc., for the case when physical infrastructure resources and services are considered.



11.1 Resilience (Cimellaro *et al.*, 2010a).



11.2 Spatial and temporal dimension of RBD using PEOPLES approach.

11.3.3 Spatial vs temporal scale of community resilience

Resilience can be considered as a dynamic quantity that changes over time and across space. It can be applied to engineering, economic, social and institutional infrastructure, and it can be used for various geographic scales. The first step to quantify R is to define the *spatial scale* (e.g. building, structure, community, city, region) of the problem of interest (Fig. 11.2). It is important to mention that the entire recovery process is affected by the *spatial scale* of the disaster. Huge disasters will need a longer recovery process. The *spatial scale* will also be used to define the performance measures for the global functionality of the system. The second step is to define

the *temporal scale* (short-term emergency response, long-term reconstruction phase, mid-term reconstruction phase, etc.) of the problem of interest. The selection of the control period T_{LC} will affect the R ; therefore, when comparing different scenarios the same control period should be considered.

11.3.4 The seven dimensions of community resilience

In order to emphasize the primary role of the human system in community sustainability, the acronym ‘PEOPLES’ (Renschler *et al.*, 2010, 2011) has been adopted to describe a framework that is built on and expands previous research at the Multidisciplinary Center of Earthquake Engineering Research (MCEER). This framework linked several previously identified resilience characteristics (technical, organizational, societal and economic) and resilience attributes (r4: *robustness, redundancy, resourcefulness and rapidity*) (Bruneau *et al.*, 2003; Bruneau and Reinhorn, 2007; Cimellaro *et al.*, 2010b). These are the four attributes along which resilience can be improved:

1. *Rapidity* is the ‘capacity to meet priorities and achieve goals in a timely manner in order to contain losses and avoid future disruption’ (Bruneau *et al.*, 2003). Mathematically it represents the slope of the functionality curve given in Fig. 11.1.
2. *Robustness* referring to engineering systems is, ‘the ability of elements, systems or other units of analysis to withstand a given level of stress, or demand without suffering degradation or loss of function’ (Bruneau *et al.*, 2003). It is therefore the *residual functionality right after the extreme event*.
3. In earthquake engineering, *redundancy* is ‘the quality of having alternative paths in the structure by which the lateral forces can be transferred, which allows the structure to remain stable following the failure of any single element’ (FEMA, 2000). In other words, it describes the availability of alternative resources in the recovery process of a system.
4. *Resourcefulness* is ‘the capacity to identify problems, establish priorities, and mobilize resources when conditions exist that threaten to disrupt some element, system, or other unit of analysis’ (Bruneau *et al.*, 2003). This property is difficult to quantify since it mainly depends on human skills and improvisation during the extreme event.

Resourcefulness and *redundancy* are strongly interrelated. For example, resourcefulness can create redundancy that did not exist previously. In fact, one of the major concerns with the increasingly intensive use of technology in emergency management is the tendency to over-rely on these tools, so that if technology fails, or it is destroyed, the response falters. To forestall

this possibility, many planners advocate redundancy. Changes in resourcefulness and redundancy will affect the shape and the slope of the recovery curve and thus the recovery time T_{RE} . It also affects *rapidity* and *robustness*. It is through redundancy and resourcefulness (as means of resilience) that rapidity and robustness (the goals of resilience) of an entire system can be improved. Further details about the description of these attributes can be found in Cimellaro *et al.* (2010b).

PEOPLES incorporates MCEER's definitions of service functionality, and its components (assets, services, demographics) and parameters influencing their integrity and resilience.

The seven dimensions of the RBD are the following:

1. **P**opulation and demographics;
2. **E**nvironment/ecosystem;
3. **O**rganized government services;
4. **P**hysical infrastructure;
5. **L**ifestyle and community competence;
6. **E**conomic development;
7. **S**ocial-cultural capital.

Table 11.1 shows the complete list of components and sub-components of the 'PEOPLES framework'.

The first dimension *Population and demographics* is used to describe and differentiate communities using, for example, the median income and age distribution which might be critical for understanding its economic, health and potential resilience. One measure of functionality of population and demographics (Q_p) within a given community can be quantified by using the social vulnerability index (SoVI) proposed by Cutter (1996). Social vulnerability (a counterpart of social resilience) is defined as the inability of people, organizations and societies to withstand adverse impacts from multiple stressors to which they are exposed.

The *Environmental/ecosystem* dimension is typically measured by the amount of disturbance an ecosystem can absorb without drastically altering its functions, processes and structures, or by the ability of an ecosystem to cope with disturbance. The *Organized governmental services* dimension includes traditional legal and security services such as police, emergency, fire departments and, increasingly, the military. In this dimension, are also included the services provided by public health and hygiene departments as well as cultural heritage departments. Each of these organized government services plays a key role in sustaining communities both before and after extreme events.

The *Physical infrastructure dimension* focuses on a community's built environment. It incorporates both facilities and lifelines and different performance indicators. *Community competence dimension* deals with

community action, critical reflection and problem solving skills, flexibility and creativity, collective efficacy, empowerment and political partnerships (Norris *et al.*, 2008). *Economic development dimension* includes both the static assessment of a community's current economy (economic activity) and the dynamic assessment of a community's ability to continuously sustain economic growth (economic development).

Social/cultural capital dimension incorporates several subcategories, including education services, child and elderly services, cultural and heritage services, and community participation. Measuring social/cultural capital requires acquisition of tallies, such as the number of members belonging to various civil and community organizations. It also requires surveys of community leaders and their perceptions (e.g. quality of life surveys).

Further details about the description of each one of these dimensions can be found in Renschler *et al.* (2010, 2011). The general framework at the community level is described by equations below, where for each dimension a performance indicator and/or functionality is defined by combining with different functionality dimensions:

$$Q_{TOT}(t) = Q_{TOT}(Q_P, Q_{Env}, Q_O, Q_{Ph}, Q_L, Q_{Eco}, Q_S) \quad [11.5]$$

where Q_{TOT} = global functionality; and Q_x = functionality of each of the seven dimensions. Within each dimension, functionality is defined as a combination of functionalities of their respective subsystems. For example, the functionality of the physical infrastructure Q_{ph} is defined as follows:

$$Q_{Ph}(t) = Q_{Ph}(Q_{Hosp}, Q_{Ele}, Q_{Road}, Q_{Water}, \dots) \quad [11.6]$$

where Q_{hosp} = functionality of health care facilities; Q_{Ele} = functionality of the electric network; Q_{Road} = functionality of the road network; Q_{Water} = functionality of the water network, etc. Once the geographic scale is defined, it is possible to plot the global functionality Q_{TOT} over the region of interest in a contour plot at a given instant of time t , so *time-dependent functionality maps* of the region can be obtained. When also the temporal scale is defined through the control time T_{LC} , then the *resilience contour map* of the region of interest can be plotted (e.g. Fig. 11.3; Fig. 11.15). The *resilience contour map* is obtained by integrating functionality maps over time using Equation (11.5), therefore they will be *time independent*, but they will vary in space from point to point in the selected region. Finally, the community resilience index R_{com} is given by the double integral over time and space as follows:

$$R_{com} = \int_{A_C} R(\bar{r})/A_C \, d\mathbf{r} = \int_{A_C} \int_{t_{OE}}^{t_{OE}+T_{LC}} Q_{TOT}(t)/(A_C T_{LC}) \, dt \, d\mathbf{r} \quad [11.7]$$

where A_c is the total area of the selected region. For each dimension, a contour plot can be determined and combined using a layered approach as

Table 11.1 Complete list of components and subcomponents of PEOPLES framework

1 Population and demographics		
(a) Distribution/density		
(i)	Urban	
(ii)	Suburban	
(iii)	Rural	
(iv)	Wildland	
(b) Composition		
(i)	Age	
(ii)	Gender	
(iii)	Immigrant status	
(iv)	Race/ethnicity	
(c) Socio-economic status		
(i)	Educational attainment	
(ii)	Income	
(iii)	Poverty	
(iv)	Home ownership	
(v)	Housing vacancies	
(vi)	Occupation	
4 Physical infrastructure		
(a) Facilities		
(i)	Residential	
(1)	Housing units	
(2)	Shelters	
(ii) Commercial		
(1)	Distribution facilities	
(2)	Hotels	
	– accommodations	
(3)	Manufacturing facilities	
(4)	Office buildings	
(iii) Cultural		
(1)	Entertainment venues	
(2)	Museums	
(3)	Religious institutions	
(4)	Schools	
(5)	Sports/recreation venues	
(b) Lifelines		
(i)	Communications	
(1)	Internet	
5 Lifestyle and community competence		
(a) Collective action and decision making		
(i)	Conflict resolution	
(ii)	Self-organization	
(b) Collective efficacy and empowerment		
(c) Quality of life		
6 Economic development		
(a) Financial services		
(i)	Asset base of financial institutions	
(ii)	Checking account balances (personal and commercial)	
(iii)	Consumer price index	
(iv)	Insurance	
(v)	Number and average amount of loans	
(vi)	Number of bank and credit union members	
(vii)	Number of banks and credit unions	
(viii)	Savings account balances (personal and commercial)	
(ix)	Stock market	
(b) Industry – Employment – Services		

2 Environmental/ecosystem

- (a) Water quality/quantity
- (b) Air quality
- (c) Soil quality
- (d) Biodiversity
- (e) Biomass (vegetation)
- (f) Other natural resources

3 Organized governmental services

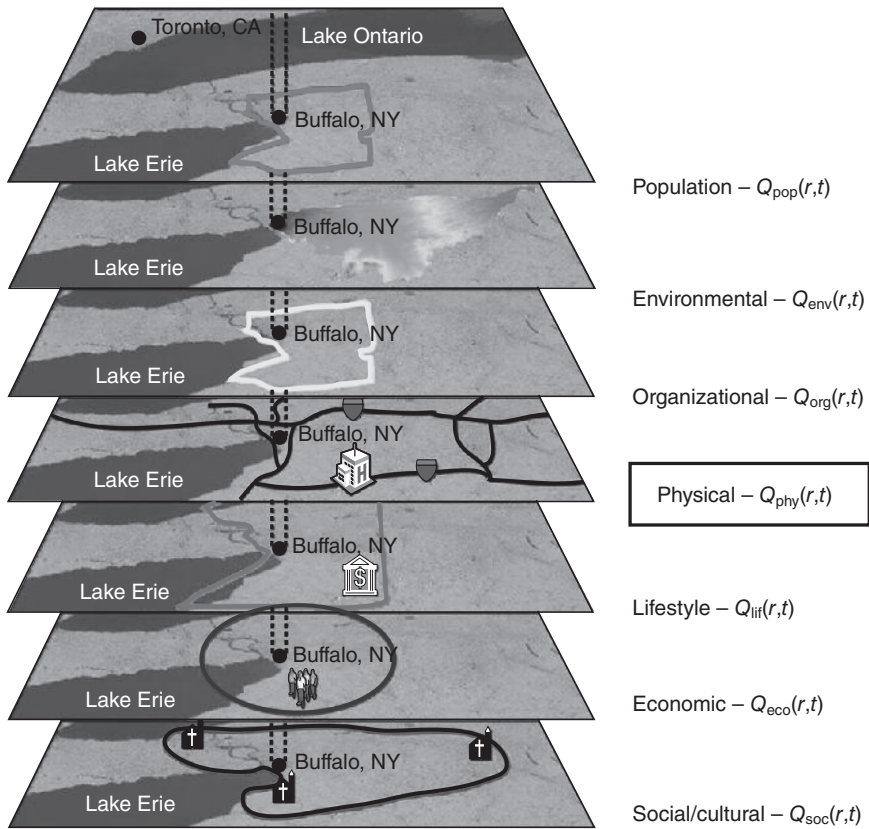
- (a) Executive/administrative
 - (i) Emergency response and rescue
 - (ii) Health and hygiene
- (b) Judicial
- (c) Legal/security

- (2) Phones
- (3) TV
- (4) Radio
- (5) Postal
- (ii) Health care
 - (1) Acute care
 - (2) Long-term acute care
 - (3) Primary care
 - (4) Psychiatric
 - (5) Specialty
- (iii) Food supply
- (iv) Utilities
 - (1) Electrical
 - (2) Fuel/gas/energy
 - (3) Waste
 - (4) Water
- (v) Transportation
 - (1) Aviation
 - (2) Bridges
 - (3) Highways
 - (4) Railways
 - (5) Transit
 - (6) Vehicles
 - (7) Waterways

- (i) Agriculture
- (ii) Construction
- (iii) Education and health services
- (iv) Finance, insurance and real estate
- (v) Fortune 1000
- (vi) Fortune 500
- (vii) Information, professional business, other
- (viii) Leisure and hospitality
- (ix) Manufacturing
- (x) Number of corporate headquarters
- (xi) Other business services
- (xii) Professional and business services
 - (1) Employment services
 - (a) Flexibilities
 - (b) Opportunities
 - (c) Placement
 - (2) Transport and utilities
 - (3) Wholesale and retail
- (c) Industry – production
 - (i) Food supply
 - (ii) Manufacturing

7 Social/cultural capital

- (a) Child and elderly services
 - (b) Commercial centres
 - (c) Community participation
 - (d) Cultural and heritage services
 - (e) Education services
 - (f) Non-profit organizations
 - (g) Place attachment
-

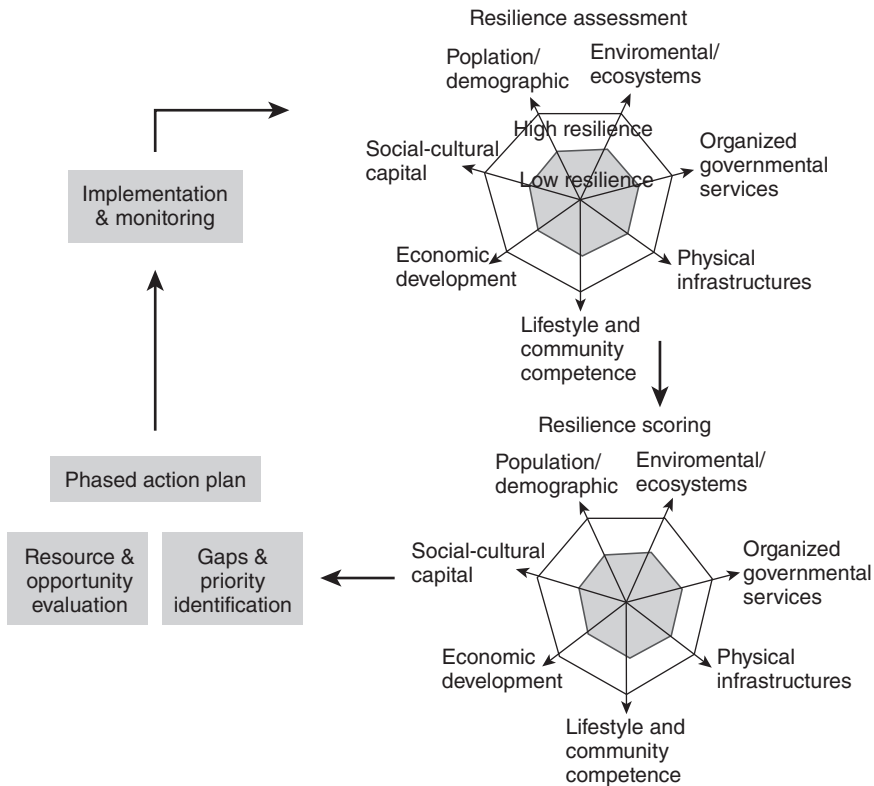


11.3 Layer model of PEOPLES.

shown in Fig. 11.3. Then a radar graph can be plotted and the area will determine the final value of the resilience score (Fig. 11.4) for the region of interest. This will identify gaps as well as priority actions, which will enter in the decision process.

In summary, a schematic step-by-step procedure of the MCEER methodology described in Fig. 11.5 is the following:

1. Define extreme event scenarios (e.g. PSHA and ground motion selection).
2. Define the system model.
3. Evaluate the response of the model.
4. Compute performance measures (e.g. losses, recovery time, functionality, resilience).
5. Identify remedial mitigation actions (e.g. advanced technologies) and/or resilience actions (e.g. resourcefulness, redundancy, etc.).

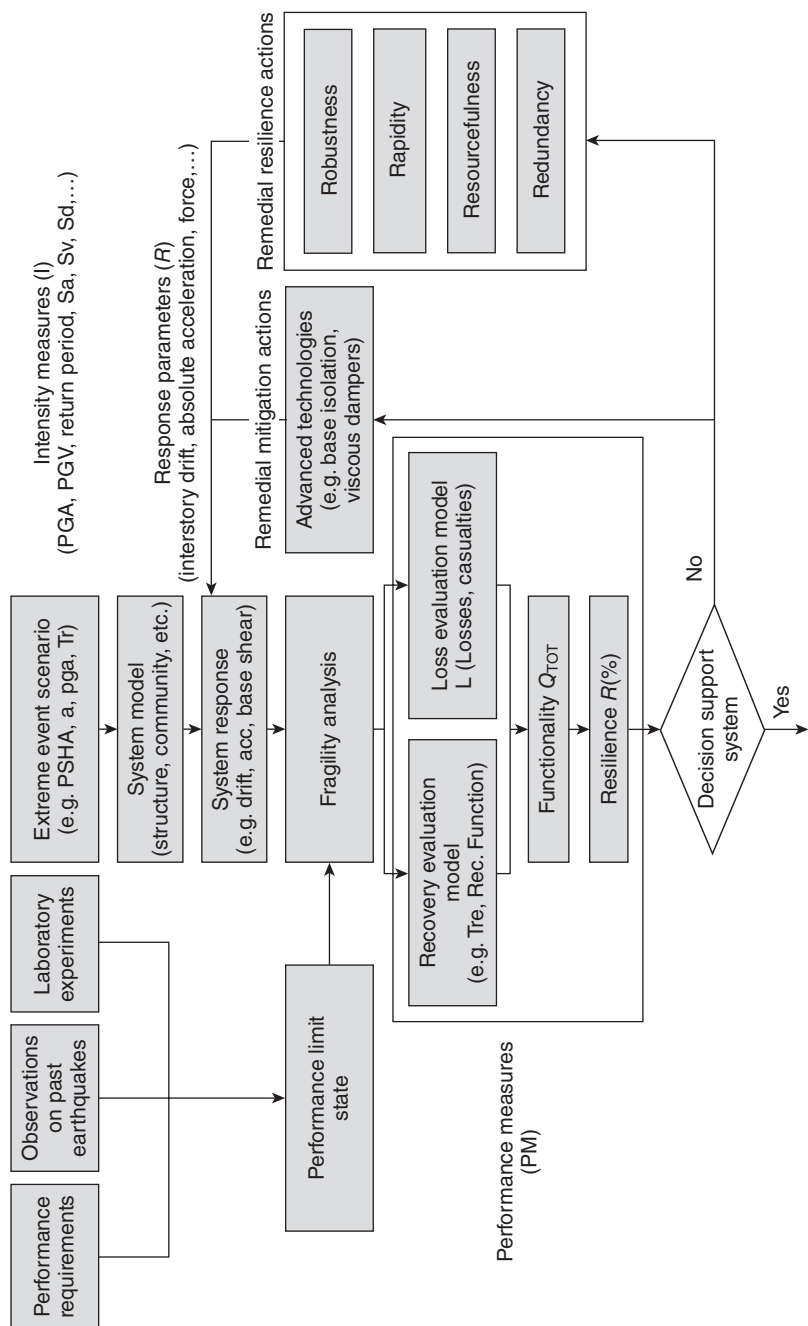


11.4 Resilience scoring using PEOPLES methodology.

This design approach has analogies with the feedback loop taken from control theory. The same framework can be used for a region as well as a single structure (e.g. hospital). In this case, functionality reduces to the functionality of a single hospital Q_{Hosp} which can be evaluated for example with the procedure described in Cimellaro *et al.* (2011), where the waiting time of a patient before receiving assistance is the main parameter of response to measure resilience. The hospital performance is described using a double exponential function, called metamodel which is able to estimate the hospital capacity and the dynamic response in real time incorporating the influence of damage of structural and non-structural components.

11.3.5 Recovery models

In general, the performance measure of a community and a system in general during transient analysis is a function of time t and other



11.5 MCEER centre methodology for RBD based on control (feedback loop) approach.

parameters that depend on the type of a community considered. Therefore at time t after the crisis, functionality is given by

$$Q(t) = f(t, x_1, \dots, x_n) \quad [11.8]$$

where $x_1, \dots, x_n$ are the parameters involved in describing the recovery model. Several models have been presented in Cimellaro *et al.* (2010a) to describe the recovery function which can be either *empirical* or *analytical*, depending on the source of data and the type of analysis.

Empirical recovery functions are based on test or field data interpretation and engineering judgment. They can be built using the maximum likelihood method based on data reported from past extreme events as well as Monte Carlo simulations of specified community models. Since the complexity of the problem changes case by case, no specific model is presented in this part.

Analytical recovery functions are developed from community response data obtained through analysis of the system using numerical simulations. For example, for the case of earthquake events, they can be obtained from nonlinear time history analysis, response spectral analysis, etc.

Since the recovery process is characterized by uncertainties, the parameters considered in the model are modelled as random variables in order to quantify the uncertainties in the system. These uncertainties can be divided in aleatoric and epistemic uncertainties (Ang and Tang, 2007).

Several models can be fitted to the observed data and, subsequently, model selection can be carried out using as goodness of fit measure, such as the r^2 value. The essential requirement of the analytical recovery models is simplicity, therefore the model should be selected so that it is easy to fit to real or numerical observation data and the number of parameters involved should be as low as possible. Five different recovery models are reported below, grouped according to the two control periods (*short term* vs *long term*). *Long-term recovery models* are used when the reconstruction phase needs to be modelled, while *short-term recovery models* are used when the emergency phase after the extreme event needs to be focused upon.

Several *long-term recovery models* are proposed in Cimellaro *et al.* (2010a). They can be grouped according to the number of parameters (one, two or three parameters). Complex recovery models with more parameters can be proposed, but simpler mathematical models have benefits over more complex ones. They have fewer unknown parameters, and thus it is easier to fit to data (fewer experiments are needed). There is also less chance of ‘overfitting’. With more free parameters, a model can be made to fit any data; however, at best the exercise is little more than curve fitting (with little meaningful understanding gained), while at worst the model may give an overconfidence in its predictive ability.

The simplest recovery model is the *uniform cumulative distribution* recovery function (also known as the *linear model*). This model is usually adopted when there is no information regarding the preparedness, resources available, societal response, etc.

$$Q(t) = Q_0 + F(t/t_0, t_0 + T_{RE}) \cdot [Q_R - (Q_0 - L_0)] \quad [11.9]$$

where Q_0 is the initial functionality after the drop; L_0 is the initial total loss of functionality after the drop; Q_R is the residual functionality after the recovery process ends (see Fig. 11.1); and $F(t/t_0, t_0 + T_{RE})$ is the uniform cumulative distribution function (CDF) which is given by

$$F(t/t_0, t_0 + T_{RE}) = \frac{(t - t_0)}{T_{RE}} I(t_0, t_0 + T_{RE}) \quad [11.10]$$

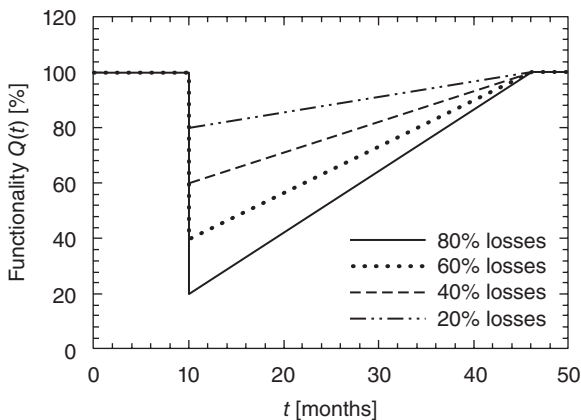
where $I(t_0, t_0 + T_{RE})$ is the interval step function. The model is characterized by only one parameter (Fig. 11.6) which defines the slope of the curve and represents *rapidity* (Cimellaro *et al.*, 2010a). The model can also be generalized by dividing the recovery process in several time intervals using a *multilinear model* that is given by:

$$Q(t) = Q_i + \sum_i H(t - t_i) \frac{(t - t_i)}{(t_{i+1} - t_i)} (Q_{i+1} - Q_i) \quad [11.11]$$

where Q_i is the residual functionality at the step i and Q_{i+1} is the residual functionality at the step $i + 1$; $H(\cdot)$ is the Heaviside step function.

Alternatively, *lognormal cumulative distribution* (CDF) recovery function, can be adopted, having three parameters (L_0, θ, β), and it is given by

$$Q(t) = Q_0 + F(t/\theta, \beta) \cdot [Q_R - (Q_0 - L_0)] \quad [11.12]$$



11.6 Uniform cumulative distribution recovery function.

where

$$F(t/\theta, \beta) = \frac{1}{\beta\sqrt{2\pi}} \int_{-\infty}^t \frac{e^{\frac{[\log(x)-\theta]^2}{2\beta^2}}}{x} dx \quad [11.13]$$

This model combines both the exponential recovery model proposed by Kafali and Grigoriu (2005) and the trigonometric recovery model proposed by Chang and Shinozuka (2004). The parameter L_0 in Equation (11.12) can be used to define the initial total loss of functionality after the drop (Fig. 11.7a). The parameter θ can be used to define the time frame (Fig. 11.7b) when the societal response and recovery are driven by lack or limited organization and/or resources. The parameter β defines the rapidity of the recovery process (Fig. 11.7c).

Another recovery model presented in Cimellaro *et al.* (2010a) is the *harmonically over-damped recovery function* having three parameters (L_0 , ω , ζ). It is defined as

$$Q(t) = Q_0 + \left\{ 1 - e^{-\alpha t} \left[\left(\frac{\alpha + \beta}{2\beta} \right) e^{\beta t} + \left(\frac{\beta - \alpha}{2\beta} \right) e^{-\beta t} \right] \right\} [Q_R - (Q_0 - L_0)] \quad [11.14]$$

where L_0 defines the initial total loss of functionality after the drop (Fig. 11.8a); $\alpha = \omega\zeta$; $\beta = \omega\sqrt{(\zeta^2 - 1)}$; ω and ζ are related to the rapidity dimension ($\zeta \geq 1$; $\omega \geq 1$). Furthermore, rapidity of recovery increases when either ω increases or ζ reduces as shown in Figs 11.8b,c. For critically damped systems ($\zeta = 1$), placing the same initial condition $Q(0) = 1 - L_0$ and $\dot{Q}(0) = 0$, the solution is given by

$$Q(t) = 1 - L_0 e^{-\omega t} (1 + \omega t) \quad [11.15]$$

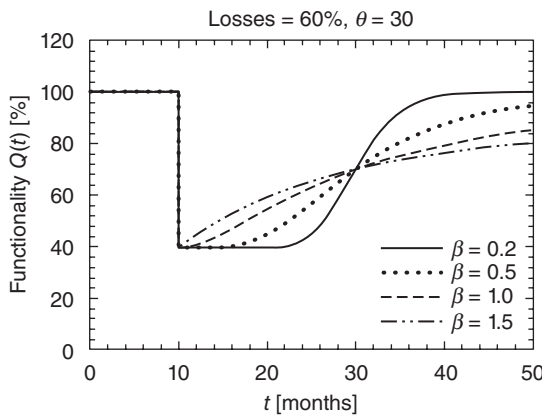
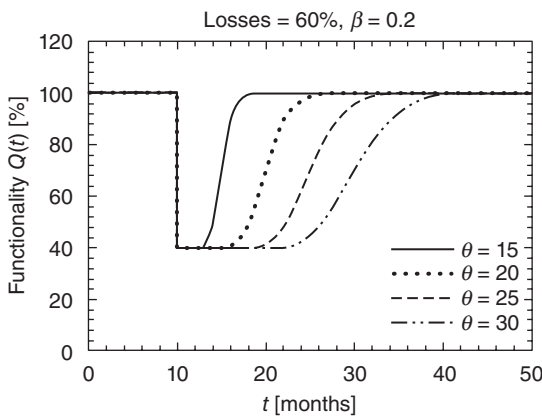
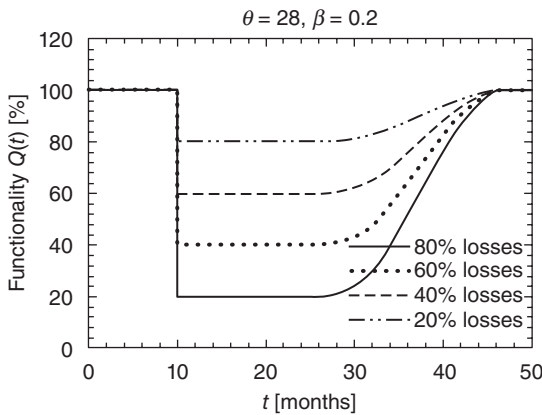
The second group of recovery models is called *short-term recovery models* and instead of using CDF shape models such as in the long term recovery models, they use the probability density functions (PDF) shape models. The simplest recovery model after the linear model proposed in Equation (11.9) is the Rayleigh PDF recovery model, defined as:

$$Q(t) = 1 - L_0 \frac{f(t|b)}{\max(|f(t|b)|)} \quad [11.16]$$

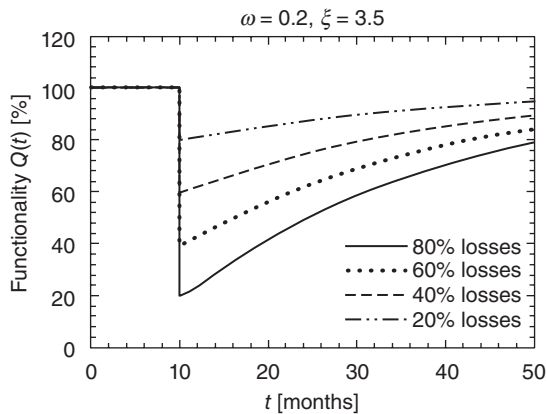
where

$$f(t|b) = \frac{t}{b^2} e^{\left(\frac{-t^2}{2b^2}\right)} \quad [11.17]$$

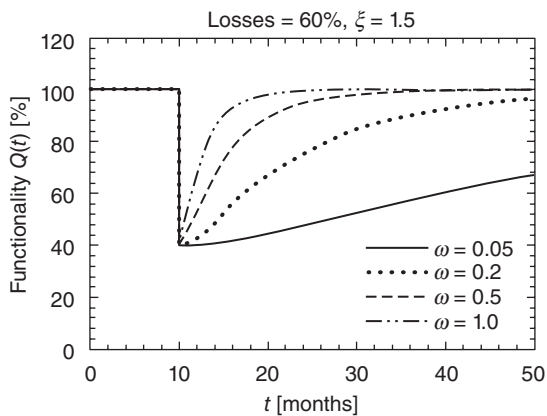
The model is calibrated using two parameters: L_0 is related to the robustness dimension (Fig. 11.9a), while b is related to the rapidity and the delay in the recovery process (Fig. 11.9b).



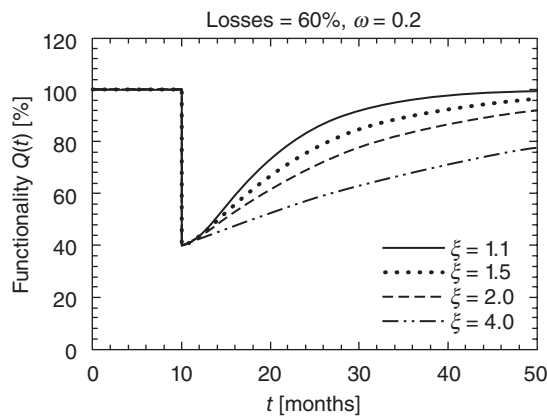
11.7 Lognormal cumulative distribution recovery function.



(a)

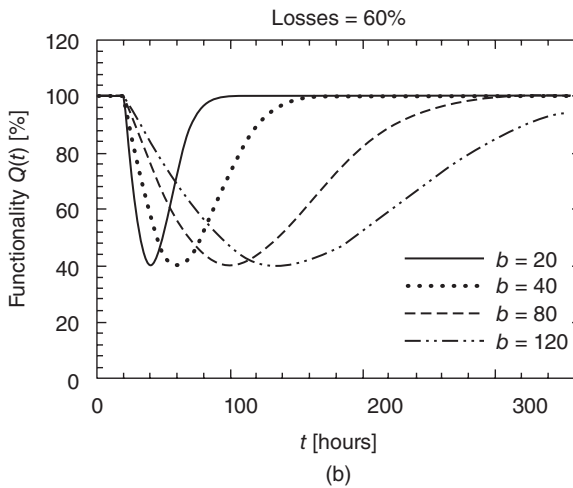
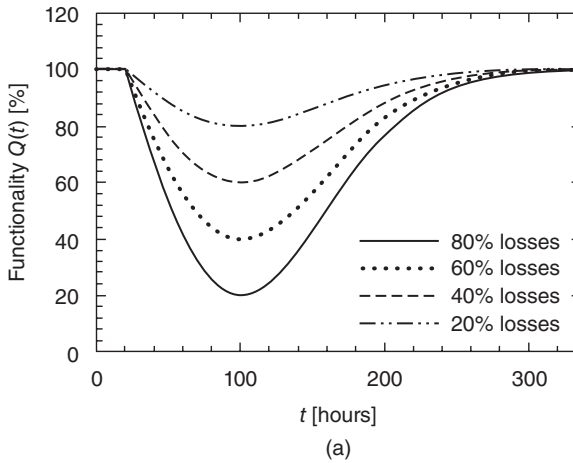


(b)



(c)

11.8 Harmonically over-damped recovery function.



11.9 Rayleigh probability density recovery function.

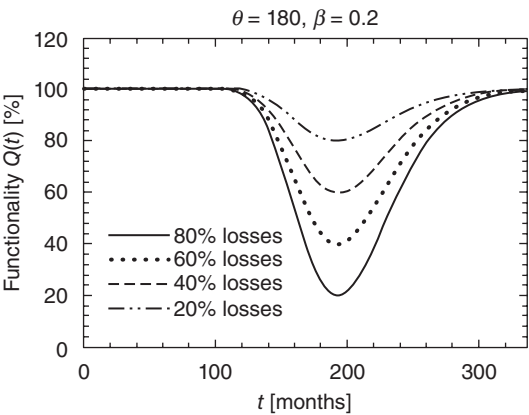
Another model is the *lognormal probability density recovery function* which is given by

$$Q(t) = 1 - L_0 \frac{f(t|\theta, \beta)}{\max(|f(t|\theta, \beta)|)} \quad [11.18]$$

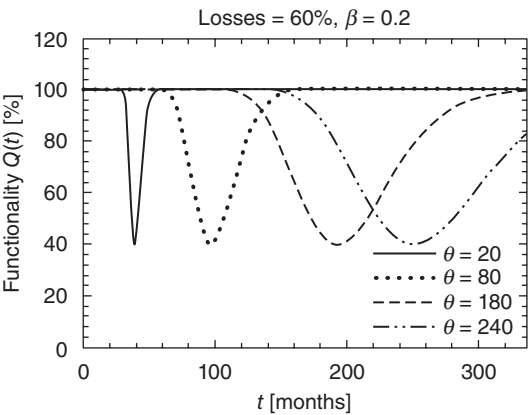
where

$$f(t|\theta, \beta) = \frac{1}{x\beta\sqrt{2\pi}} e^{-\frac{(\log(x)-\theta)^2}{2\beta^2}} \quad [11.19]$$

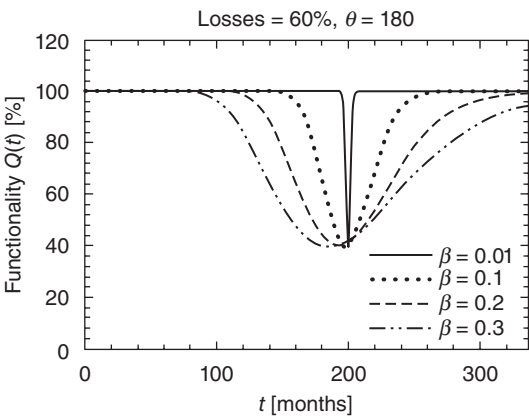
The sensitivity of the three parameters on the recovery process is shown in Fig. 11.10.



(a)



(b)



(c)

11.10 Lognormal probability density recovery function.

In the short-term emergency response, more complex analytical recovery models are available such as the *metamodel* for describing the organizational performance of a hospital facility (Cimellaro *et al.*, 2010b). The model is based on a *double exponential function* and its parameters are calibrated based on simulated data obtained by a discrete event simulation model. The *metamodel* is capable of estimating the hospital capacity and dynamic response in real time, incorporating the influence of structural and non-structural damaged components on the entire organizational model. It is important to mention that the constants in all the models presented can be continuously updated as soon as more data are available using a Bayesian approach.

11.3.6 Uncertainties in RBD

Either a deterministic or probabilistic approach can be used within the RBD methodology; the latter approach is preferred when a particular level of confidence of achieving performance objective is of interest. Five random variables are involved in the probabilistic description of the resilience index when uncertainties are included and variables are dependent. The joint PDF of resilience, intensity, response, performance and recovery is given by the following expression:

$$f_{R,T_{RE},Q,X,I}(r, t_{RE}, q, x, i) = f_{R,T_{RE},Q,X,I}(r|t_{RE}, q, x, i) \cdot f_{T_{RE},Q,X,I}(t_{RE}|q, x, i) \cdot f_{Q,X,I}(q|x, i) \cdot f_{X,I}(x|i) \cdot f_I(i) \quad [11.20]$$

where I = intensity measures; X = response measures; Q = performance measures; T_{RE} = recovery time measures; R = resilience index. The marginal PDF of the resilience index is given by:

$$f_R(r) = \int \int \int \int f_{R,T_{RE},Q,X,I}(r, t_{RE}, q, x, i) dt_{RE} \cdot dq \cdot dx \cdot di \quad [11.21]$$

Therefore the expected value of the resilience index is given by

$$m_r = E\{R\} = \int_{-\infty}^{\infty} r \cdot f_{R,T_{RE},Q,X,I}(r, t_{RE}, q, x, i) \cdot dr \quad [11.22]$$

11.3.7 Communicating risk in RBD

Even if the most general and correct formulation to evaluate performance in RBD is the probabilistic approach described in Equations (11.20), (11.21) and (11.22), engineers need to know which measures of risk are most meaningful or relevant to decision makers, and then be able to communicate

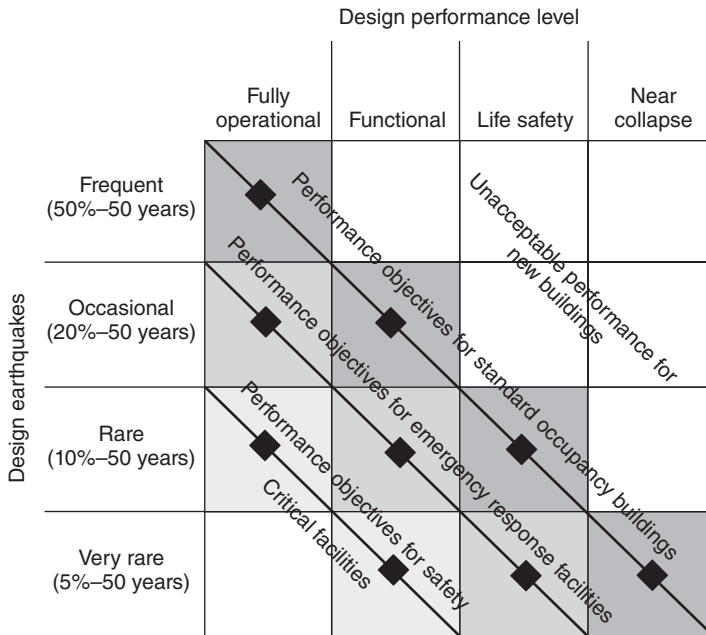
those risks, as well as the costs and benefits of mitigation, in concise, credible, meaningful terms. Keller *et al.* (2006) have shown that when problems are formulated in terms of frequencies rather than probabilities, the perceived threat of the risk is increased.

The probabilistic approach described in the previous paragraph is comprehensive and general, but the information provided to the public (decision makers, politicians, etc.) should be deterministic (scenario basis or event), because it is simpler and easier to understand. In communicating risk effectively, public has difficulty in interpreting probabilistic information (Patt and Schrag, 2003). In fact, according to Kahneman and Tversky (2000), small probabilities (which are frequently associated with natural hazard events) are often underestimated. 'By eliminating probability, which is a confusing concept for a lot of people, the [risk] becomes way more impactful for an average person'. Many authors believe the scenario approach may also impact the emotions associated with an event.

11.3.8 Definitions of resilience performance levels

The objective of PBEE is to design, construct and maintain facilities with enhanced damage control. A comprehensive document was prepared by the SEAOC (1995) Vision 2000 Committee (2000) that includes interim recommendations. The performance design objectives couple expected or desired performance levels with levels of seismic hazard as illustrated by the performance design objective matrix shown in Fig. 11.11. According to the ATC 58 (2011), the performance assessment types can be classified into: (i) intensity-based; (ii) scenario-based; or (iii) time-based. PBD levels focus on the performances a building can achieve during the shaking, and are associated with engineering demand parameters such as deformation. More recently, SPUR (Maffei, 2009), which is the San Francisco Planning and Urban Research Association, introduced other definitions of performance levels for physical infrastructure based on recovery target states which take into account the safety as well as the recovery time. Five performance measures for buildings have been identified: (i) *Safe and operational*; (ii) *Safe and usable during repair*; (iii) *Safe and usable after repair*; (iv) *Safe but not repairable*; (v) *Unsafe*. However, no clear quantification of these performance levels has been provided so far. In Japan, using public interviews, the accepted downtime and performance goals for bridges have been evaluated (Fig. 11.12), attempting to find what are the desired performance goals expected by citizens after an extreme event like an earthquake. This information is very useful to define the boundaries of different resilience performance levels.

The newly proposed resilience performance levels (RPL) focus on building performance after the ground shaking stops, and recognize the



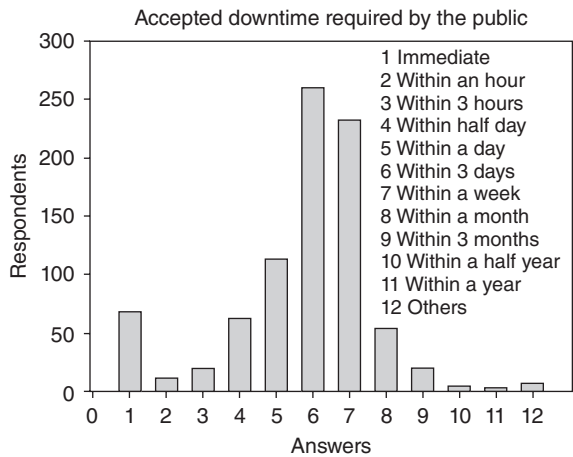
11.11 Recommended seismic performance objectives for buildings (SEAOC, 1995).

importance of the recovery time T_{RE} in the assessment of the RPLs for structures and communities (Uma *et al.*, 2006). A two-dimensional performance domain consisting of performance levels $PL(i, j)$, defined by the combination of functionality (index j) and recovery time (index i) is proposed. By accounting for the effect of the temporal dimension, a three-dimensional performance matrix (Fig. 11.13) can be visualized as a set of predefined joined performance domains ('masks') for different seismic intensity levels IM and different RPLs.

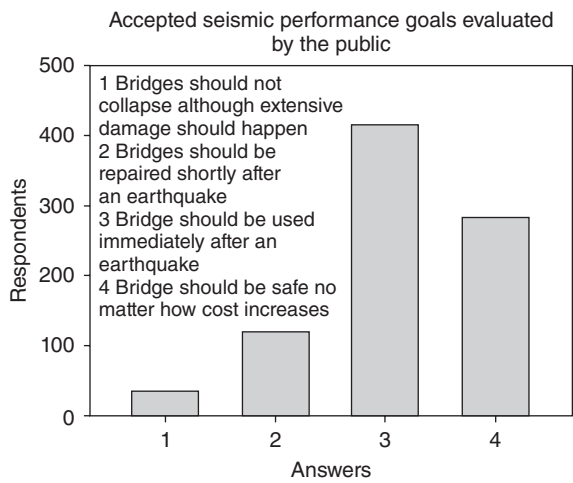
11.4 Case studies

The entire framework described in this chapter has been implemented in a software which uses a Google Earth graphical interface (Arcidiacono *et al.*, 2011). The software takes into account interdependencies between the road and building networks. Two case studies are considered:

- the old medieval housing units of L'Aquila downtown;
- Treasure Island in San Francisco Bay.

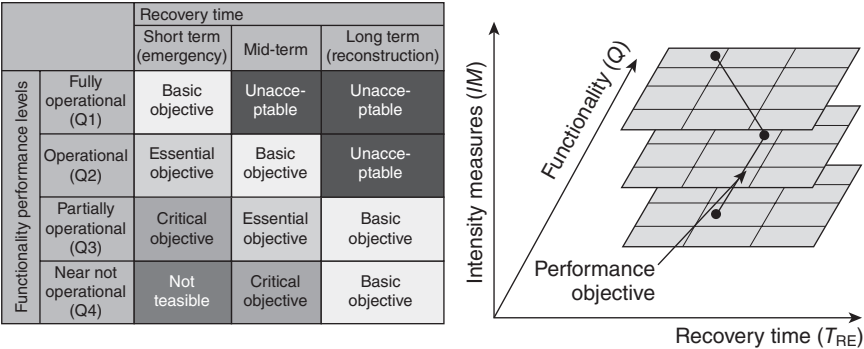


(a)



(b)

11.12 Survey results on the acceptable recovery process.



11.13 Three-dimensional resilience performance objectives matrix for structures, communities, systems, etc.

11.4.1 Old medieval housing units of L'Aquila downtown

The old medieval centre of L'Aquila town, Italy; was subjected to the 2009 Italian earthquake. Twenty-two buildings in Piazza del Duomo and the entire road network have been modelled with realistic features (Figs 11.14a,b) using the data from Open Street Map website (<http://www.openstreetmap.org/>). Most of the buildings selected are residential build-



(a)



(b)

11.14 Selected housing units (a) in l'Aquila downtown and road network (b).

ings, but different materials and class of occupancy are considered. All buildings are located on soil type B. The data provided for each building are: (i) dimensional parameters and location; (ii) physical parameters; and (iii) capacity curves, damping ratio, occupancy class, the total occupancy area, the full replacement costs, the construction building times, the recovery costs, etc. Plan dimensions and location of the housing units are plotted directly by the user in the Google map interface during the construction of the database in the region of interest. Physical parameters are loaded using the classification given in the PEOPLES Resilience Framework and it will be used to build the corresponding fragility curves. The building seismic response is evaluated using the Capacity Spectrum Method (CSM), which is also adopted by the software HAZUS (FEMA, 2005), because it is faster when several building performance need to be evaluated in a given region. Further details about the case study can be found in Arcidiacono *et al.* (2011).

Four different scenarios were considered assuming different recovery actions, based on the number of construction sites working in a day. It is assumed that there is no limit to the economic budget. Functionality as well as resilience indices were evaluated for two different control periods: at two years and at the end of the construction site work. From the 3D histogram plotted on Google Earth (Fig. 11.15), it is possible to see how the damage states in all buildings are distributed in the selected region of interest. Figure 11.15b shows the contour plot of functionality immediately after the disaster. Figure 11.16a shows the resilience index when the control period is two years, while Fig. 11.16b shows the functionality curves for different scenario events; the best scenario is the first one.

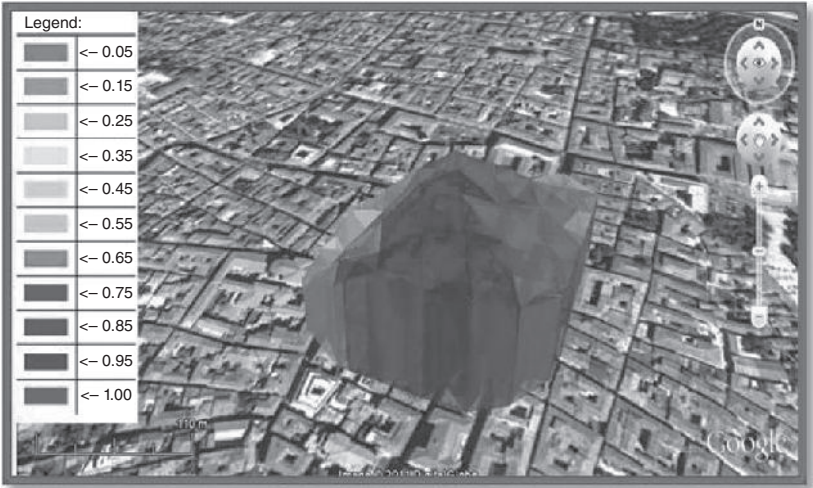
11.4.2 Treasure Island in San Francisco Bay

Twenty-five buildings and the road network in the Treasure Island, which is an island connecting the City of San Francisco with Oakland through the Bay Bridge on Highway 80 (Fig. 11.17), have been considered as a case study. In this example, the interdependencies between the housing units and the road network have been taken into account. If a building is not accessible from the road network after an earthquake, it will not be repaired, remaining its functionality at a reduced state. If a building collapses, the road in the area that is influenced by the collapsed building will lose its functionality.

The earthquake considered as the input to the model corresponds to a return period (T_r) of 2450 years, while the input data related to the road network are taken from Open Street Map database. The main assumptions of the recovery scenario are: (i) no limit on the economic budget (EB); (ii) maximum of three simultaneous starts of construction at building sites



(a)

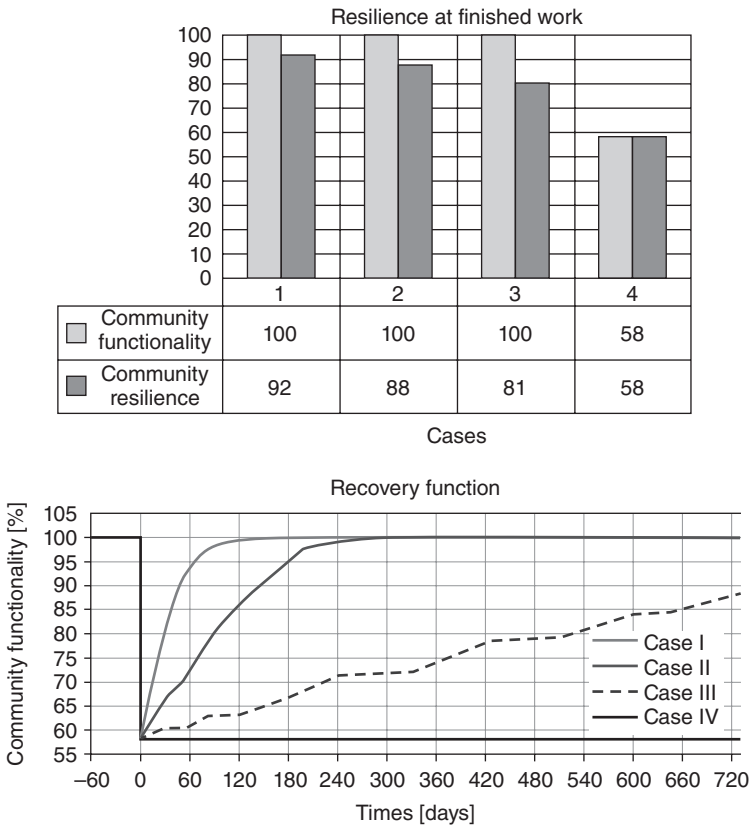


(b)

11.15 Graphical output of the software implementing the MCEER methodology (© 2011 Google).

(CSS); and (iii) maximum of three construction sites per day inside the selected area (CS).

Figure 11.18a is a 3D histogram, plotted on Google Earth, of the discrete probabilities of damage states for buildings and road network (no damage in black, collapse in white). Figure 11.17b shows the functionality in all buildings and in the entire road network immediately after the disaster.



11.16 Resilience vs functionality for different scenario events.

Buildings and districts inside the Island have zero functionality, because with the collapse of the bridges the entire Island is not accessible – so all building units and roads are unusable. The functionality curves of the physical infrastructure dimension are plotted in Fig. 11.18b. The global functionality Q_{ph} has been evaluated with equal weight coefficients for the road network and the housing units. At the 40th day, when there is the recovery of the first bridge that connects the Island with the mainland, the global functionality curve of the physical infrastructure dimension has a gap, because the roads and the building units can be reused and repaired. The recovery time T_{EW} of the selected community is equal to 3.19 years; while the resilience indices are 74.34% and 97.52% for buildings and road network, respectively, while the global resilience index combining both buildings and road network is 85.93% when equal weight is given to both systems.

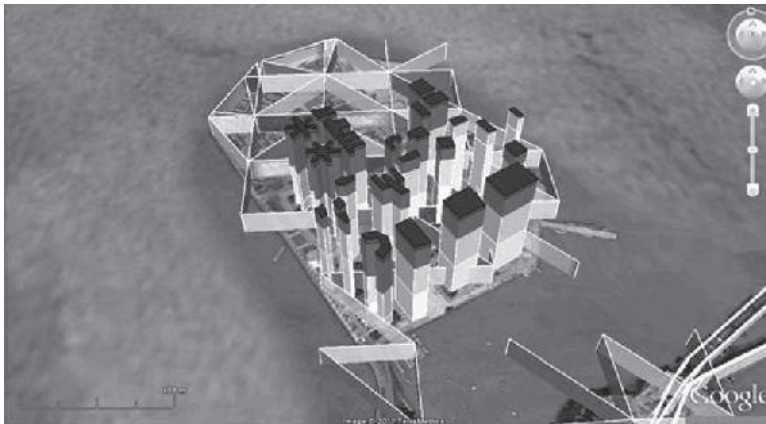


(a)

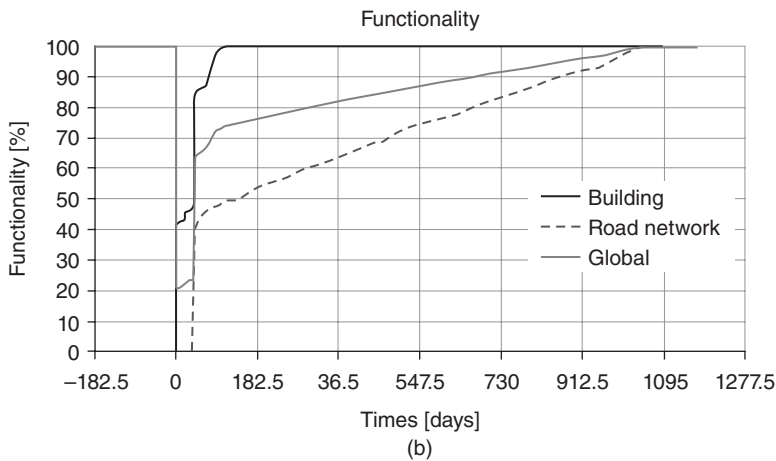


(b)

11.17 (a) Buildings in Treasure island in San Francisco bay. (b) Functionality after the risk assessment.



(a)



(b)

11.18 (a) Discrete probability damage state for buildings. (b) Combined functionality of road network and buildings.

11.5 Conclusions

After many recent extreme events worldwide, the international community has become conscious that resilience is the key to describing earthquake engineering performance. A new general design methodology is developed in this chapter called ‘resilience-based design’ (RBD). This can be considered as an extension of PBD, which is only a part of the total ‘design effort’. The goal of RBD is to make individual structures and communities as ‘resilient’ as possible, developing technologies and actions that allows each structure and/or community to regain its function as promptly as possible.

Even if the performance evaluation of an individual structure is the engineers' goal, the level of performance required should not be determined individually, but on a community basis. A resilience performance level matrix combining functionality and recovery time as performance levels and performance objectives at increasing level of seismic intensity is presented. The resilience-based approach is illustrated to show its efficiency and better description with respect to PBD.

11.6 Future trends

In the near future more attention should be given to interdependencies among different dimensions of the framework which might cause drastic cascading effects (physical network interdependencies, etc.). Furthermore, in order to coordinate future recovery processes and international collaborations, common formats and tools should be applied for rapid resilience assessments, for example the reports from the recent earthquakes should follow the same format. New open source community software frameworks should be developed through education and training, in order to share a common research vision and to raise awareness of stakeholders worldwide and locally.

11.7 Acknowledgements

The research leading to these results has received funding from the European Community's Seventh Framework Programme – Marie Curie International Reintegration Actions – FP7/2007–2013 under the Grant Agreement no. PIRG06-GA-2009-256316 of the project ICRED – Integrated European Disaster Community Resilience. It also received support from the R&D Programme under the grant Agreement no. 3435777CDC of the project ECRIS – European Communities Resilient Integrated through Smart phones.

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Assessing seismic risks for new and existing buildings using performance-based earthquake engineering (PBEE) methodology

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Abstract: Performance-based earthquake engineering (PBEE) aims to quantify performance of facilities using metrics that are of immediate use to both engineers and stakeholders. A rigorous yet practical implementation of a PBEE methodology is presented. The methodology considers seismic hazard, structural response, resulting damage, and repair costs associated with restoring the building to its original condition, using a fully consistent, probabilistic analysis of the associated parts of the problem. The methodology is demonstrated using an actual high-rise building designed using three design approaches. The proposed procedure is consistent with common building design, construction, and analysis practices such that it can be readily adopted in earthquake engineering practice today.

Key words: performance-based earthquake engineering (PBEE), high-rise building design, performance assessment, seismic risk assessment, life-cycle cost.

12.1 Introduction

Earthquake engineering has evolved from using a set of prescriptive provisions, indirectly aimed at providing life safety, to performance-based approaches with direct consideration of a range of performance objectives, such as minimizing the repair cost of the facility under a range of earthquake shaking intensities. Performance-based approaches have several advantages; they offer a more effective way to design structural systems to achieve higher performance objectives and take into consideration more comprehensively various performance metrics that allows stake holders to make informed risk-based decisions. Whereas engineers are familiar with performance measures such as drift, acceleration, and strain, many decision makers prefer performance metrics that relate more directly to business decisions, such as downtime or repair costs. An engineering challenge has been to consistently consider seismic hazard, structural response, damage, and consequences, such that a fully probabilistic statement of expected performance of structures can be made.

A rigorous, yet practical approach to performance-based earthquake engineering (PBEE) is presented in this chapter. The approach considers the seismic hazard, structural response, damage, and repair costs associated with restoring the building to its original condition using a fully consistent probabilistic analysis. The approach could be generalized to consider other performance measures, such as casualties and down-time. The procedure is organized to be consistent with conventional building design, construction, and analysis practices so that it can be readily incorporated into a design approach.

12.2 Performance-based earthquake engineering (PBEE) framework

Owing to uncertainties in earthquakes, it is best to deal with earthquake-related problems using a probabilistic approach instead of deterministic approach. The basic probability theory that is used to derive the PBEE framework is briefly summarized herein. Additional references can be found in probability and statistics textbooks (e.g. Ang and Tang 2007). Eq. [12.1] shows the notation of conditional complementary cumulative distribution function (CCDF) of a random variable X given the value of another random variable $Y = y$. For example, this function can represent the probability of the inter-story drift ratio of a building exceeding 1 percent given the magnitude 7 earthquake has happened. Eq. [12.2] shows total probability theorem for the event A given the condition that n mutually exclusive and collectively exhaustive discrete random variables E_i have happened. For example, $P(A|E_i)$ represents the conditional probability of building repair cost exceeding 1 million dollars given a magnitude 7 earthquake has happened, while $P(E_i)$ represents the probability of a magnitude 7 earthquake happening at the building site. Multiplying the conditional probability $P(A|E_i)$ by probability $P(E_i)$ and summing over a range of shaking intensities (for example, probability of magnitude 6 and 8 earthquakes happening at the site independently), the product could represent the total probability that the building has a repair cost exceeding 1 million dollars under the range of earthquake shaking intensities considered.

$$G(x|y) = P(X > x | Y = y) \quad [12.1]$$

$$P(A) = \sum_{i=1}^n P(A|E_i)P(E_i) \quad [12.2]$$

Eq. [12.2] is further modified to Eq. [12.3] to account for E being a continuous random variable. Similar to Eq. [12.2], Eq. [12.3] shows the total probability of event $A > a$ given the event E has occurred.

$$P(A > a) = \int_E P(A > a | E = de) dP(de) = \int_E G(a|e) dG(e) \quad [12.3]$$

where de represents a small range of the continuous random variable E and the integration bound is set over the entire range of E .

12.2.1 Derivation of the PBEE framework

Equation [12.4] and Fig. 12.1 show the PBEE framework established by the Pacific Earthquake Engineering Research (PEER) Center (Moehle and Deierlein 2004, Goulet *et al.* 2007, Solberg *et al.* 2008, Yang *et al.* 2009, ATC-58 2010, Bradley and Lee 2010).

$$\lambda(DV > dv) = \int \int \int G(dv|dm) dG(dm|edp) dG(edp|im) |d\lambda(im)| \quad [12.4]$$

As implemented in Eq. [12.4], the PEER PBEE framework decomposes the performance assessment into four analysis steps: (1) seismic hazard analysis, (2) response analysis, (3) damage analysis, and (4) loss analysis. The outcome of the analysis is then then combined using the total probability theorem as described in Eq. [12.4]. The decomposition of the framework into the individual analysis phases allows the experts from different disciplines to work independently towards the PBEE assessment. The following paragraphs summarize the PBEE assessment in each of the analysis phases.

- **Seismic hazard analysis:** Probabilistic seismic hazard analysis is usually used to describe the seismic hazard at a site. The outcome of the probabilistic seismic hazard analysis is a seismic hazard curve, $\lambda(IM)$, that quantifies the annual rate of exceeding a given value of seismic intensity measure (IM). For example, $\lambda(IM)$ may represent the rate at which peak ground acceleration will exceed $0.2 g$ for a particular site in a given year. In addition, seismic hazard analysis is used to characterize the ground



12.1 Performance-based earthquake engineering framework (Yang *et al.* 2009) dv represents the decision variable (such as repair cost); dm represents damage measure (such as amount of crack in the concrete shear wall); edp represents engineering demand parameter (such as inter-story drift ratio); im represents intensity measure (such as the peak ground acceleration).

motions that can be used in the response analysis. Details of the ground motion selection and scaling procedures can be identified in many leading research articles for additional discussion; see Abrahamson (2006) and Haselton *et al.* (2009).

- *Response analysis:* Once the seismic hazard is quantified, the response of structural and nonstructural components under seismic excitations, sometimes referred to as engineering demand parameters (*EDP*), is obtained through the use of analytical, physical, or hybrid models. The outcomes of response analysis are statistical functions that relate engineering demand parameters (such as drift or stress) to the hazard experienced by the structure.
- *Damage analysis:* Based on test data, post-earthquake reconnaissance reports, or analytical simulation, the damage states of structural and nonstructural component damage can be characterized in terms of fragility curves. The fragility curves are cumulative distribution functions (CDFs) representing the probability that a damage state has been reached or exceeded given a quantitative measure of the *EDP*. The outcome of the analysis are quantitative description of the damage state when the *EDP* has reached a certain threshold values.
- *Loss analysis:* Loss analysis translates the results of the damage analysis to decision variables that can be used by building owners and stakeholders to make a risk management decision. The outputs of the loss analysis can be, for example, the probability of exceeding a certain threshold for the repair cost during a period of time, the expected monetary loss for repairs of the structure, and the total monetary loss for the structure with a particular probability of exceedance.

Random variables are used to quantify performance and to preserve the statistical uncertainties inherent to the problem. The seismic hazard analysis uses a probabilistic analysis of the seismic environment, ground shaking attenuation relations, and site conditions to derive a model for the seismic shaking intensity at a site. The output of the seismic hazard analysis is a statistical function that represents the annual rate of exceedance of certain intensity measures $\lambda(IM > im)$. Response analysis uses the *EDP* as the random variable and produces the conditional probability function, $G(edp|im)$, to represent the statistical relationship between *EDP* and *IM*. Damage analysis uses the damage measure (*DM*) as the random variable and the results of the analysis is a conditional probability function, $G(dm|edp)$, that relates *DM* and *EDP*. Lastly, the loss analysis uses decision variable (*DV*) as the random variable and produces the conditional probability function, $G(dv|dm)$, that relates *DV* and *DM*.

The decomposition of the PBEE process outlined above is made possible using the following statistical independence assumptions:

- $G(dm|edp, im) = G(dm|edp)$. This means the conditional probability of DM given EDP and IM is equivalent to the conditional probability of DM given EDP only.
- $G(dv|dm, edp) = G(dv|dm)$. This means the conditional probability of DV given DM and EDP is equivalent to the conditional probability of DV given DM only.
- $G(dv|dm, im) = G(dv|dm)$. This means the conditional probability of DV given DM and IM is equivalent to conditional probability of DV given DM only.

Using total probability theorem, the probability of exceedance for EDP , DM , and DV is presented in Eqs. [12.5] to [12.7], respectively.

Response analysis:

$$\begin{aligned} P(EDP > edp) &= \int_{im} G(edp|im) dG(im) \Rightarrow dP(EDP > edp) \\ &= \int_{im} dG(edp|im) dG(im) \end{aligned} \quad [12.5]$$

Damage analysis:

$$\begin{aligned} P(DM > dm) &= \int_{edp} G(dm|edp) dG(edp) \Rightarrow dP(DM > dm) \\ &= \int_{edp} dG(dm|edp) dG(edp) \end{aligned} \quad [12.6]$$

Loss analysis:

$$P(DV > dv) = \int_{dm} G(dv|dm) dG(dm) \quad [12.7]$$

By conditioning Eq. [12.7] on im , Eq. [12.7] can be expressed as

$$\begin{aligned} P(DV > dv|im) &= \int_{dm} G(dv|dm, im) dG(dm|im) = \int_{dm} G(dv|dm) dG(dm|im) \\ &\Rightarrow G(dv|im) = \int_{dm} G(dv|dm) dG(dm|im) \end{aligned} \quad [12.8]$$

By conditioning Eq. [12.6] on im , Eq. [12.6] can be expressed as

$$\begin{aligned} dP(DM > dm|im) &= \int_{edp} dG(dm|edp, im) dG(edp|im) \Rightarrow dG(dm|im) \\ &= \int_{edp} dG(dm|edp) dG(edp|im) \end{aligned} \quad [12.9]$$

Substituting Eq. [12.9] into Eq. [12.8]

$$G(dv|im) = \int \int_{dm \ edp} G(dv|dm) dG(dm|edp) dG(edp|im) \quad [12.10]$$

Eq. [12.10] represents the conditional probability of a decision variable exceeding a value dv given a value of intensity measure $IM = im$. Similar derivation can be made for $G(dm|im)$ and is shown in Eq. [12.11].

$$G(dm|im) = \int_{edp} G(dm|edp) dG(edp|im) \quad [12.11]$$

12.2.2 Annual rate of exceeding a threshold value

$G(dv|im)$, $G(dm|im)$, and $G(edp|im)$ are the conditional probabilities of the performance measures DV , DM , and EDP given an intensity measure $IM = im$. To translate the conditional probability to a quantity that can be readily used by engineers and other stakeholders to make a risk management decision, these conditional probabilities are multiplied by the absolute value of the derivative of the annual rate of exceedance of a given value of

the ground motion intensity measure $\left| \frac{d\lambda(im)}{dim} \right|$. Because the seismic hazard curve, $\lambda(im)$, is defined as the annual rate the earthquake ground motion intensity measure IM exceeds a value im , a derivative is used to compute the annual frequency (rate of occurrence) of the intensity measure $IM = im$. An absolute value is used to insure that this rate is a positive number regardless of the shape of the hazard curve itself.

The product of the annual rate of occurrence and the conditional probability of the performance measure given an intensity measure gives the annual rate of the performance measure exceeding a threshold value.

In other words, $\left| \frac{d\lambda(im)}{dim} \right|$ represents the annual frequency of a random variable IM equaling im and $G(dv|im)$ represents the probability that the random variable DV takes values larger than dv for shaking intensity equaling im . Thus, their product gives the number of occurrences of $DV > dv$ annually, as shown in Eq. [12.12].

$$\frac{\lambda(DV > dv)}{dim} = G(dv|im) \left| \frac{d\lambda(im)}{dim} \right| \quad [12.12]$$

Integrating Eq. [12.12] for all intensity measures, the annual rate that DV exceeds a threshold value dv for all intensity measures considered is shown in Eq. [12.13].

$$\lambda(DV > dv) = \int_{im} G(dv|im) \frac{|d\lambda(im)|}{|d\lambda(im)|} d\lambda(im) = \int_{im} G(dv|im) |d\lambda(im)| \quad [12.13]$$

Finally the PEER PBEE framework shown in Eq. [12.4] can be obtained by substituting Eq. [12.10] into Eq. [12.13]. Note that Eq. [12.4] uses the same conditional probability for successive earthquake events. This implies the structure is non-deteriorating or is restored to its original condition immediately after any structural damage due to an earthquake.

12.2.3 Implementation of the PBEE framework

The PBEE framework described in the previous section can be used as the basis for developing rigorous PBEE procedures. The challenge is to implement the methodology in a manner that is practicable to use in a typical design office setting. Two issues must be addressed to achieve this goal: (1) the performance measures (DV , DM , and EDP) and their conditional cumulative distribution functions must be quantified and formulated in a straightforward way using data readily available to practicing engineers; and (2) the intense computations required to integrate the PEER PBEE framework (Eq. [12.14]) must be transformed into procedures and routines that are transparent and easy to implement. An implementation that fulfills these two goals is presented in the following steps:

1. Define structural and nonstructural components to be considered in the performance assessment.
2. Conduct seismic hazard analysis and ground motion record selection.
3. Evaluate the response of the building.
4. Generate additional correlated EDP vectors.
5. Compute the total repair cost.
6. Different representations of the total repair cost.

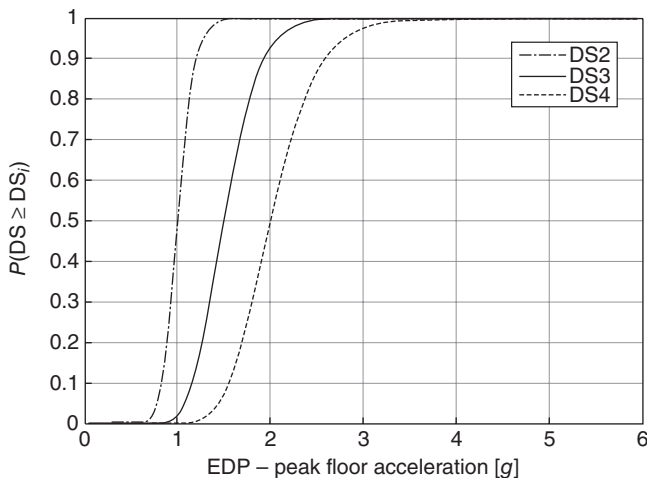
Define structural and nonstructural components to be considered in the performance assessment

The outcome of this step is a series of repair quantity tables for a structure. These tables correlate the structural and nonstructural component damage states and repair actions needed to restore them. They are formulated using the following procedure. Depending on the structural system and intended function of the structure, relevant structural and nonstructural components of the building are selected and separated into different performance groups (PG). Each performance group consists of one or more building components whose performances are similarly affected by a particular EDP . For example, one performance group might consist of all similar

nonstructural components whose performance is sensitive to floor acceleration and another performance group might be sensitive to the inter-story drift between the second and third floor. The selection of components in each performance group is based on engineering judgment of the importance of the contribution of these components to the overall performance of the structure.

A sufficient number of damage states (DS) are defined for each performance group to completely describe a range of damage extents to the components in the performance group. The damage states are defined in relation to the repair actions needed. For each damage state, a damage model (fragility relation) is used to define the probability the component will be equal or greater than the damage state given an *EDP* value. Figure 12.2 shows an example of fragility curves defined for a performance group.

As shown in Fig. 12.2 the performance group has four damage states. DS1 represents no damage, DS2 represent slight damage, DS3 represents severe but repairable damage, and DS4 represents total replacement. Depending on the demand expressed using the *EDP* value, the probabilities of the performance group being in each damage state can be identified from the fragility curves. For example, if the *EDP* equals 1.5 g, the probability of the components in the performance group being in DS1 is close to zero, DS2 is approximately 0.5, DS3 is approximately 0.42, and DS4 is approximately 0.08.



12.2 Example of fragility curves.

Table 12.1 Sample repair quantity table for the performance group shown in Fig. 12.2

Repair item	Unit	DS1	DS2	DS3	DS4
Water damage	sf**	0	0	10000	20000
Remove furniture	sf	0	4000	10000	20000
Ceiling system removal	sf	0	0	0	20000
MEP* removal	sf	0	0	500	2000
Replace ceiling tiles	sf	0	2500	8000	8000
Replace ceiling system	sf	0	0	0	20000
MEP* replacement	sf	0	0	500	2000

* MEP = Mechanical, electrical and plumbing.

** sf = square feet.

After the performance groups have been identified, building data, such as as-built documents or in-use surveys, are employed to quantify the repair quantities of the components of each performance group in the building. For example, square footage of partition walls and number of pocket doors may be computed. Because each damage state is defined according to the repair action, the total repair quantities for each item in the performance group at different damage states can be defined according to the functionality of the structure. Table 12.1 shows an example of the repair quantities for each item in the sample performance group shown in Fig. 12.2 at different damage states. Additional damage states and repair items can be added for different performance groups.

Conduct seismic hazard analysis and ground motion record selection

A conventional seismic hazard analysis is conducted, taking into account the site and the basic dynamic characteristics of the building. One outcome of the seismic hazard analysis is a hazard curve that quantifies expected ground motion intensity measures as a function of exceedance probability considered in the PBEE analysis of the building. The hazard curve and engineering judgment are used to identify several discrete hazard levels for which the performance of the building will be further examined. Another outcome of the seismic hazard analysis is a suite of ground motions representing the seismicity of the site at different seismic hazard levels. For example, a suite of ground motion records representing the seismic hazard with a 10% probability of exceedance in 50 years at the site may be provided. A typical suite comprises several ground motions with their intensity scaled to the level implied by the seismic hazard. The motions may be further divided into subclasses by record type, such as near-field or far-field ground motions.

Table 12.2 Sample *EDP* matrix, $\mathbf{X}$

GM number	<i>EDP</i> 1	<i>EDP</i> 2	...	<i>EDP</i> <i>N</i>
GM 1	0.66	1.07	...	0.75
GM 2	0.68	0.95	...	0.27
⋮	⋮	⋮	⋮	⋮
GM <i>M</i>	0.76	1.10	...	0.52

Evaluate the response of the building

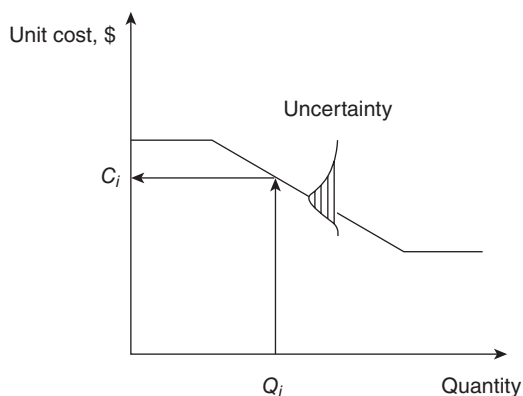
With the selected ground motions, a series of dynamic analyses, including nonlinear dynamic analysis if necessary, can be used to determine the earthquake response of the building. Depending on the *EDP* associated with each performance group (defined in Step 1), peak values of the *EDPs* obtained from dynamic analyses are summarized in an *EDP* matrix, $\mathbf{X}$, as shown in Table 12.2. The columns of $\mathbf{X}$ represent different *EDPs* of interest (for example inter-story drift or floor acceleration) and the rows of $\mathbf{X}$ represent the peak *EDPs* recorded from the dynamic analysis of a single ground motion. One *EDP* matrix will be defined for each intensity measure considered.

Generate additional correlated EDP vectors

Computing additional *EDP* vector realizations using dynamic analysis is hampered by the paucity of actual strong ground motions and the computational cost. Therefore, to circumvent running additional nonlinear dynamic analyses, a joint lognormal distribution is fitted to the *EDP* matrix. Correlated *EDP* vectors can then be generated using a computationally inexpensive procedure presented in Section 12.3.4.

Compute the total repair cost

With the generated correlated *EDP* vectors, the total repair quantities for all repair items in the building due to each earthquake scenario can be calculated. Depending on the values of the *EDP* associated with the performance group, a random number generator with a uniform distribution is used to select the damage state for the damaged model defined in Step 1. Once the damaged states have been identified, the repair quantities for each item in the performance group are obtained from the repair quantity table, such as the one shown in Table 12.1. This process is repeated for all performance groups and the total repair quantities for each item are summed from the quantities obtained from each performance group.



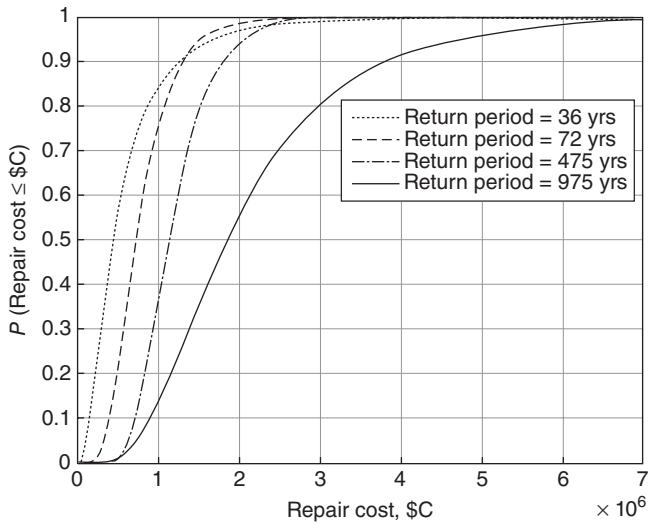
12.3 Example of cost function (Yang *et al.* 2009).

Once the total repair quantities are identified, the total repair cost for the building is computed by multiplying the total repair quantity by the unit repair cost. Figure 12.3 shows an example of the unit repair cost function. The uncertainty of the unit price is represented by using a random number generator, based on the tabulated beta factors for the cost functions, to adjust base unit costs up or down before multiplying by the total quantities associated with each repair measure. This is the repair cost for one realization of the *EDPs*. The process is repeated a sufficiently large number of times to obtain a distribution of total repair costs given the seismic hazard level. In the methodology adopted here, the performance groups are assumed to be statistically independent.

Different representations of the total repair cost

Steps 1 through 5 present a logical and consistent methodology that can be used to obtain a distribution of the total repair cost of a building for one intensity measure. Figure 12.4 shows an example of such distribution curves for different intensity measures considered. These curves can be readily used as a basis for making risk management decisions. For example, the curve demonstrates the amount of seismic risk increase (in terms of the total repair cost) as a function of the return period of earthquake ground shaking. Similar curves can be generated to compare the performance of different structural framing systems, or different retrofitting strategies on the same building.

The repair cost information can be further refined by computing the annual rate of total repair cost exceeding a threshold value. Such annual rate information is obtained by first computing the complement of the



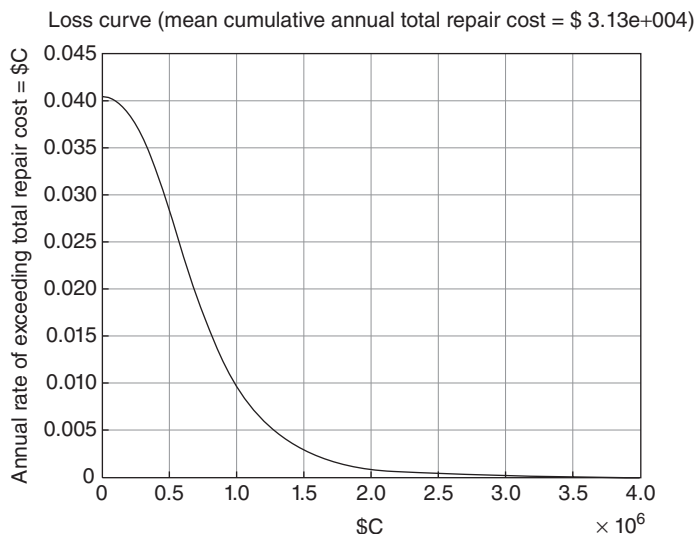
12.4 Example of distribution of repair cost.

cumulative distribution function shown in Fig. 12.4, then multiplying it by the slope of the hazard curve at the corresponding ground motion intensity measure level, and finally integrating the resulting curves across the intensity measure interval considered in the seismic hazard analysis. Repeating this process for all repair cost values produces a loss curve that represents the annual rate of the total repair cost (TC) exceeding a threshold value. Figure 12.5 shows a sample loss curve obtained from the sample building performance assessment shown in Fig. 12.4. Note that the procedure presented here is identical to the process presented in Eq. [12.4] where the conditional probability, $G(TC|im)$ (complement of the CDF generated using the procedure presented in Step 1 through Step 5), is multiplied by the derivative of the seismic hazard curve and integrated through all seismic intensity measure levels.

Lastly, following the derivation presented in Der Kiureghian (2005), valid for a non-negative random variable X , the expected cumulative value of X is

$$E\left[\sum X\right] = \int_0^{\infty} x |d\lambda(x)| = \int_0^{\infty} \lambda(x) dx \quad [12.14]$$

where the last expression is obtained from integration by parts. Thus, the area under the loss curve represents the mean cumulative annual total repair cost for all earthquake events occurring in one year. Information



12.5 Example of loss curve (Yang *et al.* 2009).

such as this can be used by building owners/stakeholders to make a rational decision regarding building insurance.

The procedures in Step 3 through Step 5 have been implemented in a computer program and thus automated (Yang 2009). Input to the program requires the user to define the performance groups, the repair quantity tables, the repair cost functions, the *EDP* matrices obtained by running a limited number of nonlinear dynamic analyses, and the total number of repair cost simulations required to compute the loss function. Given these input quantities, the program generates loss functions in a variety of different formats, including those described above.

12.2.4 Generating correlated *EDP* vectors

To generate correlated *EDP* vectors, the peak quantities of the *EDPs* recorded from sets of dynamic analysis are tabulated into matrix **X**, as shown in Table 12.2. Each column of **X** represents an *EDP*, while each row of **X** represents different *EDP* recorded for the same ground motion. Because the entries of the *EDP* matrix **X** are obtained from the maximum of the absolute value of the *EDP* response, **X** is assumed to be lognormal distribution. **X** is then transformed to a normal distribution **Y** by taking the natural log of **X**. The mean vector **M_Y**, the diagonal standard deviation matrix **D_Y**, and the correlation coefficient matrix **R_{YY}** are then calculated using Eq. [12.15] through Eq. [12.18].

$$\mathbf{M}_Y = [\text{mean}(\mathbf{Y})]^t \quad [12.15]$$

$$\mathbf{D}_Y = \text{diag}[\text{std}(\mathbf{Y})] \quad [12.16]$$

$$\mathbf{R}_{YY} = \text{corrcoef}(\mathbf{Y}) \quad [12.17]$$

$$\mathbf{L}_Y = [\text{chol}(\mathbf{R}_{YY})]^t \quad [12.18]$$

where $\mathbf{Y}$ represents a $m \times n$ matrix; $[\text{mean}(\mathbf{Y})]^t$ represents an $n \times 1$ vector, where the i th entry of the vector represents the mean value of the i th column of $\mathbf{Y}$; $\text{diag}[\text{std}(\mathbf{Y})]$ represents an $n \times n$ diagonal matrix, where the i th diagonal entry of the matrix represents the standard deviation of the i th column of $\mathbf{Y}$; $\text{corrcoef}(\mathbf{Y})$ represents an $n \times n$ matrix, where the entry at the i th row and j th column represents the correlation coefficient between the variable at the i th and j th column of the matrix $\mathbf{Y}$; $[\text{chol}(\mathbf{R}_{YY})]^t$ represents the Cholesky factorization of the matrix $\mathbf{R}_{YY}$, where the outcome is an $n \times n$ lower triangular matrix.

With the statistical distribution calculated from Eq. [12.15] through Eq. [12.18], the additional correlated *EDP* vectors can be generated using a function of random variable as shown in Eq. [12.19].

$$\mathbf{Z} = \mathbf{A}\mathbf{U} + \mathbf{B} \quad [12.19]$$

where $\mathbf{Z} = [Z_1 \ Z_2 \ \dots \ Z_n]^t$ represents an $n \times 1$ vector. Each entry of $\mathbf{Z}$ represents a random variable (e.g., first-story drifts, second-story drifts, $\dots$). $\mathbf{A}$ is an $n \times m$ constant coefficient matrix representing the linear transformation from $\mathbf{U}$ to $\mathbf{Z}$. $\mathbf{U} = [U_1 \ U_2 \ \dots \ U_m]^t$ represents an $m \times 1$ vector of random variables. $\mathbf{B}$ represents an $n \times 1$ constant coefficient vector.

Because the transformation from $\mathbf{U}$ to $\mathbf{Z}$ is affine, the mean vector and covariance matrix of $\mathbf{Z}$ are related to the mean vector and covariance matrix of $\mathbf{U}$ by Eq. [12.20] and Eq. [12.21], respectively.

$$\mathbf{M}_Z = \mathbf{A}\mathbf{M}_U + \mathbf{B} \quad [12.20]$$

$$\mathbf{\Sigma}_{ZZ} = \mathbf{A}\mathbf{\Sigma}_{UU}\mathbf{A}^t \quad [12.21]$$

where $\mathbf{M}_Z$ and $\mathbf{M}_U$, and $\mathbf{\Sigma}_{ZZ}$ and $\mathbf{\Sigma}_{UU}$ are the mean vector and covariance matrix of $\mathbf{Z}$ and $\mathbf{U}$, respectively.

By selecting $\mathbf{U}$ to be a vector of uncorrelated standard normal random variables with zero mean and unit standard deviation, the mean vector and covariance matrix of $\mathbf{U}$ can be simplified to Eq. [12.22]:

$$\mathbf{M}_U = \mathbf{0} \text{ and } \mathbf{\Sigma}_{UU} = \mathbf{I} \quad [12.22]$$

where $\mathbf{0}$ is an $m \times 1$ vector with all its entry equals to zeros. This is because the mean of the random variables, $U_{i=1,2,\dots,m}$, is selected to be 0, $\mathbf{I}$ is an m

$\times m$ identify matrix, this is because the means U_i and U_j is uncorrelated, for $i \neq j$.

Substituting Eq. [12.22] into Eq. [12.20] and Eq. [12.21], the mean vector and covariance matrix of $\mathbf{Z}$ can be reduced to

$$\mathbf{M}_Z = \mathbf{A}\mathbf{0} + \mathbf{B} = \mathbf{B} \text{ and } \Sigma_{ZZ} = \mathbf{A}\mathbf{A}^t = \mathbf{A}\mathbf{A}^t \quad [12.23]$$

Because $\mathbf{Y}$ and $\mathbf{Z}$ are both joint normal distribution, if the mean vector and covariance matrix of $\mathbf{Z}$ matches the mean vector and covariance matrix of $\mathbf{Y}$, $\mathbf{Y}$ and $\mathbf{Z}$ will have the same joint normal distribution. Equations [12.24] and [12.25] show the selection of $\mathbf{A}$ and $\mathbf{B}$ such that $\mathbf{Z}$ will have the same joint normal distribution as $\mathbf{Y}$.

$$\mathbf{M}_Z = \mathbf{M}_Y \Rightarrow \mathbf{B} = \mathbf{M}_Y \quad [12.24]$$

$$\Sigma_{ZZ} = \Sigma_{YY} \Rightarrow \mathbf{A}\mathbf{A}^t = \Sigma_{YY} \Rightarrow \mathbf{A} = [\text{chol}(\Sigma_{YY})]^t \quad [12.25]$$

where chol is the Choleski factorization of any square and positive definite matrix.

Because the entry of the covariance matrix is not bounded, the computation might cause numerical instability. The Choleski factorization of Σ_{YY} is calculated using Eq. [12.26].

$$\begin{aligned} \Sigma_{YY} &= \mathbf{D}_Y \mathbf{R}_{YY} \mathbf{D}_Y \Rightarrow \mathbf{A}\mathbf{A}^t = \mathbf{D}_Y \mathbf{R}_{YY} \mathbf{D}_Y \Rightarrow \mathbf{A} \\ &= [\text{chol}(\mathbf{D}_Y \mathbf{R}_{YY} \mathbf{D}_Y)]^t = \mathbf{D}_Y [\text{chol}(\mathbf{R}_{YY})]^t = \mathbf{D}_Y \mathbf{L}_Y \end{aligned} \quad [12.26]$$

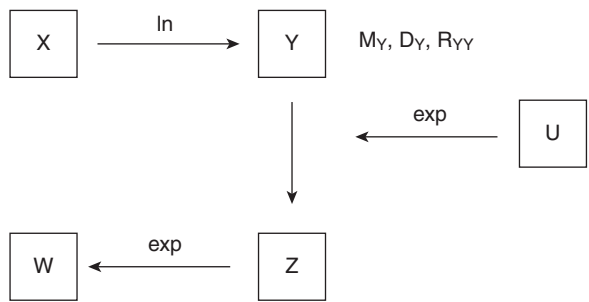
where $\mathbf{D}_Y$ is a diagonal matrix with standard deviations of random variables $\mathbf{Y}$ along its diagonal and $\mathbf{R}_{YY}$ is the correlation coefficient matrix of random variables $\mathbf{Y}$. By substituting Eqs. [12.24] and [12.26] into Eq. [12.19], the correlated EDP vector can be generated using Eq. [12.27].

$$\mathbf{Z} = \mathbf{D}_Y \mathbf{L}_Y \mathbf{U} + \mathbf{M}_Y \quad [12.27]$$

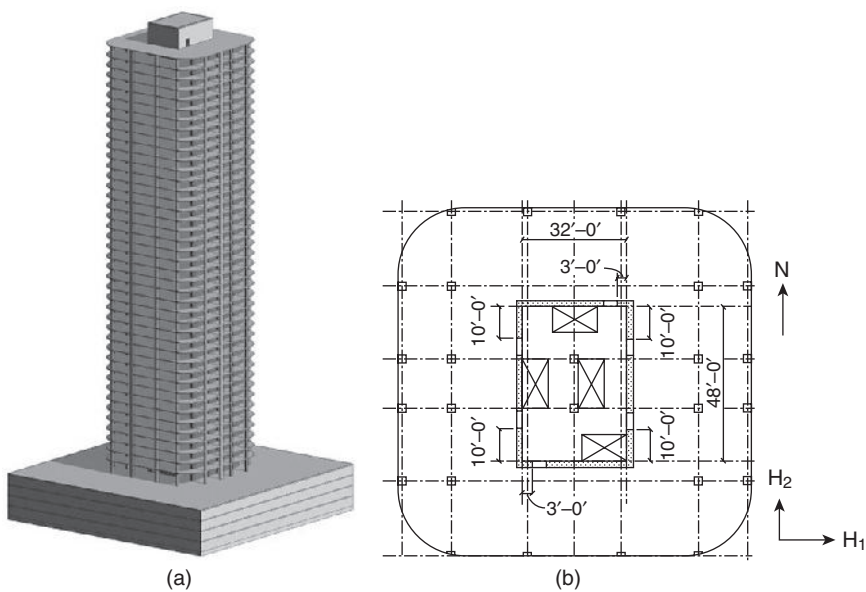
Finally, the generated *EDP* vectors are transformed to the lognormal distribution, $\mathbf{W}$, by taking the exponential of $\mathbf{Z}$. Figure 12.6 summarizes the process of generating correlated *EDP* vectors.

12.3 Application: seismic performance assessment of high-rise buildings

This section shows an example of applying the performance-based assessment procedure to compare the seismic performance of a high-rise building designed using three design approaches. Effectiveness of the design methods is assessed using the repair costs associated with anticipated earthquake ground shaking.



12.6 Process of generating the correlated *EDP* vectors (Yang *et al.* 2009).



12.7 (a) Isotropic view and (b) plan view of the prototype building (Moehle *et al.* 2011).

12.3.1 Description of the prototype buildings

Figure 12.7 shows a 42-story residential prototype building located in Los Angeles, California. Also indicated in Fig. 12.7 are the orientation, floor dimension, and major principal axes of the building. This building is used as one of the prototype model within the PEER Tall Building Initiative (TBI) research project (Moehle *et al.* 2011). The seismic force resisting system consists of a centrally located concrete core wall with concrete cou-

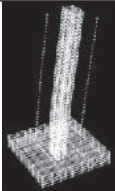
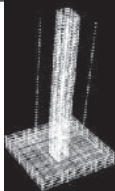
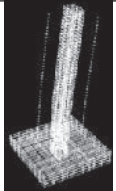
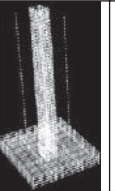
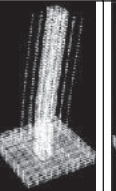
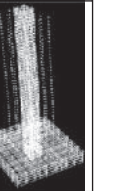
pling beams. The gravity system consists of concrete columns with post-tensioned concrete slabs.

The prototype building was designed using three design provisions:

- Design A – Code: Prescriptive provisions as outlined in the 2006 International Building Code (IBC 2006). All prescriptive provisions of the building code were followed except the height limit.
- Design B – LATBC: A performance-based design as outlined in the 2008 seismic design criteria published by the Los Angeles Tall Buildings Structural Design Council (LATBSDC 2008). All prescriptive provisions of the LATBSDC document were followed with the following exceptions (based on consensus of the TBI research team): (1) the minimum base shear specified by LATBSDC document was not followed; and (2) a serviceability analysis was checked using an earthquake with 25-year return period and 2.5% viscous damping. Only 20% of elements were allowed to reach 150% of their capacity.
- Design C – PEER TBI design: A performance-based plus design outlined by the PEER TBI project (TBI 2010). The building was designed with higher performance objectives, including a serviceability analysis using a 43-year return period earthquake with 2.5% viscous damping. For the serviceability analysis, ductile elements (coupling beams for core wall building) were allowed to reach 150% of their capacity, and the wall piers were limited with an axial stress $0.3f'_c$ (where f'_c represents the nominal compressive strength of the concrete at 28 days). The minimum strength was calculated based on the maximum of the 43-year return earthquake and wind loads.

Figure 12.8 shows the comparison of the modal properties for the three designs.

Major structure and non-structural components, and contents of the building were identified and grouped into performance groups. Table 12.3 summarizes the performance groups included in the assessment. As indicated in Table 12.3, there are a total of 1765 performance groups included in this study. This includes shear walls at all floors, shear wall boundary elements at all floors, link beams at all locations, gravity columns at all floors, curtain walls at all floors, interior partitions at all floors, elevators, and contents at all floors. These components are selected to represent the majority of the components in the building. Additional performance groups can be included. Also indicated in Table 12.3 is the *EDP* that is used to quantify the damage state of each performance group. For example, the damage state of the shear wall (SW) performance groups is selected based on the shear demand capacity ratio (DCR) of the shear wall. Figure 12.9 shows the fragility curves for the performance groups listed in Table 12.3. Table 12.4 summarizes the damage states and the associated repair costs of each

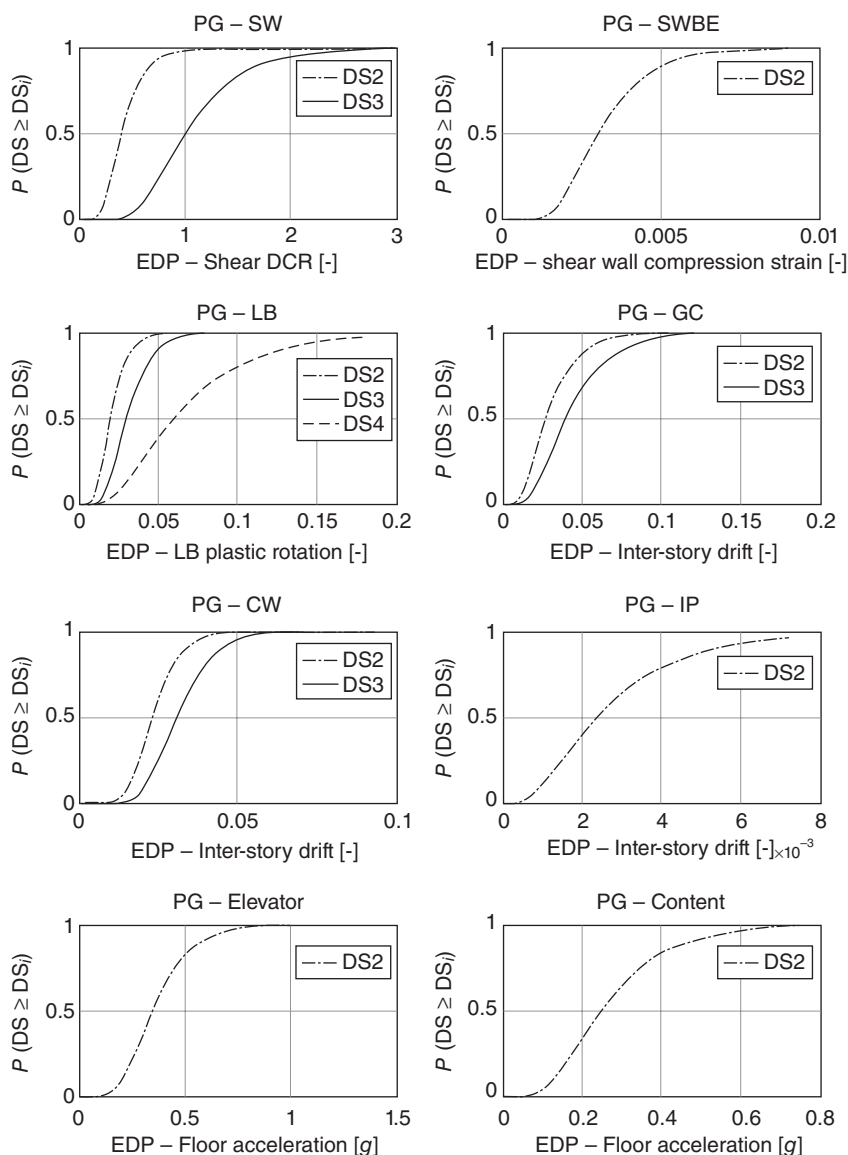
Design	Design A	Design A	Design B	Design B	Design C	Design C
Mode shape						
Period	T1 = 5.2 sec	T2 = 4.0 sec	T1 = 4.8 sec	T2 = 3.6 sec	T1 = 4.6 sec	T2 = 3.5 sec
Mass participation factor in H1 direction	M _{H1} = 0.61	M _{H1} = 0	M _{H1} = 0.6	M _{H1} = 0	M _{H1} = 0.58	M _{H1} = 0
Mass participation factor in H2 direction	M _{H2} = 0	M _{H2} = 0.63	M _{H2} = 0	M _{H2} = 0.61	M _{H2} = 0	M _{H2} = 0.61
Mass participation factor in V direction	M _V = 0	M _V = 0	M _V = 0	M _V = 0	M _V = 0	M _V = 0

12.8 Comparison of the modal properties.

Table 12.3 Performance groups included in the performance assessment of the prototype building

PG number	PG name	Items	EDP
1 to 470	SW	Core-wall webs at all floors	Shear demand capacity ratio (DCR)
471 to 1222	SWBE	Core-wall boundary elements at all floors	Wall strain
1223 to 1482	LB	Coupling beams at all locations	Plastic rotation
1483 to 1529	GC	Slab-column connections at all floors	Inter-story drift ratio
1530 to 1623	CW	Curtain walls at all floors	Inter-story drift ratio
1624 to 1717	IP	Interior partitions at all floors	Inter-story drift ratio
1718	Elevator	Elevators	Floor acceleration
1719 to 1765	Content	Contents at all floors	Floor acceleration

performance group in each damage states. The values of the fragility curves and the associated repair costs were obtained from TBI research project (Moehle *et al.* 2011). Similar data can be obtained from past experimental tests and/or post-earthquake reconnaissance reports.



12.9 Fragility curves included in the loss analyses for the buildings.

12.3.2 Site seismic hazard

A probabilistic seismic hazard analysis of the site was conducted (Moehle *et al.* 2011). A total of five hazard levels were included in the analysis. The hazard levels were designated using SLE25, SLE43, DBE, MCE, and OVE that correspond to return periods of 25, 43, 475, 2475 and 4975 years,

Table 12.4 Description of the damage states and repair costs for each performance groups included in the performance assessment (× \$US 1000)

PG	DS1	DS2	DS3	DS4	Note
SW	No damage \$0	Cracking \$15 to \$22	Complete failure \$30 to \$44		Per component
SWBE	No damage \$0	Spalling \$5.7			Per component
LB	No damage \$0	Residual cracks ≤ 1/16" wide \$20.4	Residual cracks > 1/8" and minor spalling \$25.5	Buckling or fracture of diagonal bar, crushing of concrete \$37.4	Per component
GC	No damage \$0	Slab yield w/ residual crack \$19.5	Punching \$23.5		Per component
CW	No damage \$0	Glass cracking \$1.6 to \$3.0	Glass falling \$1.6 to \$3.0		Per component
IP	No damage \$0	Damaged \$9.7 to \$15.8			Per floor
Elevator	No damage \$0	Damaged \$47.1			Per building
Content	No damage \$0	Damaged \$50			Per floor

respectively. A total of 75 pairs of ground motions (15 pairs per hazard level) were selected and scaled to match the target spectrum at each of hazard levels. The multiple hazard levels were selected to represent the range of earthquake shaking intensities exposed by the structure.

12.3.3 Response analysis

Nonlinear analytical models of the prototype buildings were developed using Perform-3D (CSI 2009). The analytical model needs to have sufficient

analytical capability to accurately model the nonlinear dynamic response of the structure. Figures 12.10 and 12.11 show the maximum inter-story drift ratio and floor acceleration in the H1 direction obtained from the time history analyses, respectively. The dash line represents the maximum response obtained from each earthquake and the solid line represents the mean of the maximum responses.

12.3.4 Results of the performance assessment

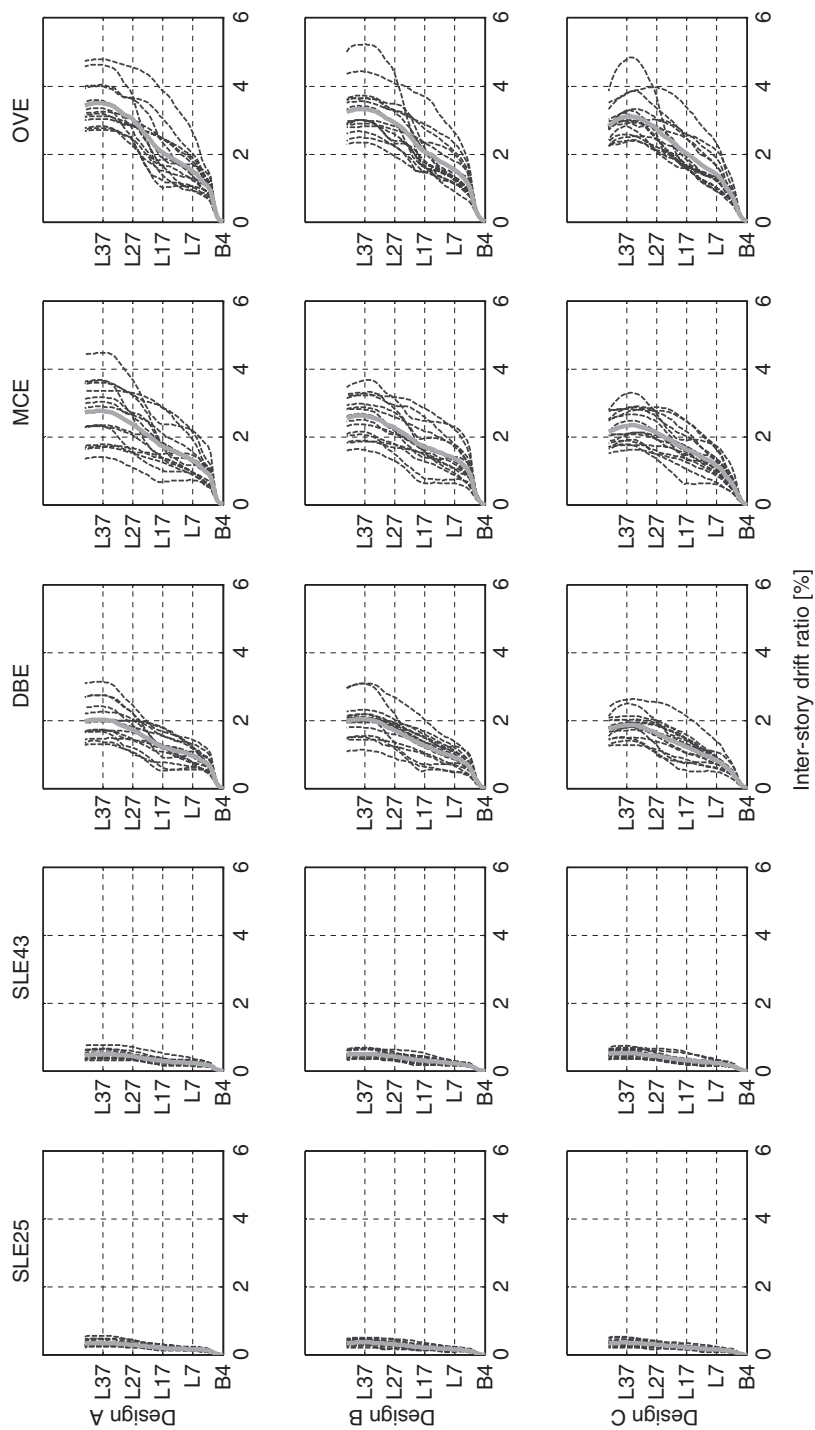
Using the procedure presented in Section 12.3.3, the performance of the prototype model under different levels of earthquake shaking intensities was analyzed. Figure 12.12 shows the repair cost distributions for Designs A, B, and C at all five hazard levels considered. As expected, as the shaking intensity increases, the repair cost increases. The result shows Design C has the lowest total repair cost as compared to Designs A and B for all hazard levels considered. Designs A and B have comparable distributions of the repair cost for all hazard levels. The probable maximum loss (PML) could be defined as the 90% non-exceedance cost for the DBE shaking intensity. By this definition, the PML are 19.9, 20.7, and 16.9 million USD for the Designs A, B, and C, respectively.

The distribution of the repair cost at each hazard level (Fig. 12.12) can be multiplied by the slope of the hazard curve, and integrated through all hazard levels to obtain the annualized loss curve (Fig. 12.13). Additionally, the mean annualized loss (MAL) can be calculated by integrating the area under the loss curve (Der Kiureghian 2005). As shown in Fig. 12.13, the annual rate of exceeding any fixed repair costs increases from Design C to A to B. Additionally, the MAL is \$326,000, \$336,000, and \$282,000 for Designs A, B, and C, respectively. This information can be used to evaluate whether Design A, B, or C is a better long-term investment option. For comparison purpose, the initial construction cost obtained from the TBI research (Moehle *et al.* 2011) for the three designs are shown in Table 12.5. The results shows Design C might cause 2 million more in the initial construction cost; however, on average Design C will have \$44,000 to \$54,000 less in earthquake related cost per year compared with designs A and B,

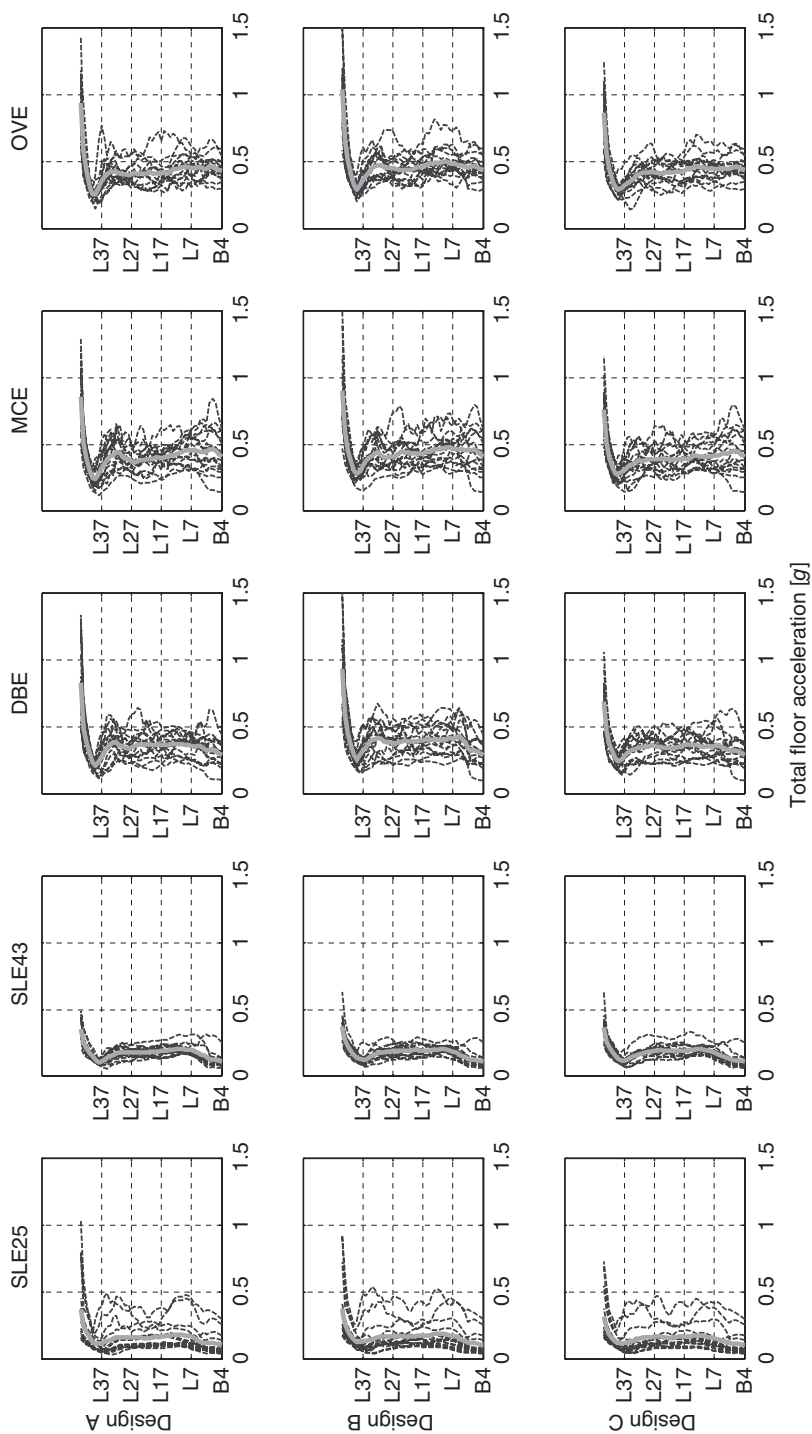
Table 12.5 Initial structural and content costs in millions of \$US

	Design A	Design B	Design C
Structural cost*	126	126	128
Structural cost* + content*	140	140	143

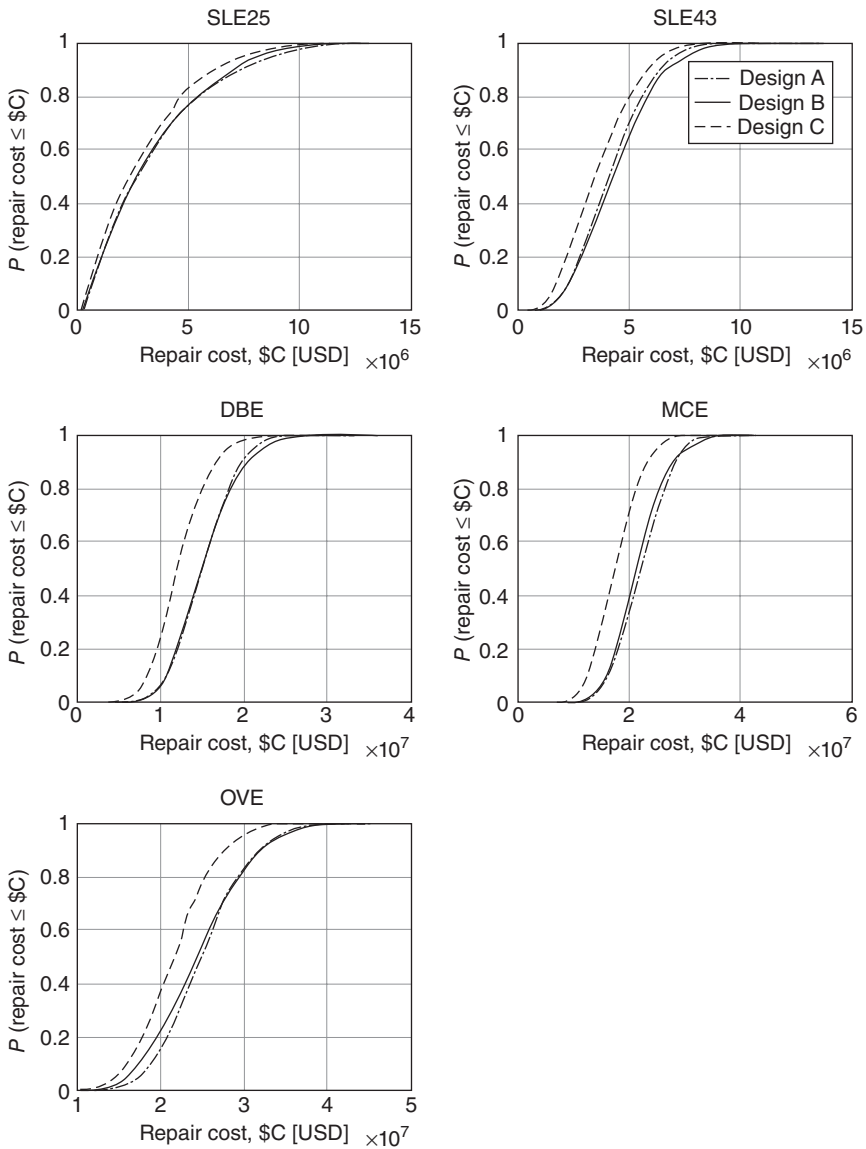
* The values presented are above the grade construction only.
Source: Moehle *et al.* (2011).



12.10 Maximum inter-story drift ratio in H1 direction.

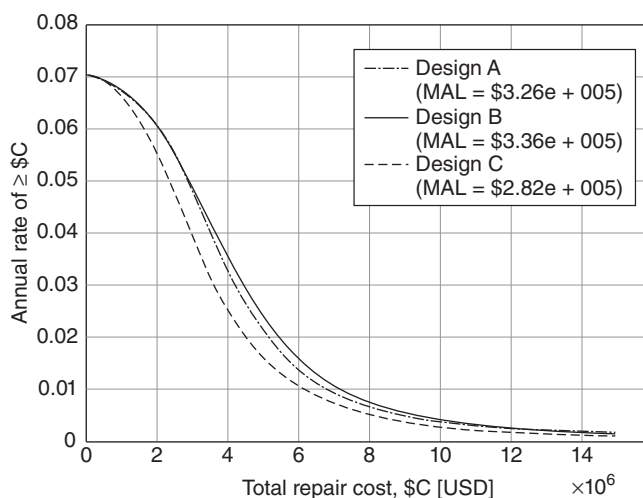


12.11 Maximum floor acceleration in H1 direction.



12.12 Repair cost distribution of the prototype model.

respectively. It should be noted that these differences do not account for the indirect cost created from the interruption of business. It is expected that the difference in financial loss will get larger once the indirect losses are included.



12.13 Loss curves of the prototype building using the three design approaches.

12.4 Conclusions

A fully probabilistic PBEE evaluation framework as well as a practical implementation of that framework using a novel solution strategy is presented. The procedure starts with systematic data collection to describe the seismic environment and the vulnerability of the structure. The seismic environment is described through probabilistic seismic hazard analysis. The vulnerability of the structure is described using the fragility curves of the structural and nonstructural components. The seismic response of the structure is examined by conducting a limited number of nonlinear dynamic analyses. Using the results of the nonlinear dynamic analyses, a large array of correlated *EDP* vectors are then generated using a fully probabilistic procedure presented within this chapter. With the combination of the correlated *EDP* vectors and the fragility curves, the damage states of the structure are then identified using random number generators. Finally, the repair costs of the entire building are then calculated through lookup tables based on the damage state identified for each component. The process is repeated a large number of times to generate the distribution of the structural repair costs of buildings under different ranges of earthquake shaking intensities considered.

This procedure has been implemented in a computer program to facilitate computation. An example of applying the proposed performance-based assessment procedure is used to compare the seismic performance of a high-rise building design using three design approaches. This example

illustrates how the performance-based assessment procedure can be used to rationalize the design approaches and to achieve a higher performance measure, such as lower lifetime maintenance cost for new and existing structural systems. The presented methodology can be easily expanded to consider other performance measures such as down-time and casualty rate, which can be used to make an informed risk management decision.

12.5 Acknowledgments

The author would like to acknowledge the contribution from Professor Jack Moehle (University of California, Berkeley), Professor Bozidar Stojadinovic (ETH, Zurich), and Professor Armen Der Kiureghian (University of California, Berkeley) for the development of the PBEE methodology presented herein. The building designs were carried out by Magnusson Klemencic Associates. PERFORM-3D analysis software was donated by Computers and Structures Incorporated. The fragility data used for the performance assessment were obtained through collaborative research with the researchers within the Tall Building Initiative project funded by the Pacific Earthquake Engineering Research Center and the Applied Technology Council ATC-58 project.

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Assessing the seismic vulnerability of masonry buildings

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Abstract: Masonry structures represent the highest proportion of the building stock worldwide and in regions affected by destructive seismicity. Together with reinforced concrete buildings they account for the largest proportion of casualties in earthquakes. However, well-built masonry buildings have survived for centuries, even in regions of high seismicity. In the past, seismic vulnerability assessment of masonry structures has been conducted using empirical methods, based on post-event collection of damage data. This approach does not allow easy correlation of structural behaviour with observed damage and quantification of the effect of seismic strengthening on seismic resistance. After an initial review of the main issues associated with the seismic vulnerability assessment of masonry buildings, the chapter presents the rationale and algorithm of a procedure based on limit state analysis and collapse mechanisms (FaMIVE), to derive capacity curves for masonry structures. It is shown how the procedure can be used to derive vulnerability and fragility functions following the methodology of the capacity spectrum method and the performance-based assessment method. Treatment of model and data uncertainties concludes the chapter.

Key words: masonry structures, seismic vulnerability, limit analysis, damage states, capacity curves, performance points, uncertainty.

13.1 Introduction

Notwithstanding several decades of academic research into the seismic behaviour of unreinforced masonry structures, their intrinsic vulnerability to seismic action is perceived as deficient. This is clearly manifested in current seismic codes and standards worldwide that restrict this structural type only to regions of low-to-moderate seismicity, allowing only for modest span and height dimensions. Millennial evidence proves that masonry is a sustainable and resilient material available to human kind, which is also able to withstand earthquakes, if adequately built.

Seismic vulnerability analysis of masonry structures has traditionally been carried out using 'empirical' (or statistical) methods. These can be divided in categorisation methods, which classify buildings into typologies characterised by propensity of damage, or inspection and rating methods,

wherein scores are attributed to each significant vulnerability component (UNDP/UNIDO, 1985). Several of these methods have been developed in the past 30 years and the more robust and commonly used are reviewed in Section 13.2.1.

Recent dramatic increases in computational powers and consequent improvement and refinement in numerical modelling of relatively complex structures, using either static or dynamic approaches (e.g. FEMA-356, 2000; Vamvatsikos and Cornell, 2005), has facilitated greater exploitation of analytical approaches for vulnerability assessment. Analytical methods need experimental validation of the various parameters used to define the vulnerability, and so far relatively little experimental work on the behaviour of masonry structures has been undertaken by the international community, albeit a wide variety of unreinforced masonry typologies, both historic and modern, exists. The framework within which analytical methods can be applied and their limitations are presented in Section 13.2.2.

An attempt to exploit positive aspects of the two classes of methods above is made with the so-called hybrid methods. These combine numerical input/output from analytical models with statistical and probabilistic data to define exposure and vulnerability distribution. This allows the analytical burden to be reduced while grounding results in a geographical context. Applicability and reliability of such approaches are constrained by the ability to define certain mechanical and structural characteristics in numerical terms and by the need to define a common approach for treating the various sources of uncertainty associated with vulnerability, exposure and hazard. This is further discussed in Section 13.2.3.

The review and discussion of existing methods sets out the rationale for a combined method, called FaMIVE (Failure Mechanism Identification and Vulnerability Evaluation), and developed by the author and her co-researchers, in the past decade. The hypotheses and analytical approach to derive capacity curves and fragility functions are presented in detail in Section 13.3. Results obtained from the FaMIVE method are presented in Section 13.3.2. in terms of damage limit states and drift, and compared with the recommendation of Eurocode 8 and the experimental evidence to discuss issues of validation and calibration. The relevance of different categories of input parameters is discussed with respect to mean capacity curves for different cases. A method for deriving performance points in the acceleration/displacement response spectra plane is introduced and results are evaluated in terms of Eurocode 8 recommendations on knowledge level and confidence factors in Section 13.3.3. Finally, fragility curves are derived for various cases (e.g. different locations and limit states). A discussion on treatment of various sources of uncertainties is presented in Section 13.4 and conclusions and future research needs are summarised in Section 13.5.

13.2 Vulnerability approaches: empirical and analytical

13.2.1 Empirical methods

The damage probability matrix (DPM) method has been the most common vulnerability assessment method in Europe. Based on field observations, it expresses in a discrete form the conditional probability of obtaining a damage level j , due to a ground motion of macro-intensity i , $P(D) = P[D = D_j/I_i]$. The method relies on the wealth of observed damage data available from past earthquakes, and the correlation of these with construction materials and methods in different geographical and seismic regions. First proposed by Whitman *et al.* (1973) after the San Fernando earthquake of 1971, specific applications of this method to masonry structures are numerous (Braga *et al.*, 1982; Corsanego and Petrini, 1990; Di Pasquale *et al.*, 2005) and still widely used in developing countries and regions with extensive historic seismicity records (Askan and Yucemen, 2010; Zobin *et al.*, 2010). Notwithstanding its popularity, the DPM has major limitations: discrete definition of damage levels/states and dependence on a specific seismic and architectural context. Hence, it may not be applicable to different geographic locations, in the absence of direct damage data.

Some of these limitations are overcome by the vulnerability index method (VIM) (Benedetti *et al.*, 1988), based on the vulnerability index I_v . The vulnerability index is a summation of weighted parameters, each associated with a constructional or structural feature of the typology, which affects its vulnerability. In this way the definition of vulnerability relates only to the mechanical characteristics of the building, while damage data from past earthquakes are used to calibrate the vulnerability functions, by relating I_v to observed global damage levels for buildings of the same typology, and hence extending applicability in regions having experienced the same level of macro-seismic intensity or peak ground acceleration (PGA).

The most substantial improvement of the VIM over the DPM is that the former provides a continuous vulnerability function, while the latter uses discrete vulnerability classes expressed in terms of expected damage. This means that possible interventions to shift the vulnerability level of a structural typology are readily quantifiable with I_v , while they are only recorded in the DPM if the intervention results in a shift of damage class. Moreover, the correlation of VIM with an instrumental measure of seismic action allows for application to and comparison between different seismic zones, even though in the last decade the reliability of correlating PGA and damage has been questioned by various authors (Elenas, 2000; Wu *et al.*, 2003). Giovinazzi and Lagomarsino (2004) have proposed a direct correlation of the two methods by introducing a definition of damage states and DPM as a function of I_v according to the EMS-98 macroseismic scale

(Grünthal, 1998). This approach has been further developed in the European RISK-UE project for a larger number of building typologies and vulnerability classes (Lagomarsino and Giovinazzi, 2006). The EMS-98 scale (Grünthal, 1998) is in itself a DPM approach, where vulnerability classes are defined across different construction typologies. The uncertainty inherent of attribution of a typology to a class and of its response to a seismic event in terms of damage level is dealt with by assuming a central value and a fuzzy set membership, so that both vulnerability and damage can be expressed in verbal (few, many, most) rather than numerical terms.

A first attempt at deriving generic fragility curves was made by Spence *et al.* (1992) with the parameterless scale intensity (PSI), aimed at providing fragility curves for different building typologies and at quantifying benefits from upgrading and strengthening interventions of the building stock. The advantage of this approach is that damage grades and fragility curves are independent of macro-seismic intensity scales, as they are defined relative to each other, on the assumption that for each building type it is possible to define the level of the scale corresponding to the median of the fragility curve for level of damage D3 (structural damage). Limitations of applicability are, however, related to the extensive need for observations to calibrate each fragility curve and to the difficulty in translating observed or expected damage into structural characteristics. Recent applications based on this methodology were developed in the works of Sabetta *et al.* (1998) and Rota *et al.* (2006).

13.2.2 Analytical methods

With respect to the empirical methods, analytical ones have the advantage of framing the problem of seismic vulnerability of masonry structures in structural engineering terms, defining a direct relationship among construction characteristics, structural response to seismic action and damage effects. The development of attenuation equations for specific seismic regions and corresponding derivation of seismic hazard maps in terms of spectral ordinates, as opposed to macro-seismic intensity or PGA, has given impetus to the development of analytical methods. These methods tend to feature more detailed and transparent vulnerability assessment algorithms with direct physical meaning. In the development of analytical fragility functions for masonry structures, two methods can be identified: (i) correlation between damage index and damage thresholds and (ii) correlation between acceleration/displacement capacity curves and spectral demand curves, following the HAZUS-MH (FEMA, 1999) or N2 method (Fajfar and Gašperšič, 1996). These two procedures, in line with the performance-based design for new builds, define the seismic assessment of existing building stock and prediction of losses. The required analysis steps are: (i) classification of

buildings by typology and seismic design; (ii) definition of damage states; (iii) assignment of capacity curves; (iv) definition of demand spectra (associated with return periods and performance targets); and (v) evaluation of building response in terms of performance points. The two procedures, applied to large sets of buildings, allow the derivation of fragility curves and damage scenarios for given sites and as a function of ground motion parameters which are linked with hazard levels. The damage levels are directly related to the demand parameter, which is expressed in terms of displacement or drift. As the fragility curves are developed on the basis of lognormal distribution, once the typologies have been defined, it is not necessary to have a detailed knowledge of the building stock. All that is required are the parameters defining the capacity curve for each typology, various damage thresholds, and the number of buildings belonging to each typology. Once capacity curves have been developed then the range of behaviour and variability of fragility within a building class or stock can be analysed by parametric analysis, providing important insight for retrofitting (see D'Ayala, 2005).

Analytical approaches to define seismic vulnerability of masonry buildings are becoming more and more popular, as improved engineering knowledge of the behaviour of masonry structures increases confidence in the reliability of such models. In the past decade, a relatively significant number of procedures aimed at defining reliable analytical vulnerability functions for masonry structures in urban context have been proposed (Erdik *et al.*, 2003; Lang and Bachmann, 2004; Borzi *et al.*, 2008; Erberik, 2008). Although they share similar conceptual hypotheses, they differ substantially in modelling/numerical complexity and treatment of uncertainties. Two main limitations are common to all: limited geographic applicability and limited number of failure mechanisms. Specifically, for the latter, most of the analytical models consider only in-plane or frame-like behaviour, disregarding overturning and out-of-plane mechanisms, often occurring at lower levels of shaking and hence substantially affecting the seismic vulnerability in an urban context. Analytical vulnerability assessment of existing masonry buildings features in many risk assessment integrated systems, from HAZUS-MH (FEMA, 1999), to SELENA (Molina *et al.*, 2009), to ELER (Demircioglu *et al.*, 2009); however, their choices of capacity curves are predetermined.

Among nonlinear analysis methods, the collapse-mechanism method presents the advantage of requiring few input parameters and allowing consideration of different failure modes. Procedures using this approach are based on collapse multipliers which identify the occurrence of different possible mechanisms for the given typology and structural characteristics. Among the first to use this approach was Bernardini *et al.* (1990) who developed a numerical routine (VULNUS), combined with a fuzzy set

theory, in recognition of the limited knowledge associated with many of the parameters. Only one in-plane and one simple out-of-plane mechanism were initially considered as possible failures. D'Ayala *et al.* (1997) proposed a validation of the mechanism approach with extensive *in situ* observation and correlation of the seismic collapse load factor with the PGA, proving the validity of the approach through a case study for the historic centre of Lisbon. D'Ayala and Speranza (2003) further enhanced this approach by developing the FaMIVE method, a mechanical approach based on a suite of 12 possible failure mechanisms directly correlated with *in situ* observed damage (D'Ayala and Kansal, 2004) and laboratory experimental validation (Shi *et al.*, 2008; D'Ayala and Shi, 2011). The FaMIVE algorithm produces vulnerability functions for different building typologies and quantifies the effect of strengthening and repair intervention on reduction of vulnerability. The method has been applied in Nocera Umbra, Italy, following the 1997 earthquake (D'Ayala and Speranza, 2003), in Istanbul, Turkey; in Bhuj, India, following the 2003 Gujarati earthquake; and in Abruzzo, Italy, following the 2009 L'Aquila earthquake (D'Ayala and Paganoni, 2011). The FaMIVE procedure has also been applied in conjunction with the VULNUS approach and the AeDes (Baggio *et al.*, 2009) and EMS98 (Grünthal, 1998) vulnerability classes (Bernardini *et al.*, 2008).

13.2.3 Hybrid methods

Several researchers have used a hybrid method by combining post-earthquake damage statistics with analytically derived nonlinear behaviour using pushover analysis (e.g. Barbat *et al.*, 1996; Kappos *et al.*, 1998). Recently, Kappos *et al.* (2008) have applied it to estimating direct losses for masonry buildings in Thessaloniki, Greece. This procedure is very useful when the damage data collection of an area of interest with a specific intensity is only partially available. However, the use of different data sources derived from different procedures, for which a direct cross-correlation is not readily available, might not necessarily result in reduced uncertainty of the output.

An application of this approach to masonry structure within the Risk-UE project shows the importance of knowledge of the building stock for reliable seismic vulnerability assessment, as increased level of uncertainties are compounded both on capacity and demand curves (Lagomarsino and Giovinazzi, 2006). Application made using code prescription in terms of capacity curves provides a good first approximation but needs extensive *in situ* calibration, as shown by Erberik (2008).

A summary of the methods reviewed above is presented in Table 13.1. While this is by no means an exhaustive list of the many applications of seismic vulnerability assessment for masonry structures available in the literature, it concentrates on procedures that have been specifically

Table 13.1 Summary of procedures for the seismic vulnerability assessment of masonry structures

Method	Reference	Data requirement	Data collection	Assessment type	Approach	Demand input	Output
VULNUS	Bernardini <i>et al.</i> (1990)	Geometry, material parameters, structural details	On site observation or systematic survey	Analytical	Mechanism method	PGA, response spectrum	Lateral force, lateral drift, vulnerability index
FaMIVE	D'Ayala (2005) D'Ayala and Speranza (2003)	Geometry, material parameters, structural details	On site observation or systematic survey	Analytical	Capacity spectrum-based method, mechanism	PGA, response spectrum	Lateral force lateral drift, damage scale, vulnerability index
SP-BELA	Borzi <i>et al.</i> (2008)	Structural description	Random generation	Analytical	Push-over based method	PGA, response spectrum	Lateral force, lateral drift
CSBM	RISK-UE, Kappos <i>et al.</i> (2008)	Structural description	Systematic survey and random generation	Analytical	Capacity spectrum-based method	PGA, response spectrum	Lateral force, lateral drift
SELENA	Molina <i>et al.</i> (2009)	Typological description, structural description	On site observation and random generation	Analytical	Capacity spectrum-based method	PGA, response spectrum	Lateral force, lateral drift
HAZUS-MH	FEMA (1999)	Typological description, structural description	Exposure database, systematic survey	Empirical/ analytical	Capacity curves method	PGA, response spectrum	Lateral force, lateral drift

VIM	Benedetti <i>et al.</i> (1988), Lagomarsino and Giovinazzi (2006)	Typological description	On site observation	Empirical	Vulnerability index method	Macro-seismic intensity, PGA	Phenomenological damage scale
DPM	Whitman <i>et al.</i> (1973)	Typological description	On site observation	Empirical	Damage probability matrix	Macro-seismic intensity	Phenomenological damage scale
AeDES	Baggio <i>et al.</i> (2009)	Typological description	On site observation	Empirical	Vulnerability index method	Macro-seismic intensity	Phenomenological damage scale
PSI	Spence <i>et al.</i> (1992)	Typological description	Exposure database	Empirical statistical	Damage probability function	Macro-seismic intensity	Phenomenological damage scale
Hybrid methods	Kappos <i>et al.</i> (1998), Barbat <i>et al.</i> (1996)	Geometry, material	On site observation	Empirical/ analytical, statistical basis	Hybrid methods	Macro-seismic intensity	Damage scale lateral forces

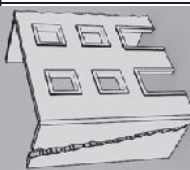
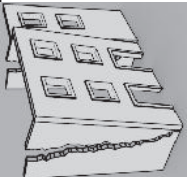
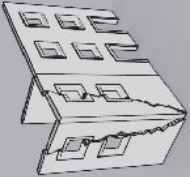
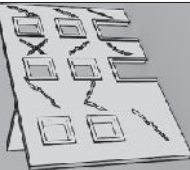
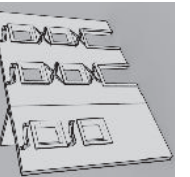
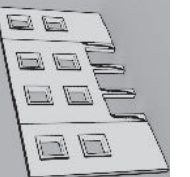
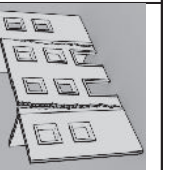
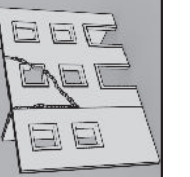
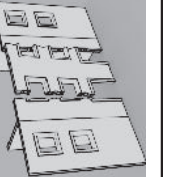
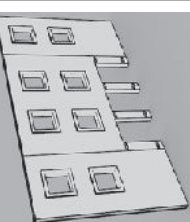
developed for vulnerability assessment at territorial scale. For this reason procedures aimed at the assessment of single buildings are not included. The choice of the most suitable procedure is highly dependent on the resources available for the data collection, the computational expertise available, and ultimately the scale and aim of the study. Empirical procedures can be used for fairly large-scale studies to define damage scenarios; however, if the purpose of the study is to identify within a district or urban centre specific buildings in need of strengthening, so as to increase their seismic resistance, then a suitable analytical procedure should be preferred.

13.3 Collapse-mechanism approach to seismic vulnerability of masonry buildings

In defining a capacity spectrum-based approach for masonry structures, the major difficulty is in devising a procedure sufficiently ‘open’ or ‘flexible’ to accommodate the variability in construction typical of masonry building. Such variability relates to geometry, material and structural parameters, and results in a relatively large number of possible failure modes; these factors should be considered for a realistic assessment of the vulnerability of a building population.

D’Ayala (2005) has proposed a version of FaMIVE that uses the collapse mechanism approach to derive capacity curves, directly related to the failure modes of a building. The post-elastic displacement is quantified in terms of simple stability considerations and bilinear pushover curves are obtained. Considering that performance assessment in terms of expected displacement better represents the post-elastic behaviour of the building, the approach has been implemented by defining limit displacement conditions and compares these values to displacement demands obtained through inelastic displacement spectra. Since the mechanical approach allows us to evaluate the capacity of the structure as an analytical function with a small number of geometrical and mechanical parameters, it is possible to apply a suitable procedure for the uncertainty propagation to distinctly quantify the uncertainties associated both with the ‘capacity’ (model error, parameter uncertainties, uncertainties on damage states) and with the ‘demand’ (seismic input uncertainty). In the following, uncertainties associated with the model and the capacity curves are further discussed.

Based on the evidence collected from post-earthquake site surveys, D’Ayala *et al.* (2008) have identified a suite of 12 possible modes of failure, as shown in Fig. 13.1. Each mode of failure corresponds to different constraints between the façade and the rest of the structure, hence a collapse mechanism can be univocally defined and its collapse load multiplier computed.

Combined mechanisms					
	B1: façade overturning with one side wall		B2: façade overturning with two side walls		C: overturning with diagonal cracks involving corners
	H1: diagonal cracks mainly in piers		H2: diagonal cracks mainly in spandrel		M1: soft storey due to shear
	A: façade overturning with vertical cracks		D: façade overturning with diagonal crack		E: façade overturning with crack at spandrels
	F: overturning constrained by ring beams or ties		M2: soft storey due to bending		G: façade overturning with diagonal cracks

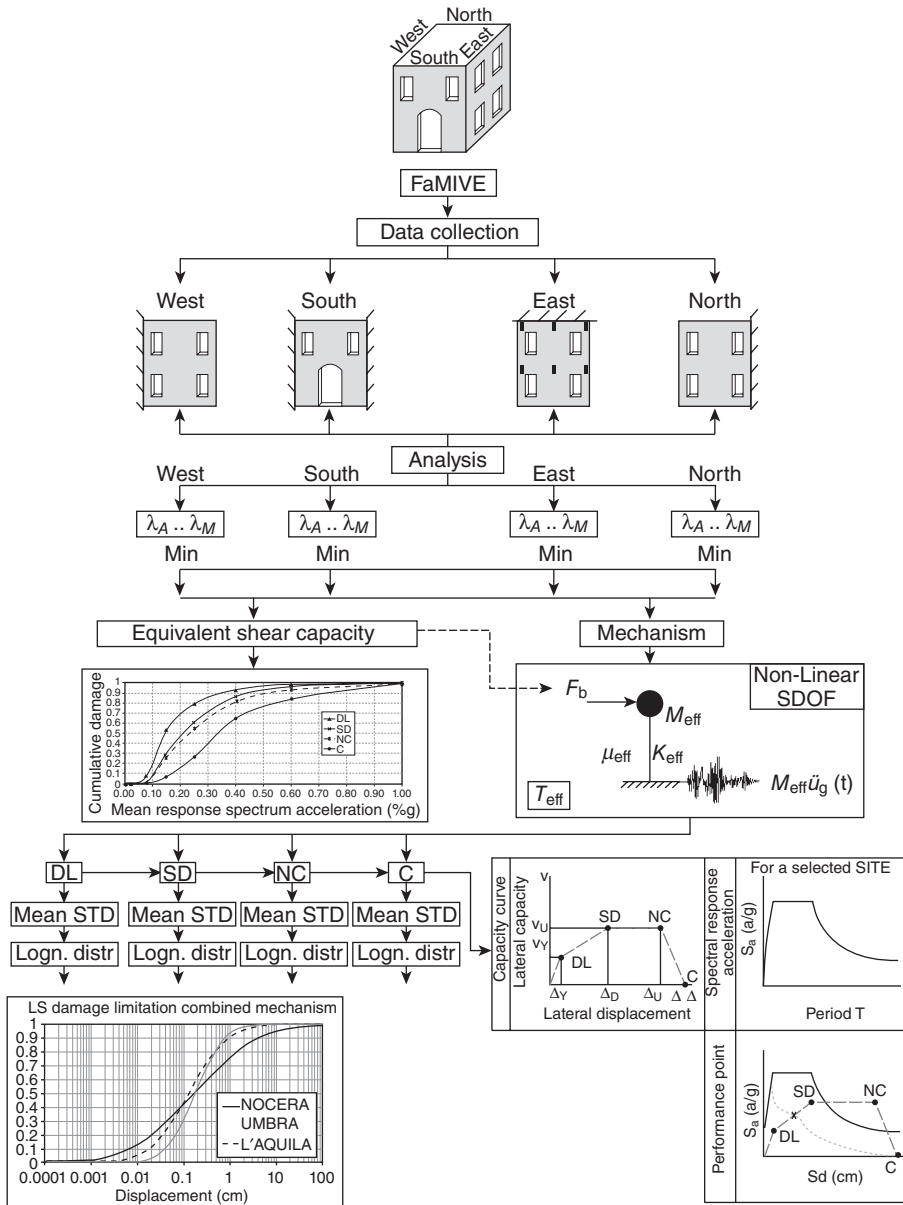
13.1 Mechanisms for computation of limit lateral capacity of masonry façades.

The FaMIVE method first calculates the collapse load multiplier for each possible mechanism of each façade in a building, given geometric and structural characteristics and constraints. To choose the most critical failure mode leading to the greater possible loss (i.e. the most vulnerable condition for the façade or building), the collapse multiplier for each failure mode is weighted by the mass of walls and floors involved in the mechanism. The characteristics of the identified mechanism are then used to derive an equivalent nonlinear single-degree-of-freedom (SDOF) capacity curve to be compared to a seismic demand, and eventually to define performance points, as illustrated in the flowchart in Fig. 13.2.

In Fig. 13.2, λ_i represents the collapse load multiplier for each possible failure mode, and can be interpreted as the lateral acceleration capacity of the structure. Hence fragility curves in terms of lateral acceleration capacity can be directly derived. Alternatively, having defined an equivalent nonlinear SDOF, and its representative capacity curves, four damage limit states are computed, corresponding to damage limitation (DL), structural damage (SD), near collapse (NC) and collapse (C), respectively. Details on the derivation of the capacity curves and the computation of the limit states are provided in Section 13.3.2 and 13.3.3.

In order to derive fragility curves, the next step consists of defining limit state performance criteria to be correlated to damage states. This step involves uncertainties, as very limited evidence exists to establish such correlation over a wide range of building typologies and shaking levels. While robust databases of damage states exist in the literature, no attempt has been made so far to record permanent drift and corresponding ground shaking in a consistent way, so as to provide empirical evidence for capacity curves. As an alternative, a number of authors have worked on correlating performance indicators and damage indicators to obtain capacity curves experimentally, by way of shaking table tests or push-over test (e.g. Paquette and Bruneau, 2006; Meisl *et al.*, 2007; Magenes *et al.*, 2010).

Paquette and Bruneau (2006) and Magenes *et al.* (2010) have performed tests to define in-plane behaviour of masonry house-models. Several issues arise when extrapolating such data to building stock performance. The first problem is the scaling usually applied to specimens tested on shaking tables, as full scale of more than two storey structures are too large to be tested in most laboratories. The reduced scale invariably affects the strength of the materials, and the stiffness of the overall structure. A second issue is the limited representativeness of a particular masonry fabric (brickwork or stonework, dressed or rubble) and the limited number of parameters studied in any experimental investigations (such as slenderness of piers and layout of openings). Most importantly the majority of those tests have been carried out focusing only on the capacity of in-plane walls, while very limited experimental work has been conducted on the characterisation of out-of-



13.2 Flowchart of the FaMIVE procedure.

plane capacity for unreinforced masonry (URM) structures. Typically such studies are related to the failure of infill in concrete frames rather than the behaviour of load-bearing masonry structures. Meisl *et al.* (2007) considered the out-of-plane failure of URM bearing walls constrained by flexible

Table 13.2 Performance drift value for damage limit states

Limit state	In-plane experimental	Eurocode 8 Part 3	Out-of-plane experimental	Eurocode 8 Part 3
Damage limitation (DL)	0.18–0.23		0.33	
Significant damage (SD)	0.65–0.90	0.4–0.6	0.88	0.8–1.2 (H_0/D)*
Near collapse (NC)	1.23–1.92	0.53–0.8	2.3	1.06–1.60 (H_0/D)

*According to EC8 definition H_0 is the distance between the section where the flexural capacity is attained and the contraflexure point and D is the in-plane horizontal dimension of the wall (depth); so that their ratio represents the slenderness of the wall.

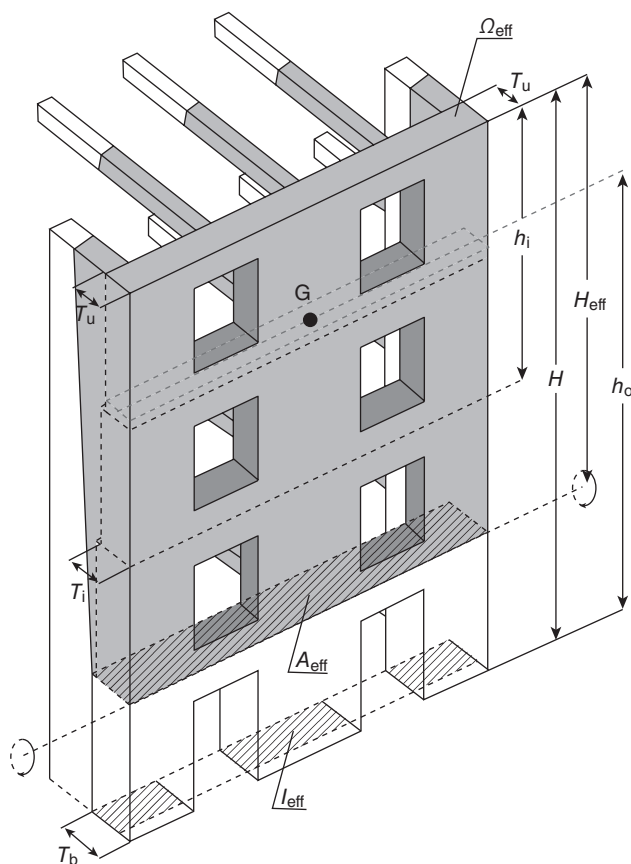
diaphragm; however, the support conditions predefine the failure mode with three horizontal cylindrical hinges (see also Griffith *et al.*, 2004), and were rather different from on site and laboratory observations collected by D'Ayala *et al.* (2008). A testing scheme, corroborated by observation of post-earthquake damage in existing masonry structures, has been devised by Al Shawa *et al.* (2012); however, by predefining a state of damage and a consequent separation between façade wall and transversal walls, the mechanism is also in this case predefined.

Ranges of existence for drift limit states can be computed for either in-plane or out-of plane prevalent behaviour, using available experimental datasets in the literature. These are summarised in Table 13.2 and compared with the EC8 (CEN, 2005) values provided for URM for the damage limit states of SD and NC. These values will be compared with the value of performance drift obtained with the FaMIVE simulations for the damage thresholds of structural damage and near collapse, respectively as defined in Fig. 13.1 and above.

13.3.1 Derivation of capacity curves

The capacity curves are defined for each façade on the basis of the following steps. The first step is to calculate the lateral effective stiffness for each wall and its tributary mass. The effective stiffness for a wall is calculated on the basis of the type of mechanism attained, geometry of the wall (Fig. 13.3), layout of opening, constraints to other walls and floors, and portion of other walls involved in the mechanism:

$$K_{\text{eff}} = k_1 \frac{E_t I_{\text{eff}}}{H_{\text{eff}}^3} + k_2 \frac{E_t A_{\text{eff}}}{H_{\text{eff}}} \quad [13.1]$$



13.3 Schematics of the meanings of the parameters used in the calculation of the effective stiffness and effective mass for the equivalent degree of freedom and in equations (13.2) to (13.4).

where, with reference to Fig. 13.3, H_{eff} is the height of the portion involved in the mechanism; E_t is the estimated elastic modulus of the masonry as it can be obtained from experimental literature for different masonry typologies; I_{eff} and A_{eff} are the second moment of area and the cross-sectional area, respectively, calculated by taking into account extent and position of openings and variation of thickness over height; k_1 and k_2 are constants which assume different values depending on edge constraints and whether shear and flexural stiffness are relevant for the specific mechanism.

The effective mass Ω_{eff} is calculated by following the same approach and it includes the portion of the elevation activated by the mechanisms plus the mass of the horizontal structures involved in the mechanism (shown in

grey in Fig. 13.3). Effective mass and effective stiffness are used to calculate a natural period, for an equivalent SDOF oscillator. The mass is applied at the height of the centre of gravity of the collapsing portion. The elastic limit acceleration A_y is identified as the combination of lateral and gravitational loads that will cause a triangular distribution of compression stresses at the base of the overturning portion, just before the onset of partialisation:

$$A_y = \frac{t_b}{6h_o} g \text{ with corresponding displacement } \Delta_y = \frac{T^2}{4\pi^2} A_y \quad [13.2]$$

where t_b is the effective thickness of the wall at the base of the overturning portion, h_o is the height of the centre of mass of the overturning portion, and T is the natural period of the equivalent SDOF oscillator. The maximum lateral capacity A_u is defined as:

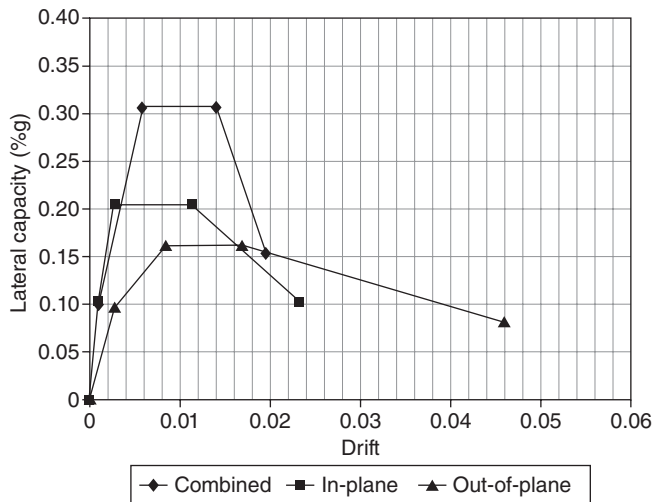
$$A_u = \frac{\lambda_c}{\alpha_1} \quad [13.3]$$

where λ_c is the load factor of the collapse mechanism chosen, calculated by FaMIVE, and α_1 is the proportion of total mass participating to the mechanism. This is calculated as the ratio of the total mass of the façade and sides or internal walls and floor involved in the mechanism, to the total mass of the structure. The corresponding displacement is identified by the condition of loss of vertical equilibrium which, can be computed as a lateral displacement of the top of the wall:

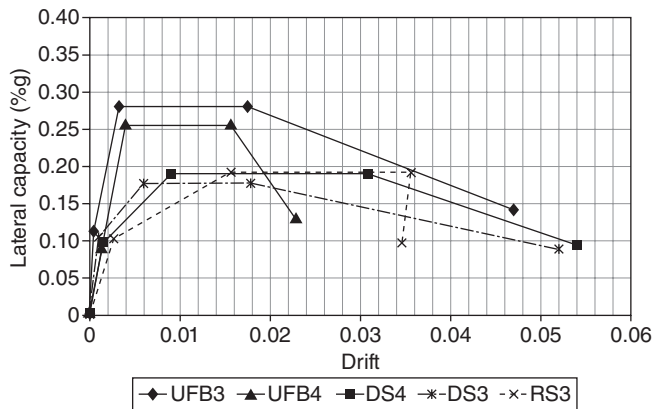
$$\Delta_u \geq t_b/3 \text{ or } \Delta_u \geq l/2 \quad [13.4]$$

Equation (13.3) and (13.4) provide the coordinates of the performance point for the NC condition. An intermediate point between (Δ_y, A_y) and (Δ_u, A_u) is also identified, corresponding to the position of the resultant of stresses for the fully partialised cross-section at the base. If a parabolic stress block is assumed, the corresponding relative displacement of the top to the base is $\Delta_{sd} \approx t_b/6$. This deflection corresponds to the attainment of the maximum base shear A_u , hence (Δ_{sd}, A_u) are the coordinates of the performance point representing structural damage.

Because the FaMIVE procedure is based on data directly collected on site, one capacity curve can be defined for each façade surveyed or for each building surveyed, when more than one façade for building is considered. The procedure also allows a direct analysis of the influence of different parameters on the capacity curves (e.g. geometrical/mechanical/structural characteristics). Figure 13.4 shows a comparison of average capacity curves by grouping the results based on different criteria for the same type of buildings. In Fig. 13.4a, the average curves are obtained by considering



(a)



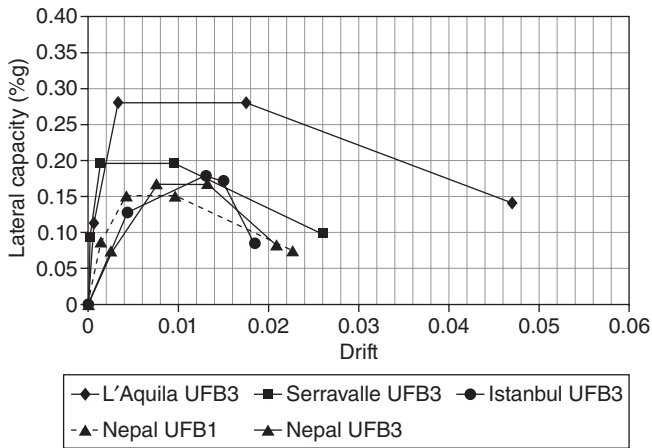
(b)

13.4 (a) average capacity curves for sample grouped by collapse mechanism classes, and (b) average capacity curves for sample grouped by structural typology (UFB stands for unreinforced fired brickwork, DS stands for dressed stone, RS for rubble stone. The number refers to different type of binders and/or floor structure).

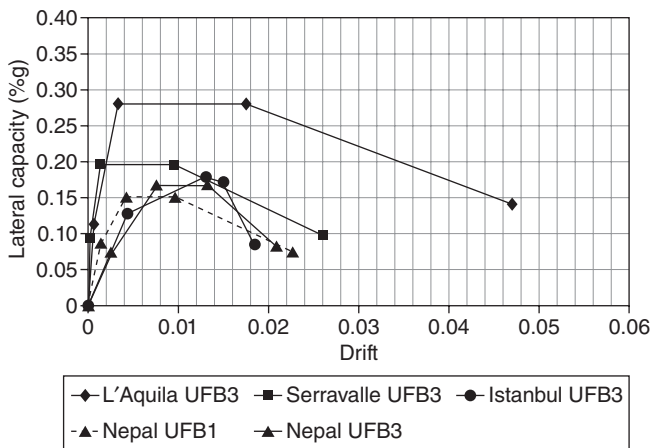
whether failure occurs by out-of-plane, in-plane or a combined mechanism involving both series of walls. In Fig. 13.4, the capacity curves are obtained by considering different structural typologies, as classified by the WHE-PAGER project (Jaiswal *et al.*, 2011). It can be seen that the correlation between mode of failure and structural typology is qualitatively good but not univocal, and the grouping affects both acceleration capacity and drift.

Substantial differences also exist for the same structural typology from different regional settings. In Fig. 13.5, average capacity curves for structural typologies based on unreinforced brickwork with different mortars and horizontal structures are compared for different sites, two in Italy, one in Turkey, and one in Nepal.

The diagrams show that the location, and hence construction details, has greater influence than structural typology on the resulting curves, in particular for sub-typology UFB5, masonry brickwork with reinforced concrete horizontal slab, characterised by the large dispersion of results. This



(a)



(b)

13.5 Average capacity curves for different locations and masonry typologies.

typology in historic centres is often the result of substantial alterations of more traditional typologies, which may affect considerably stiffness, lateral capacity and ductility. These results give sharper focus to the limitation and inaccuracy of using idealised models and average curves without adequately considering the uncertainty and variation associated with each case. A substantial variation in the drift associated with various limit states can also be observed. In order to investigate this further, taking into account both regional variation and different structural typologies, besides the prevalent structural behaviour, average values of the calculated drift are presented in Table 13.3 for comparison with the indications and limits provided by the experimental activity and the code provisions.

For the in-plane behaviour the drift threshold values for damage limitation (DL), identified as first cracking and change in stiffness, seems to be lower for the numerical simulations than for the experimental ones, although Italy II and Turkey fall within the experimental range. No provision is offered by the Eurocode 8. For the limit state of significant damage (SD), the code provisions and the experimental data define consecutive intervals,

Table 13.3 Comparison of drift values from the FaMIVE analysis with experimental and code reference ranges

	Limit state	Damage limitation	Significant damage	Near collapse	Collapse
In-plane prevalent behaviour	Eurocode 8 Part 3		0.4–0.6	0.53–0.8	
	<i>Experimental</i>	0.18–0.23	0.65–0.90	1.23–1.92	2.1–2.8
	Nepal	0.069	0.207	1.123	1.843
	Italy I	0.023	0.069	0.990	2.547
	Italy II	0.132	0.397	0.790	1.844
	Turkey	0.226	0.679	1.579	1.801
	Italy III	0.090	0.271	1.127	2.324
Out-of-plane prevalent behaviour	Eurocode 8 Part 3		0.8–1.2 (H_0/D)	1.06–1.60 (H_0/D)	
	<i>Experimental</i>	0.33	0.88	2.3	4.8
	Nepal	0.691	1.383	1.891	2.343
	Italy I	0.263	1.580	1.969	3.401
	Italy II	0.460	1.380	1.714	5.562
	Turkey	0.689	1.378	1.266	2.167
	Italy III	0.280	0.8408	1.693	4.593
Combined behaviour	Nepal	0.168	0.505	0.824	1.114
	Italy I	0.030	0.181	1.005	1.778
	Italy II	0.033	0.099	0.198	3.307
	Turkey	0.102	0.305	0.724	1.201
	Italy III	0.097	0.582	1.401	1.957

within which two of the numerical results are contained, but the others indicate stiffer behaviour (i.e. less drift). The situation is reversed for the near collapse (NC) limit state, while the collapse (C) limit state drift shows a similar pattern. For the out-of plane behaviour the code provision for walls governed by flexure are taken as reference, where the behaviour is a function of the slenderness of the wall, D , being the cross-section dimension normal to the axis of bending and the height of inflexion H_0 . The analytical results show better agreement with the experimental one for all limit states, and also with the code provisions for slenderness greater than 1.5. Finally, under the combined behaviour are grouped failure modes characterised by a three-dimensional response of the building as both the façade and walls perpendicular to it are involved. These modes develop when a good connection among the two sets of walls is guaranteed either by the masonry fabric at junctions of walls or by the presence of ties. This group exhibits greater stiffness than the previous two and hence the lower values of drift for damage limitation. However, given the higher level of redundancy, larger ductility for near-collapse can be achieved.

13.3.2 Calculation of performance points and correlation with damage states

Drift is only one aspect of structural responses that can be used to assess the vulnerability of structures. The lateral acceleration capacity and the relative proportion of drift for the three limit states identified in the previous section are essential indicators of the seismic performance. The overall behaviour of structures can be assessed by computing the performance point (i.e. intersection between capacity curve and demand curve). In order to calculate the performance point it is necessary to intersect the capacity curve derived above with the demand spectra associated with different return periods in relation to the performance criteria considered.

Two broadly equivalent approaches for the derivation of the nonlinear demand spectra exist: the N2 method (Fajfar and Gašperšič, 1996) included in the EC8 and the capacity spectrum method with over-damped spectrum (CSM) (FEMA-356, 2000). The two methods differ essentially in the way the nonlinear demand spectrum is derived: the N2 method uses a reduction factor R as function of the structure expected ductility μ , while the CSM uses an equivalent damping factor derived from the hysteresis loop of the structure. There exists a rich literature that compares the benefits of the two approaches (e.g. Freeman, 2004). In the following discussion, the N2 method will be used to illustrate the derivation of performance points.

To calculate the coordinates of the performance point in the displacement-acceleration space, the intersection of the capacity curve with the

nonlinear demand spectrum for appropriate target level of ductility can be determined as follows, given a value of A_u :

if $T < T_c$

$$\text{if } A_u \geq A_{nl}(T) \Rightarrow SD_{nl}(\mu) = \frac{T_c^2 [\beta A_{el}(0) - A_{nl}(T)]^2}{(\mu - 1)^2} * \frac{g\mu}{4\pi^2 A_{nl}(T)}$$

$$\text{if } A_{nl}(T_c) < A_u < A_{nl}(T) \Rightarrow SD_{nl}(\mu) = \frac{T_c^2 [\beta A_{el}(0) - A_u]^2}{(\mu - 1)^2} * \frac{g\mu}{4\pi^2 A_u}$$

$$\text{if } A_u \leq A_{nl}(T_c) \Rightarrow SD_{nl}(\mu) = \frac{gT_c^2 [\beta A_{el}(0)]^2}{4\pi^2 \mu A_u} \quad [13.5]$$

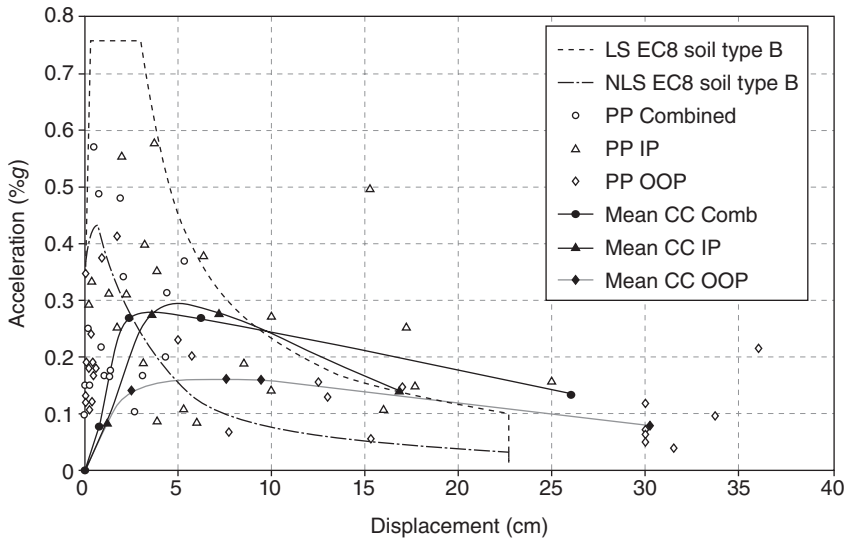
if $T \geq T_c$

$$\text{if } A_u \geq A_{nl}(T) \Rightarrow SD_{nl}(\mu) = \frac{gT_c^2 [\beta A_{el}(0)]^2}{4\pi^2 \mu A_{el}(T)}$$

$$\text{if } A_u < A_{nl}(T) \Rightarrow SD_{nl}(\mu) = \frac{gT_c^2 [\beta A_{el}(0)]^2}{4\pi^2 \mu A_u}$$

where two different formulations are provided for values of ultimate capacity A_u greater or smaller than the nonlinear acceleration $A_{nl}(T_c)$; period T_c identifies the transition from constant acceleration to constant velocity section of the elastic spectrum. In Equation (13.5), SD_{nl} is the nonlinear spectral displacement, as function of the chosen target ductility μ ; β is the acceleration amplification factor calculated as the ratio of the elastic maximum spectral acceleration $A_{el}(T)$ and the peak ground acceleration $A_{el}(0)$ for $T = 0$; $A_{nl}(T)$ is the non-linear spectral acceleration for the value of natural period T that defines the elastic branch of the capacity curve; g is the gravity acceleration. Note that in Equation (13.5) $A_{el}(T)$, $A_{nl}(T)$ and A_u are dimensionless quantities, expressed as a proportion of g .

In Fig. 13.6 the performance points (PP) for the limit state of significant damage (SD) for each building in the Nocera Umbra, Italy, case study are compared with the response spectrum for 475 year return period (or 10% probability of exceedance in 50 years). For the nonlinear spectrum obtained with the N2 method approach a ductility $\mu = 3$ has been chosen in agreement with experimental evidence. It can be seen that there is a significant scatter of performance and although the representative LS points on the mean curves for the three types of behaviour are all beyond the intersection with the spectrum, there is a consistent number of PPs under the curve, equally deficient in terms of acceleration and displacement, especially for the out of plane behaviour. Such outliers should not be overlooked as they usually point to inherent construction deficiency in a regional context, reducing the seismic resilience of a community.



13.6 Performance points in the acceleration/displacement space and average capacity curve for three mechanism types for the significant damage limit state.

13.4 Stochastic and epistemic uncertainty quantification

Advanced uncertainty modelling and probability of occurrence of given phenomena are usually confined to the hazard component of the risk equations, but when probabilistic models are developed for vulnerability components, they usually relate to very simplified modelling of seismic response of the structure. It is hence interesting to analyse the previous results by taking into account the distinction between structural behaviours and locations of the buildings.

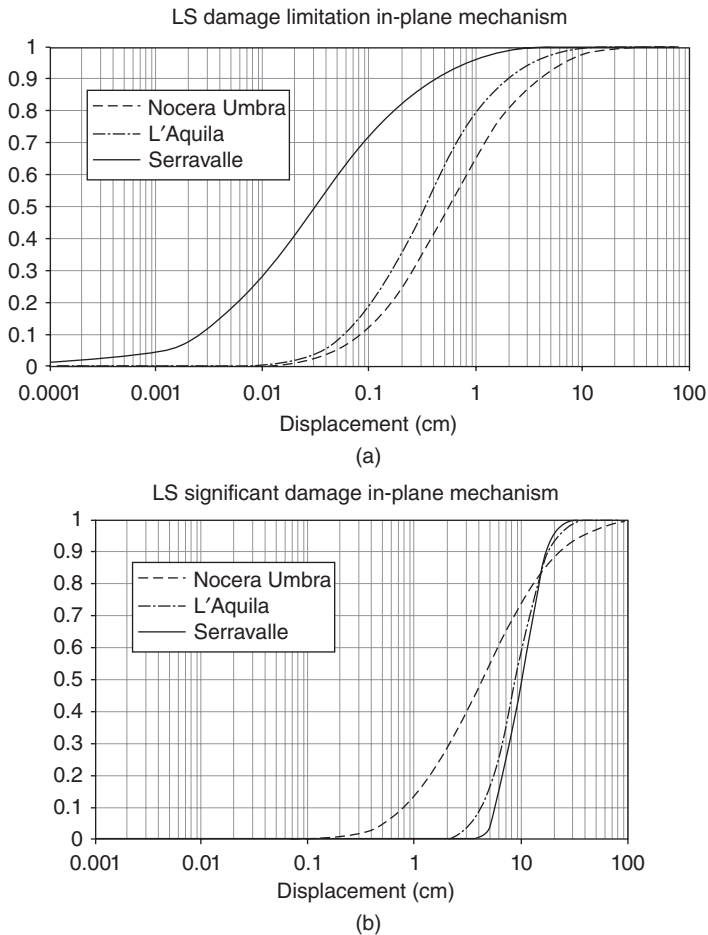
Fragility curves for different limit states are modelled as the lognormal distribution and its parameters are obtained by calculating mean and standard deviation of the performance point displacement. Specifically, the average displacement for each limit state can be calculated as:

$$\bar{\Delta}_{LS} = e^{\mu} \quad \text{with} \quad \mu = \frac{1}{n} \sum (\ln x) \quad [13.6]$$

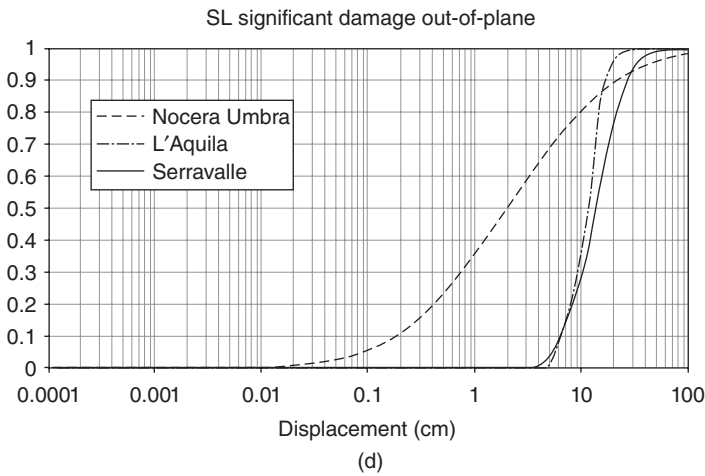
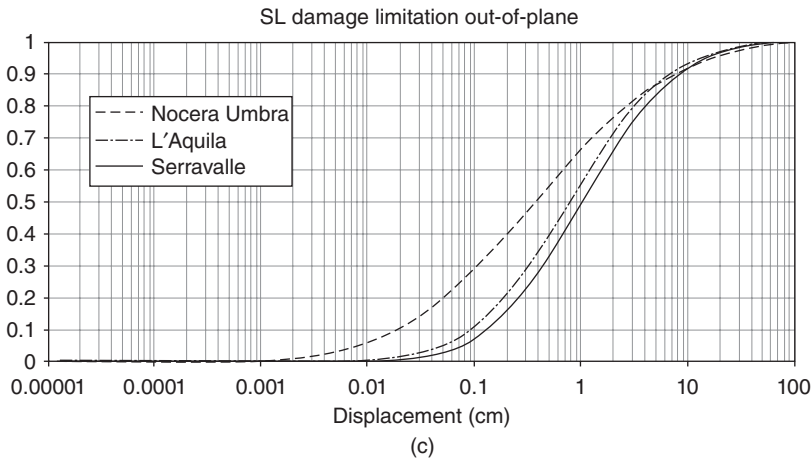
and the corresponding standard deviation as:

$$\beta_{LS} = e^{\mu + \frac{1}{2}\sigma^2} \sqrt{e^{\sigma^2} - 1} \quad \text{with} \quad \sigma = \sqrt{\frac{\sum (\ln x - \ln \bar{x})^2}{n}} \quad [13.7]$$

Figure 13.7 shows the set of fragility curves obtained for the two limit states of DL and SD as computed for three Italian sites considering the three types of structural behaviour separately. It should be noted that while the DL state shows in all cases the lognormal distribution is suitable, the same cannot be said for the SD limit state which shows a narrow range of values for all three classes of mechanism. As, once a structural typology is assigned, values of the mechanical characteristics are the same across the three cases, while the structural details are accounted for directly in the three classes of mechanisms, the variability observed on a same diagram

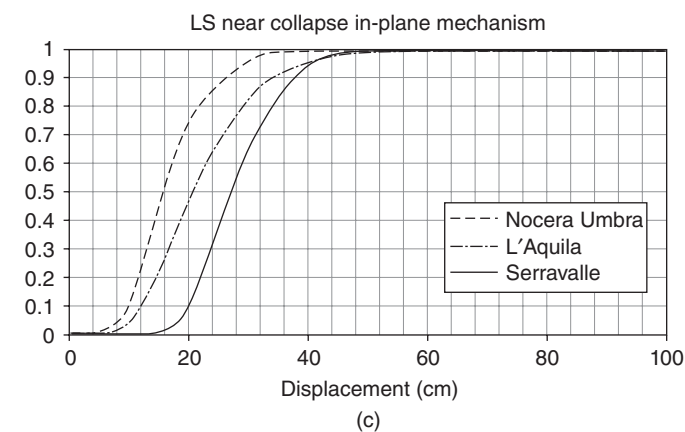
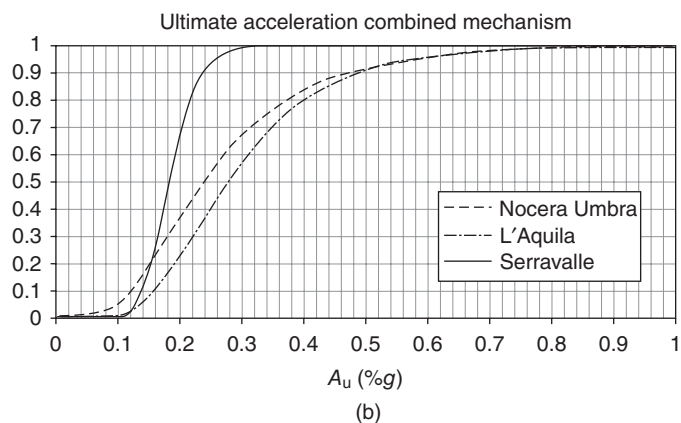
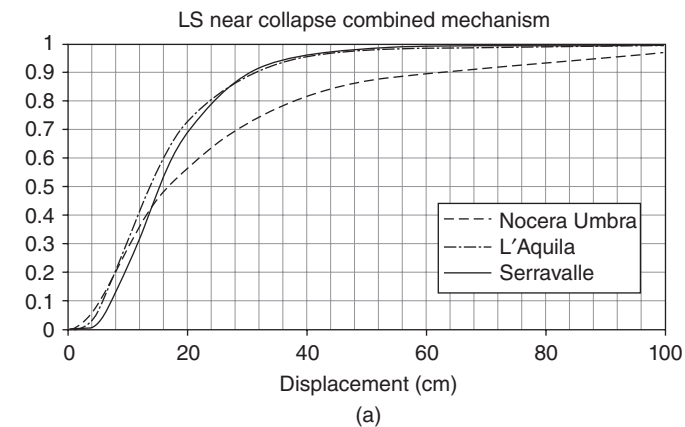


13.7 Fragility distribution for limit states (LS) of damage limitation (DL) and significant damage (SD) for the three classes of collapse mechanisms for three different Italian sites.

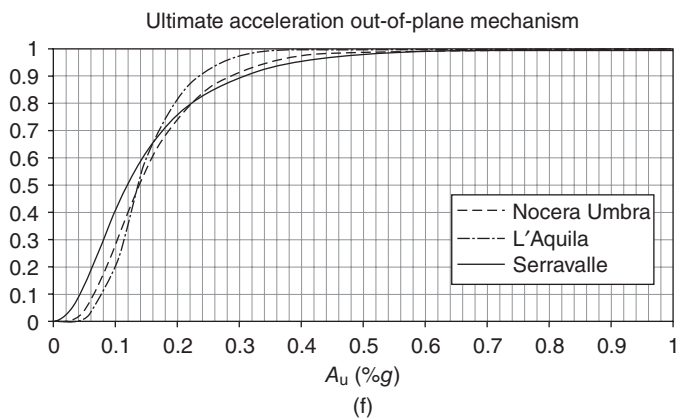
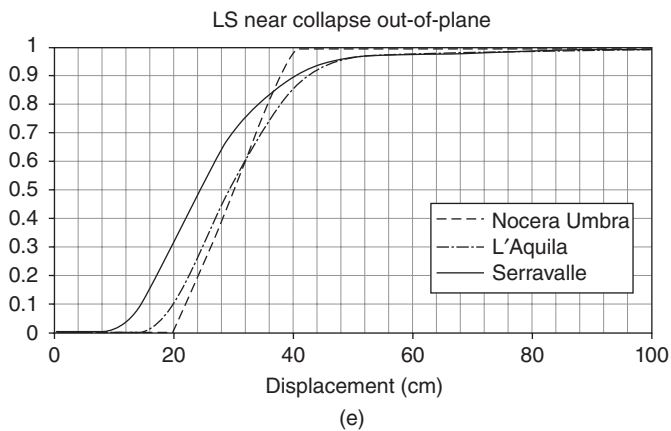
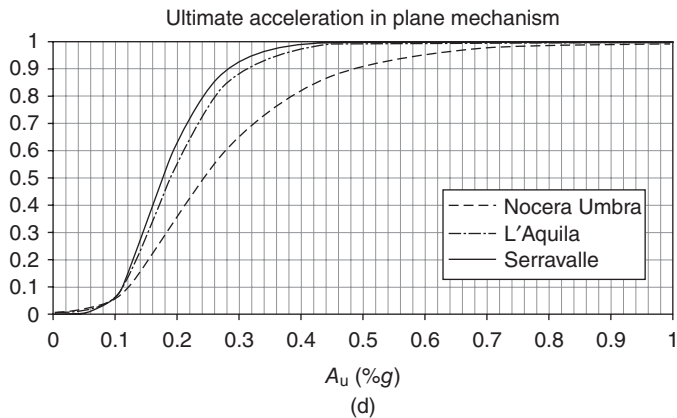


13.7 Continued.

among samples can be related directly to geometric differences at the level of the façade and the masonry fabric. Hence, curves on the right of the diagrams are less vulnerable in the case of DL or have lesser ductility in the case of SD. However, the distribution does not seem to be consistent across the three mechanism classes. Figure 13.8 shows the fragility functions of ultimate conditions as identified in terms of near collapse displacement and ultimate lateral capacity for the three failure mechanisms separately. It can be seen that while there is little difference among the three locations for the out-of-plane behaviour, both in-plane and combined behaviour show high variability. In particular the higher deformability of Serravalle for the in-plane behaviour is related to a high proportion in this case of



13.8 Fragility curves for ultimate conditions NC and A_u for three Italian samples for the three classes of collapse mechanism.



13.8 Continued.

façades with porticoes at ground level, resulting in possible soft storeys. The lower value of average limit displacement for near collapse in Nocera Umbra is to be ascribed to a large proportion of buildings of rubble stone masonry, failing condition (4) due to the small unit size. At the other end, the lower lateral capacity of the Serravalle case for the combined mechanism is associated with more slender façades.

The reliability of such results can be considered within the framework set out in the Eurocode 8, whereby the level of knowledge associated with the results of a seismic assessment of a structure is expressed as a function of the confidence factor. Eurocode 8 recognises three levels of knowledge (limited, normal and full) and three fields of information (geometry, construction details and materials). Data used in the FaMIVE approach are collected by on site visual inspection with some measurement and *in situ* detailed observation of construction details. However only limited *in situ* non-destructive tests of materials are performed and material characteristics are otherwise assigned based on past studies and surveyor experience. The level of knowledge is thus regarded as superior to KL1, *limited*, but not quite equal to KL2, *normal*. For this level of knowledge, a static nonlinear analysis, such as the limit state mechanism approach, leading to a capacity curve is deemed appropriate. Hence according to the recommended values in the Eurocode 8, the confidence factor CF should be in the range 1.2 to 1.35 depending on how close the actual knowledge is with respect to the reference level KL2. The confidence factor is then used to reduce the capacity values as obtained from the assessment.

Although the EC8 approach recognises the importance of treating epistemic uncertainties, the level of knowledge is then translated to a safety factor rather than a probability or possibility of a specific value occurring. While this approach can be considered as acceptable for the assessment of single buildings, it does not account for the natural, organic variation, given the three remits of knowledge discussed above, when considering a large number of buildings – even more so when the buildings are several decades old and have undergone alterations, changes and natural decay.

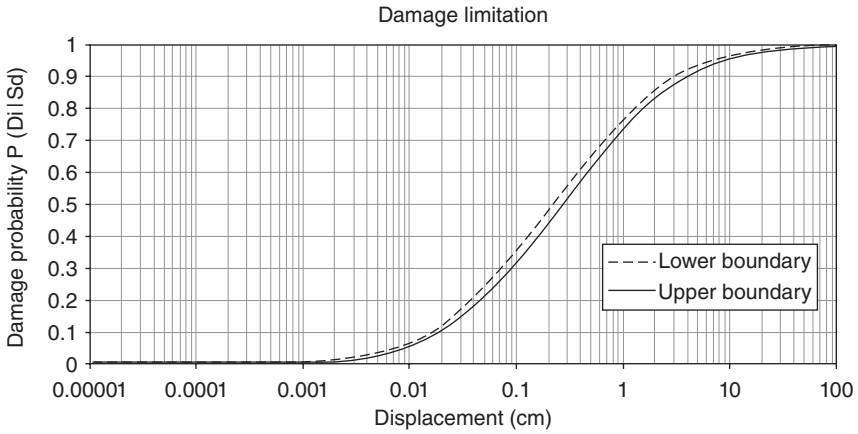
In the FaMIVE procedure a different approach is proposed to define the reliability of the input data as well as the output. Depending on whether data, in each section of the data collection form, have been: (i) collected and measured directly on site, (ii) collected on site and confirmed by existing drawings or photograph, or (iii) collected from photographic evidence only, three levels of reliability are assigned: high, medium and low, respectively. For the three reliability levels, the following confidence ranges for a given parameter are considered: $\mu \pm 5\%$, $\mu \pm 10\%$, and $\mu \pm 15\%$, respectively. The confidence levels associated with the output parameters, ultimate acceleration A_u and limit state displacements Δ_l , are computed as weighted combination of the confidence range of the input parameters.

To quantify the effect of the epistemic uncertainty on the fragility curves obtained with the FaMIVE procedure, cases from the three different locations in Italy and for the different failure behaviours introduced in the previous section, are analysed together. For each case a separate reliability parameter is calculated, as indicated above, and then two new sets of values representing the lower bound and upper bound for each case are computed. For the two sets, logarithmic mean and standard deviation are computed (Equations 13.6 and 13.7) to develop the lognormal models. These are presented in Fig. 13.9 for the three displacement limit states, respectively. The reliability indicator for the entire data is $\pm 11\%$, showing that the data reliability is medium–low, i.e. no availability of drawings in most cases and on site measurement for a modest number of cases. This is a typical situation in the aftermath of an earthquake, such as the conditions in which damage data in Nocera Umbra and L'Aquila were collected.

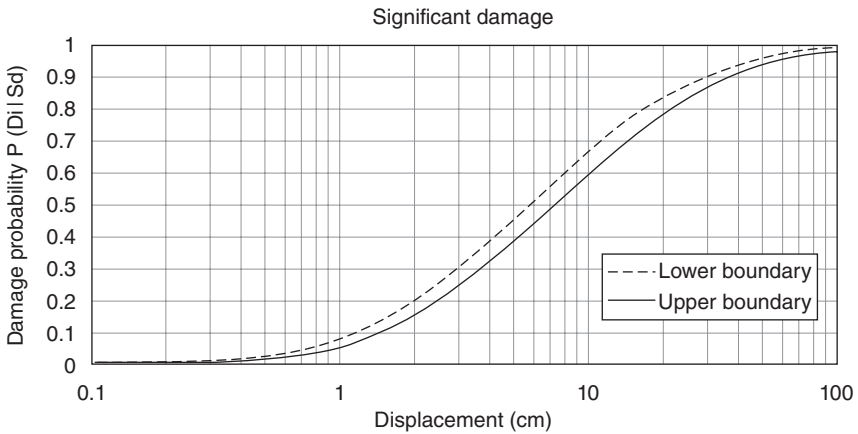
The FaMIVE procedure does not relate a different uncertainty rating to the ability of the methodology to identify the correct class of behaviour given a set of data and, within that class, to identify the correct value of limit state values. To quantify such uncertainty in a robust way, it is necessary to correlate the capability of the FaMIVE method to predict the correct mechanism, with an actual mechanism occurrence in buildings hit by an earthquake. Such analyses carried out for the case of L'Aquila (D'Ayala and Paganoni, 2011) show that the prevalent observed mechanism is correctly predicted in 50% of cases and the alternative most likely in 27% of cases, while possible and local mechanisms represent a minority, and no correct prediction represent 9% of cases. This level of error is comparable to the uncertainty associated with the data input quality as discussed above.

13.5 Conclusions

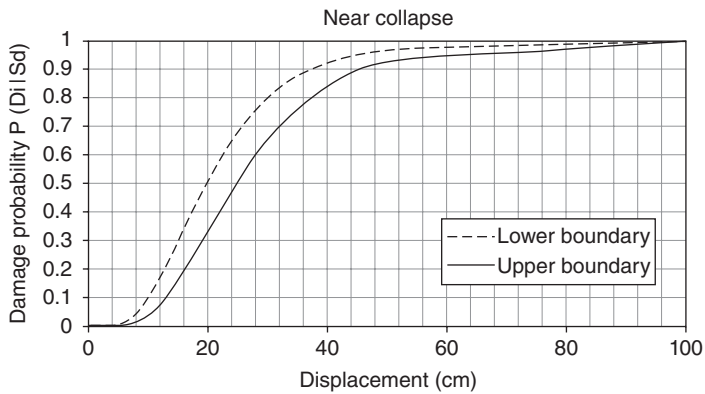
Traditionally seismic vulnerability of historic buildings has been evaluated using empirical methods based either on damage observation or construction data. In the last decade, analytical methods for assessment of masonry structures have increased in number and quality. However, most of them assume only in-plane failure behaviour, reducing the walls to frame-like behaviour. This approach has serious limitations and it is non-conservative, as it overlooks out-of-plane behaviour which might occur for structures with insufficient out-of-plane wall capacity. Nevertheless, recommendations of the Eurocode 8 consider only basic in-plane behaviour, either controlled by flexure or by shear, and set limit drifts for the performance criteria. Comparison with experimental evidence shows that these limits are usually underestimated.



(a)



(b)



(c)

13.9 Effect of epistemic uncertainty on fragility distribution for limit states.

The application of the FaMIVE procedure to a number of cases in regions of different seismicity shows the relevance of construction details and the importance of considering the different failure mechanisms separately to identify realistic risks of failure. The comparison of the computed average drift for each case with the code prescription and the experimental evidence shows that the code prescription underestimates drift for in-plane behaviour, while there is reasonable agreement for the out-of-plane, especially for the near collapse limit state. A procedure for the computation of the performance point given performance criteria and hazard return period is introduced to show the variability of performance points for different failure behaviour and how they affect probability of failure computed as average. The random uncertainty associated with this type of analysis is presented in terms of fragility curves that can be modelled as the lognormal distribution. In the same way, the epistemic uncertainty associated with the modelling procedure is also evaluated using the same approach. Results show that the epistemic uncertainty accounts for approximately 11% variation in results, while comparing results for different locations and failure mechanisms show much greater variability.

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Vulnerability assessment of reinforced concrete structures for fire and earthquake risk

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Abstract: This chapter provides an overview on the performance and vulnerability of structures in fire and earthquake. Two main scenarios of fire and earthquake are explored: (1) when a structure is damaged first by an earthquake and then exposed to a fire (fire following an earthquake) and (2) when a structure is damaged by a fire and therefore its residual seismic capacity needs to be assessed and inspected. Three case studies are presented: one for the fire following an earthquake scenario and two case studies for the second scenario. A large-scale, six-storey reinforced concrete building, previously tested on a shaking table, was selected and numerically evaluated for the scenario of the fire following an earthquake. The results of the analysis show that damage to the structural elements, during an earthquake, could result in degradation of the concrete mechanical properties and could increase heat transfer to the structural elements and therefore lower its residual fire resistance. Two reinforced concrete columns tested previously in fire were chosen and numerically assessed for their residual seismic resistance. Furthermore, a reinforced concrete column was experimentally tested in a fire and then tested under both axial and lateral load to evaluate and inspect its residual lateral load capacity. The results from these two studies indicate that a fire could considerably reduce lateral and axial load capacity as well as ductility of the structures. Finally, some of the research gaps in this area are identified and future studies recommended.

Key words: fire and earthquake, concrete structures, fire damage, seismic damage, fire test, residual lateral resistance.

14.1 Introduction

Various design codes, standards and guidelines for fire or seismic design of structures have been developed in different countries around the world to ensure the safety of occupants in buildings in the event of a fire or an earthquake. Seismic design codes provide tools for design and recommendations for analysis of structures against earthquake, while fire design codes provide requirements for the fire protection and fire resistance of building elements to reduce the risk of structural damage and loss of life in the case of a fire.

However, there is a lack of guidelines and tools for assessment, design and protection of structures for scenarios when a combination of both fire and earthquake are considered as potential threats to the structures. For instance, buildings are not designed explicitly for a fire following an earthquake. There are few reliable post-fire inspection tools to evaluate residual seismic resistance of a structure after a fire incidence (Mostafaei *et al.* 2009). Previous major seismic events have caused devastating damage to structures. The 1755 Lisbon, Portugal, earthquake of a magnitude of 8.5–9.0 triggered both fire and tsunami, and caused ~100 000 deaths; the 1906 San Francisco, United States, earthquake of a magnitude of 7.9 also resulted in widespread fire damage and produced 3000 deaths; the 1995 Kobe, Japan, earthquake with a magnitude of 7.2, followed by fire, caused ~6500 deaths; the 2004 Sumatra, Indian Ocean, earthquake with a magnitude of 9.2 led to catastrophic tsunami damage (more than 300 000 casualties); and the 2011 Tōhoku, Japan, earthquake of a magnitude of 9.0 was accompanied by both fire and tsunami, causing ~20 000 deaths and large economic loss. All of these events signify the importance of considering the impact of the consequent events on the structural performance of buildings.

On the other hand, in the United States, the average annual fire occurrence in moderate and high-rise buildings exceeds 10 000 incidents (Hall, 2001). More than 50 000 fire incidents in Canada have been reported annually (CCFMFC, 2002); approximately 3500 of which were fire incidents that occurred in apartments, hotels and dormitories. These mid- to high-rise structures experienced minor to major damage due to fire exposure, such as column and floor damage due to thermal expansion and degradation of mechanical properties of the material. Therefore, there is also a need to develop reliable tools to assess structural damage after a fire, to measure whether or not the residual seismic load capacities, after the fire damage, are sufficient and if they can satisfy the required seismic load capacity (i.e. original seismic design load).

This chapter provides information about recent studies on the performance evaluation of structures in fire following an earthquake and methodologies for analysis and testing of structures to assess their residual seismic load capacity after they are damaged by a fire. It will focus on providing a summary of research carried out at the National Research Council Canada on vulnerability assessment of structures in fire and earthquake. The main purpose of this research was not to develop design tools or solutions for such threats to structures, but to explore the need for further research and to develop an understanding of how vulnerable structures could be to scenarios of fire and earthquakes. Two case studies were carried out by considering scenarios of structural columns in fire and earthquake. One study simulates a scenario of fire following an earthquake while the other investigates a scenario of an earthquake after fire damage. In the

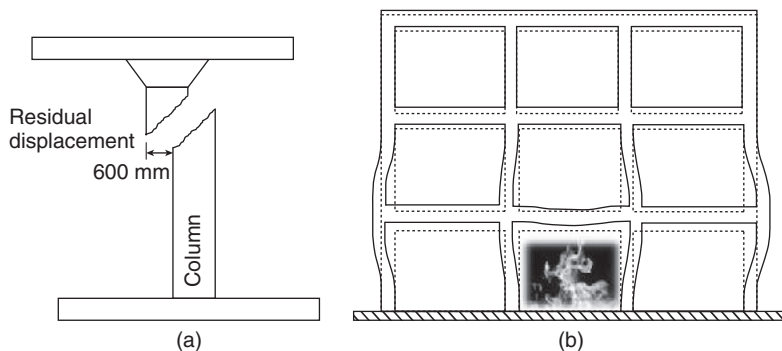
second scenario, the earthquake does not occur right after a fire, as it is less likely; an exception is in aftershocks when a fire has already been ignited due to the main shock. The case study is for a post-fire seismic evaluation scenario to perform a post-fire structural damage evaluation and to assess the residual seismic load capacity of the structures after fire. Although the information provided here would be useful for developing an understanding of the performance of different structural systems in fire and earthquake, the scope of this chapter is limited to reinforced concrete building structures, constructed with normal strength concrete.

14.2 Structural response to fire

Response of structures to fire is typically evaluated using a fire resistance approach. A common approach for fire endurance assessment of structures is to implement a full-scale fire resistance test. One of the first fire resistance/endurance tests was carried out using an experimental method in Denver, Colorado, in 1890 (Harmathy, 1993). There are now standards, such as E119 'Standard Test Methods for Fire Tests of Building Construction and Materials,' that describe how a fire resistance test can be performed. Although application of the new performance-based approach is permitted by some of the design codes, since few assessment tools are available and accessible to practitioners, it is not widely being used for evaluation of fire resistance. A large number of fire resistance tests have been carried out, since the development of the E119 standard. The purpose of these tests was to better understand how structural elements and assemblies, such as columns, beams, floors and walls, perform during a fire. The results of these tests led to the development of requirements for fire resistance design of structures.

There are many factors that contribute to the fire resistance of a building structure; the higher the fire load, the lower the fire resistance. Increase of temperature normally results in the reduction of mechanical properties of the materials, such as compression/tensile strength and modulus of elasticity. Thermal expansion of structural elements is another major effect from fire. It results in inducing large deformation to the structural elements and could cause considerable damage to the structural members even far from the fire compartment. When the structure cools after a fire, it suffers more damage due to shrinkage of the elements from the rapid temperature drop after the fire. Various studies and research have been carried out to assess structural response and damage due to fire. However, there are still gaps in the availability of appropriate tools for performance evaluation and design of the whole structural systems in fire.

A new emerging performance-based fire resistance evaluation of structures emphasizes the consideration of response and behavior of the whole



14.1 Lateral deformation of columns due to the thermal expansion of structures in fire: (a) resembling the column failure during the fire at the Military Personnel Records Centre (USA) and (b) building frame deformation due to thermal expansion.

building in the fire endurance assessment (Mostafaei *et al.* 2011). For instance, current fire resistance standards require only a constant axial load applied during the column fire test. However, effects from lateral deformation due to thermal expansion of floors and change in axial load of the column due to its interaction with the rest of the building are ignored.

Figure 14.1 shows an example of the column's lateral deformation due to the thermal expansion in fire. Figure 14.1a illustrates shear failure of a column on the 6th floor of the US Military Personnel Records Centre building due to the fire that occurred in 1973 (Beitel and Iwankiw 2002), and Figure 14.1b demonstrates how the thermal expansion induces lateral deformation to the columns. Although the response of reinforced concrete columns under lateral loads has been studied for many years, a remaining challenge has been the development of a reliable methodology for evaluating effects of such lateral deformation in performance of the structure in fire.

14.3 Seismic response of structures

Various standards and guidelines have been developed, worldwide, for seismic design of structures. Based on the seismicity and site condition of structures, methodologies have been developed for evaluating response of the structures to the seismic loads and for designing and retrofitting the structures against such a hazard. The different approaches include equivalent static seismic load and analyses, design based on the mode shapes, dynamic and time history analyses.

The main effect from a seismic load on structures is the lateral load induced by the earthquake, except for structures subjected to near field earthquakes. This is normally the main design load required by the codes/standards, since building structures are more vulnerable to the effects from the horizontal wave of the earthquake. This would perhaps be the main reason that most of the research and studies have been focused on protection of structures against structural loads induced by the lateral component of the seismic waves. For instance, for reinforced concrete columns, significant amount of efforts have been made by researchers to develop models for estimating damage and the ultimate load capacity of the columns due to lateral loads, e.g. Park *et al.* (1982), Vecchio and Collins (1986), Lynn *et al.* (1996), Elwood and Moehle (2005) and Mostafaei and Kabeyasawa (2007).

There is still a lack of information on how structures would respond to a scenario when there are consequent events after an earthquake, such as a tsunami or fire. Although including consequent events may complicate the problem to solve in order to achieve a reliable and realistic assessment, structures need to be evaluated, designed and protected considering all practical loads and consequences.

14.4 Fire performance of a reinforced concrete building following an earthquake

This is a scenario when a fire started in a building due to, for example, movement of a fire source, such as a stove or furnace induced by a tremor. In such a scenario, typically the building may suffer initial damage from the earthquake and then undergo further damage as the result of exposure to the fire, following the earthquake. Depending on the amount of available fuel, such a fire could last anywhere from a few minutes to several hours. Usually, the number of available first responders or fire-fighters is not as many as required because of the need to respond to a large number of incidents at the same time, as is normally the case after a natural disaster. Therefore, full burn-out fires would be expected in many of these post-earthquake fires. In other words, structures engulfed in a fire following an earthquake would experience considerable fire damage, for a long duration, depending on the fire loads present in the structures.

This section explores a case study to provide a better understanding of how structures would perform in such a full burning scenario, following an earthquake. The case study investigates the vulnerability of a six-storey building to a fire following an earthquake using a numerical analysis tool. The building was in fact tested in January 2006 under a seismic load representing the 1995 Kobe Earthquake ground motion, using the large-scale shaking table facility in Miki (or Hyogo), Japan. Since detailed information

and experimental results of the test were available, this is a suitable example to be investigated for the scenario of fire after an earthquake. For this purpose, it was assumed that the building structure has undergone a fire following the tested earthquake. Performance of the building in the hypothetical fire was simulated by carrying out a numerical analysis. The six-storey building was made of reinforced concrete. Figure 14.2 shows the building on the shaking table after the earthquake test.

Damage to the reinforced concrete structures due to an earthquake may include concrete spalling, shear/flexure cracks, or failure of the structural elements. If the structural elements are protected by a means of fire protection, e.g. insulations, there is also a possibility of damage to the protection systems due to large deformation as the result of the seismic vibration. The induced damage on the concrete elements could change the heat transfer mechanism and temperature distributions, and degrade the load-bearing capacity of the structural elements. This is because heat could penetrate faster to the core of the concrete elements through the cracks and the concrete loses its strength at high temperatures. Therefore, in the structural response, assessment effects of the damage need to be included in both heat transfer and structural analysis. In this case study, observations of the concrete cracks and damage, recorded during the shaking table tests, were closely studied to determine potential heat/flame penetration through the damaged elements. An analytical model was then utilized to determine the



14.2 The reinforced concrete wall-frame building specimen.

change in the concrete strength capacity due to the induced seismic deformation.

Fire load after an earthquake will be dependent on the amount of combustible materials in the buildings, e.g. furniture and plastic produces, at the time of the earthquake. A design fire could be developed based on the available fire load and ventilations. However, considering a worst case scenario and for simplicity, a standard fire was considered in this study for fire resistance evaluation of the building. A standard fire curve is a time-temperature curve, e.g. ASTM E119 (2007), which is used for fire resistance tests of structural elements. The results presented in the following section could not be applied for any building under a fire following an earthquake. Types of structural materials, combustible contents in the building, location of fire, building geometry and fire compartmentations are among the many other major factors that could change the outcomes.

14.4.1 Building structure specifications

The six-storey nearly full-scale reinforced concrete building has a total height of 15 meters, (Kabeyasawa *et al.* 2005). The building was previously tested on a shaking table under the 1995 Kobe ground motion at the E-Defense Miki (or Hyogo), Japan. Details of the structural system, dimensions, and material properties can be found in Kabeyasawa *et al.* (2005). Figure 14.2 shows a picture from the building specimen on the shaking table.

14.4.2 Structural performance of building in fire with and without seismic damage

To better understand the effects from the earthquake damage on the fire resistance of the structure, the building was numerically analyzed for two scenarios; (1) for a fire without any prior earthquake or damage, and (2) for a fire following an earthquake with seismic damage. A finite element modeling technique using a structural analysis program called SAFIR (Franssen, 2007) was employed to model the building specimen. First, analysis was carried out for scenario (1) to assess fire resistance of the building before having any seismic damage to the structure. It was assumed that the fire started on the first floor but was confined in the same floor and had not spread to other levels of the building. As a worst case scenario, the analysis was carried out for an eight hour fire exposure. A typical fire in such buildings may last less than this time. The eight hour fire exposure was selected to induce failure of the building in the first scenario.

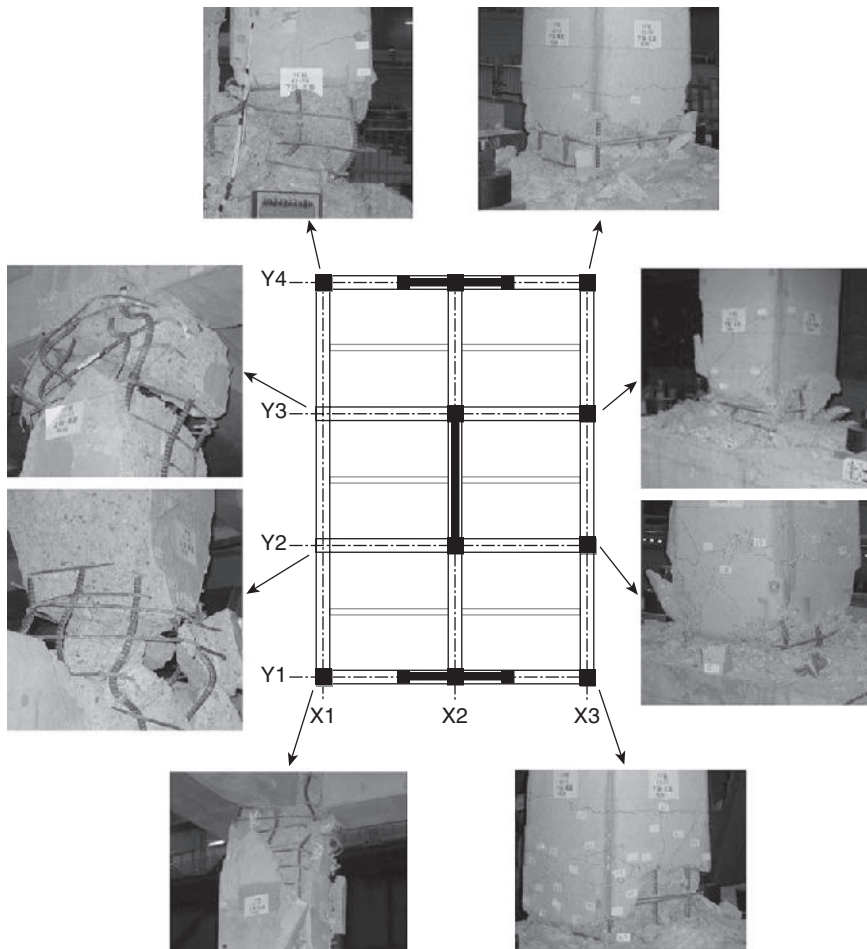
In the second scenario, fire resistance of the building was evaluated after it was damaged by the earthquake, which was the input seismic excitation

used during the shaking table test. For the sake of comparison, the same fire load and location, ASTM E-119, as that used in the first analysis, was assumed for the purpose of this assessment. An analytical method was utilized to evaluate degradation in mechanical properties of concrete due to the seismic damage and a numerical analysis was then employed to assess the effects of heat and flame penetration, into the damaged and cracked concrete elements, on the building fire resistance. Another effect to include, in a post-earthquake fire resistance evaluation, is the effects of residual lateral deformation of the structures. However, no residual lateral deformation was observed during the shaking table test of this building; therefore this was considered in this study. Note that two middle columns in Axis X1 at the first floor (see Fig. 14.3), were not included in the analysis, since both columns collapsed during the earthquake test.

14.4.3 Degradation of mechanical properties of materials

Typically, mechanical properties of materials degrade when they undergo large deformation. When a building structure is subjected to an earthquake, large lateral deformation may result in significant damage to the structural elements and degrade their material mechanical properties. In reinforced concrete structures, large flexure, shear and tensile/compression deformation may lead to the post-peak response of the material, where the materials incur permanent strength degradations. Therefore, if the deformation caused by the earthquake to the structural elements is larger than the peak-strength deformation capacity, those elements are likely to have a permanent change in their mechanical properties. For a post-earthquake fire performance assessment of structures, such permanent changes in mechanical properties due to the earthquake need to be taken into account. Although there are various models available that could estimate degradation of the material properties for structural elements of a building, there is still a challenge due to the lack of reliable analytical tools to precisely estimate such permanent damage. One of the reasons is the irregularity of the damage distributions in the elements and their cross-sections. In other words, even in a single cross-section of an element, concrete degrades differently across the section depending on the induced deformation distributions. For simplicity, in this study, an analytical approach was used to estimate average values that quantify overall damage of the elements' material properties due to the earthquake.

For the purpose of heat transfer analysis, location of cracks and crashed concrete is very important, as it affects the level of heat penetration into the elements. Since, experimental results were available, locations of cracks and damage to the concrete in different structural elements were identified and evaluated based on observations, taken place, after the shaking table



14.3 Photos taken after the shaking table test showing damage to the columns on the main floor (Mostafaei and Kabeyasawa, 2010).

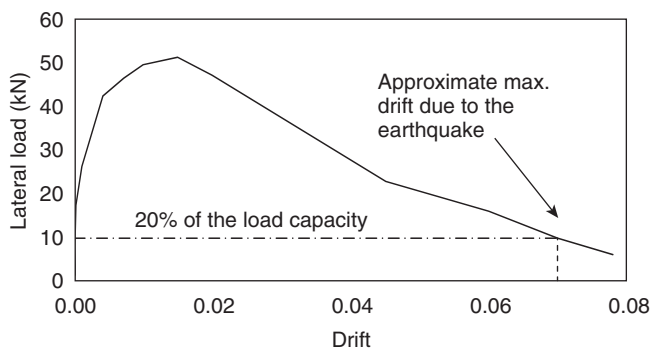
test. Only columns in the first floor suffered permanent damage and major cracks during the test. Therefore, the effects of heat penetration and degrading material properties were determined only for these elements. The rest of the elements, columns in upper floors, walls and beams were considered to have no damage. Figure 14.3 shows prevalent damage to various first-floor columns. The pictures placed on the figure were taken right after the shaking table test.

The main mechanical property of the concrete that was mostly affected by the earthquake was its compressive strength. Concrete may experience large non-linear deformation during the earthquake resulting in partial but

permanent loss of concrete compression strength. However, the main challenge for that is to quantify the compression strength of degraded concrete for different structural elements that would experience variable deformation at different locations on the cross-section. One may suggest running a time-history analysis where first the effects of the earthquake on structural elements and material properties are determined and then the response of the damaged structure is simulated for the fire effects. However, this would require a reliable analytical/numerical tool for both seismic simulation and fire analysis. Currently, there are more reliable numerical programs available for seismic response simulation and for fire resistance calculation, independently, than that for the combined simulation of fire and earthquake effects. Hence, in this study, the independent simulations were performed.

A pushover analysis, using the axial–shear–flexure interaction (ASFI) approach developed by Mostafaei and Kabeyasawa (2007), was implemented to determine the degradation of concrete compressive strength for the seismic damaged elements. The ASFI method is a displacement-based evaluation approach, developed based on coupling the traditional section analysis method, which is an axial–flexure model, with an axial–shear model, e.g. modified compression field theory (MCFT) (Vecchio and Collins 1986).

To estimate the degradation in the concrete strength, first for each structural element, a pushover analysis was performed, by the ASFI method, to determine lateral load degradation of the element at the maximum lateral deformation, experienced by the column during an earthquake. Then, the same degradation ratio was assumed as an approximate degradation ratio for the concrete compression strength of the column. Figure 14.4 shows an example of the analysis for a corner column, which indicates an 80% loss of lateral load capacity or 20% residual capacity giving an 80% degradation



14.4 Lateral load–drift response of the two columns, X1–Y1 and X1–Y4, in the ground floor, obtained by the ASFI method, and the residual load capacity.

in the concrete strength. This simulation was performed for each of the columns in the first floor with recorded seismic damage.

Estimating degradation of concrete strength based on the loss of lateral load capacity might not be considered an accurate estimation method since distribution of compression stress, and therefore the concrete strength degradation, on the cross-section of the column, under both axial and lateral load, is not uniform. Furthermore, for columns that fail in flexure, yielding the main steel bars would govern the column response and therefore concrete would degrade less than that of the lateral load capacity. However, this method may be considered as a conservative method, based on a worst case scenario, and for the sake of safety its application would be reasonable. Further research is needed to develop a more reasonable and accurate estimation methodology.

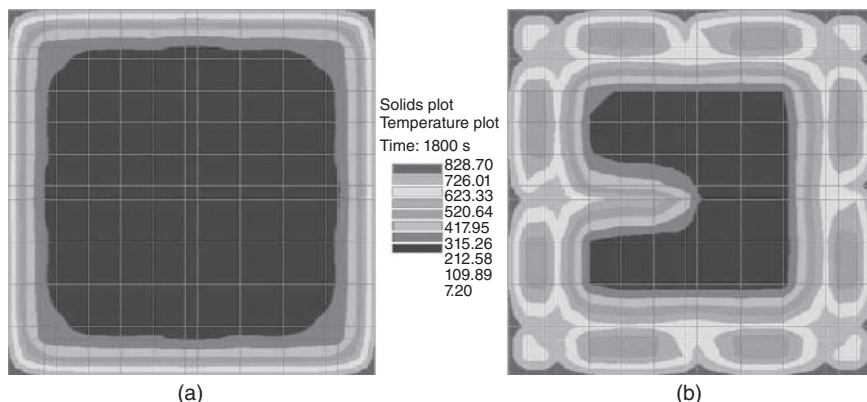
The fire resistance analysis of the whole building was then carried out when the above compressive strength degradation was included for the structural elements that were damaged during the earthquake. It is noted that in this study, experimental results were available for the seismic response of the building and its structural elements. In general, when such data are not accessible, a time history nonlinear analysis of the entire building subjected to the earthquake input motion can be performed to estimate the building seismic response.

14.4.4 Heat penetration

Openings, such as crack and spalling, could accelerate heat penetration into the structural elements and, therefore, increase the heat transfer through the elements. For instance in the case of concrete or masonry structures such cracks or spalling could occur as the result of large deformation due to an earthquake. When the cracks are large enough, these openings would let the heat and flames penetrate deep into the structural elements and elevate the temperatures rapidly in the elements. This will in turn result in degradation of material properties and reduction of building load-bearing capacity. Figure 14.5 shows the temperature distribution of the damaged and the undamaged cross-sections due to fire, for a column in the first floor of the building. The results indicate considerable effects of concrete damage due to the temperature distribution.

14.4.5 Residual drift and $P-\Delta$ effects

After an earthquake, a building could experience a large deformation which could lead to a nonlinear stage that could result in some deformation off-set or residual lateral deformation. In other words, the building could not recover its original position after the ground shaking. Such residual defor-



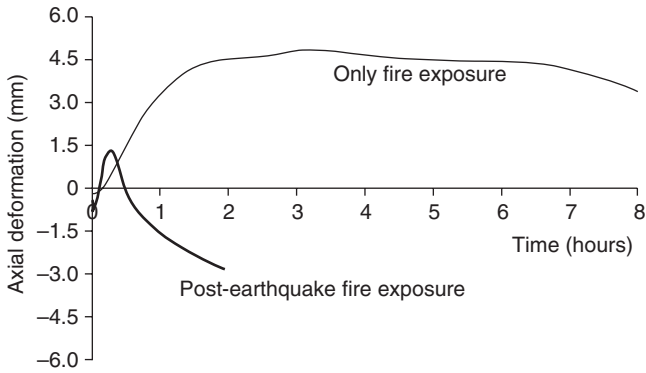
14.5 Temperature distributions on a cross-section of a column 30 minutes after the fire started: (a) section with no seismic damage and (b) section with seismic damage.

mation will cause eccentric loading on the structural columns (i.e. $P-\Delta$ effects). Typically, performance of the building would degrade, when $P-\Delta$ effects are present. In this study, since the residual lateral displacement of the building, measured during the test, was nearly zero, $P-\Delta$ effects are not included in the analysis. For a worst case scenario, this building would result in less fire resistance after the earthquake, should the structure have experienced such a residual drift. More detailed information on the results of the analysis of the six-storey building in fire following the earthquake can be found in Mostafaei and Kabeyasawa (2010).

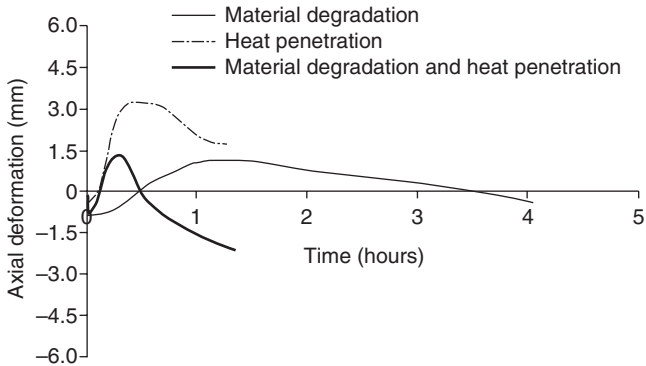
14.4.6 Main analysis results and observations

Figures 14.6 and 14.7 show the results of the fire resistance response for the short corner columns in the first floor, Axis X1, of the building before and after they were damaged by the earthquake. Based on the results of this case study, the following remarks can be made:

- The fire resistance of the six-storey building after seismic damage was considerably less than that before the earthquake.
- Factors contributing to determining the fire resistance of the building after the earthquake were mechanical properties, degradation of concrete, and the extent of crack and spalling of the concrete.
- Relatively high fire resistance was available for the building when only effects of concrete cracks and spalling in the heat transfer analysis were included in comparison with the case when only degradation of material



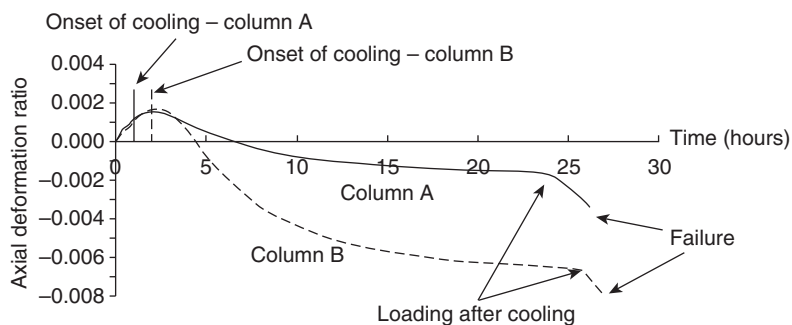
14.6 Axial deformation capacity response of a short column in the ground floor.



14.7 Axial load capacity response of a short column in the ground floor subjected to post-earthquake fire showing effects of material degradation/heat penetration on the response.

mechanical property was considered. In other words, preventing concrete from spalling or limiting the spread of cracks results in an effective post-earthquake fire protection for such buildings.

- Due to partial/complete failure of some of the structural elements, loads were redistributed to the adjacent elements. For instance in this case, the loss of the two middle columns in Axis X1 caused increase of axial load for the corner columns in the same axis. Therefore, a higher applied load may be imposed on certain structural elements during a post-earthquake fire. The increased load would in turn reduce the fire resistance of these elements.



14.8 Time history of the axial deformation ratio response for Column A and Column B (test data).

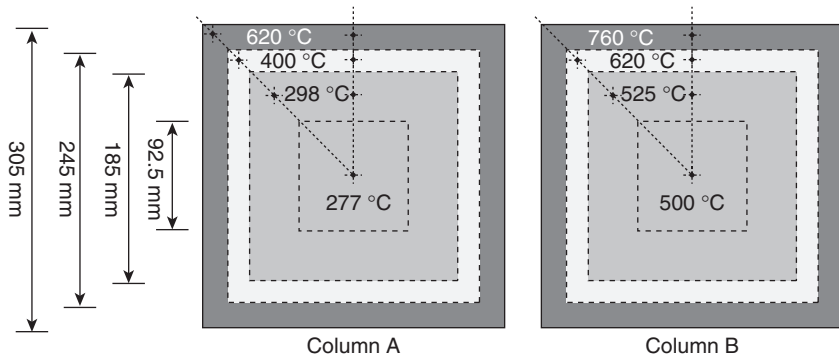
14.5 Residual seismic resistance of fire-damaged building columns

This case study explores the second scenario in which residual seismic load capacity of fire-damaged structural elements is investigated. For the purpose of this case study, two reinforced concrete columns, Column A and Column B, made with siliceous aggregate, previously tested at the National Research Council Canada (Lie *et al.*, 1986), were selected. The clear height of both column specimens was 3760 mm. Column A was exposed to a standard fire ASTM E-119 for one hour and Column B was exposed to the same fire for two hours. A constant axial load was applied during the test. After terminating the fire, the two columns were kept intact until they reached ambient temperature. Then, they were then loaded until they reached their axial load failure. Figure 14.8 shows axial deformation response of the two columns during the fire test, the cooling phase, and the final loading.

A numerical analysis was carried out using VecTor3 (2008), a nonlinear finite element analysis program for three-dimensional reinforced concrete solid structures subjected to quasi-static load conditions developed at the University of Toronto. The analysis was first done for these two column specimens for the purpose of model validation under the axial load. Then, the same numerical analysis was carried out using the software to estimate the residual seismic capacity of the two columns.

14.5.1 Axial load response analysis (verification)

Maximum temperatures reached at various depths, in the cross-section of the two columns, during the heating and cooling period were obtained and averaged from the test results (Lie and Woollerton 1988) as illustrated in Fig. 14.9. Then, the numerical simulation was carried out to evaluate the



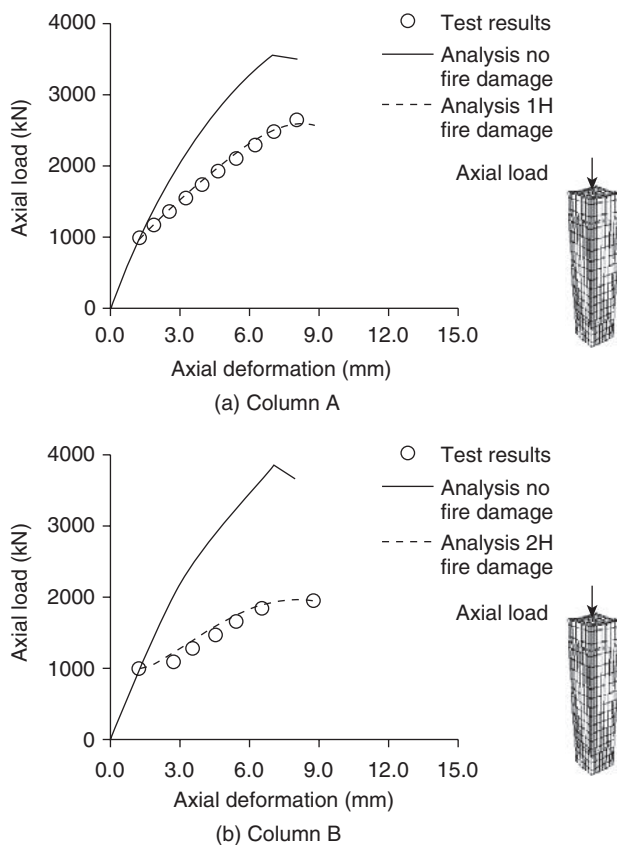
14.9 Temperature distributions assumed for the analysis, obtained from test data by averaging maximum temperatures that each part of the sections reached during the fire exposure.

axial response of the two columns under axial loads. The analysis was done considering degradation of the concrete material properties due to its exposure to the elevated temperatures.

Load–deformation results from the numerical analysis were then compared with the load–deformation ratio measured during the test. Figure 14.10 illustrates the comparison between the test and the numerical results for the two columns. The figure shows a reasonable agreement between the results, indicating a successful verification of the numerical analysis employed for the axial load response estimation. Furthermore, for comparison, the axial response of the two columns with no fire exposure, using the original properties, was also estimated and provided in the same figure. The results show that residual axial capacity was reduced to 73% for Column A; and 52% for Column B.

14.5.2 Lateral load resistance

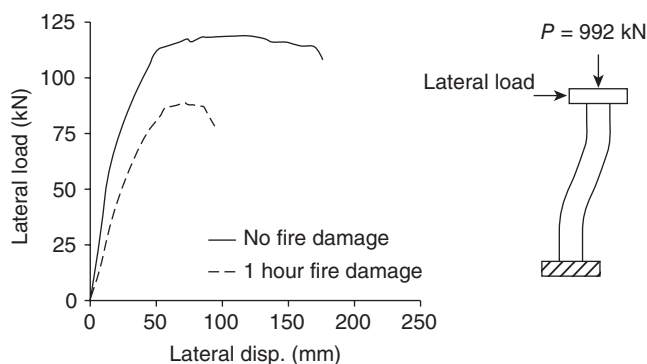
VecTor3 was utilized to investigate the residual lateral load response of the two column specimens after the fire damage. One may be concerned that validation of the numerical model for axial behavior would not apply for lateral response simulation. This is true since lateral response of the columns involves shear and flexure mechanisms in addition to the axial mechanism. It is noted that VecTor3 has been validated for such columns under lateral load in ambient temperature. Since the two column specimens in this case study are relatively long, their governing behavior would be axial and flexure when subjected to the lateral loads. In the analysis, each column section is divided into small fiber elements, hence the main impact on these elements will be either axial compression or tension response. Furthermore,



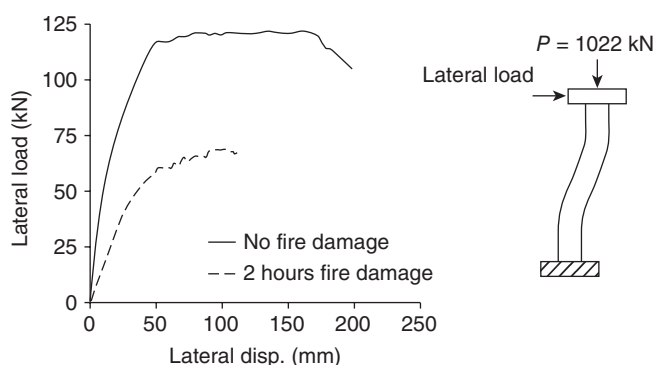
14.10 Axial load and deformation ratio relations: numerical and test results.

the VecTor3 model was validated for axial response of the two columns in the previous section. Therefore, it is anticipated that VecTor3 would produce relatively reasonable results for the two columns under the lateral loads.

For the analysis, half of the column was modeled based on the symmetric conditions, e.g. loads and supports. Lateral load was applied horizontally at the top of the columns, in a deformation controlled mode, and incremented from zero up to the column failure deformation. Figure 14.11 shows numerical results for the columns before and after fire exposure. The results indicate that the residual lateral load capacity is reduced to 73% and 58% of the original lateral load capacity for Column A and Column B, respectively. With respect to the column ductility, the ultimate lateral deformation is reduced to 48% and 61% of its original value with no fire damage for Column A and Column B, respectively. Therefore, both ductility and lateral/



(a) Column A: 1 hour fire damage



(b) Column B: 2 hours fire exposure

14.11 Lateral load and lateral deformation ratio response analysis.

seismic load capacity of the columns were substantially reduced as a result of the fire exposure.

14.5.3 Strength recovery

The loss of concrete strength due to fire exposure is largely recoverable in the long term using a proper curing method (Weigier and Fisher 1968; Poon *et al.* 2001). For instance, concrete that is exposed to an elevated temperature of 500°C , recovers 90% of its original strength in about a year (Lie, 1992). Therefore, if time permits, by employing a proper inspection and assessment approach after a fire, and applying an efficient curing/retrofitting method, concrete structures may almost regain their minimum axial and seismic resistance capacities that were required for the original seismic design. Further studies would be required to develop more efficient

inspection techniques and proper curing/retrofitting methodologies for structures damaged after a fire.

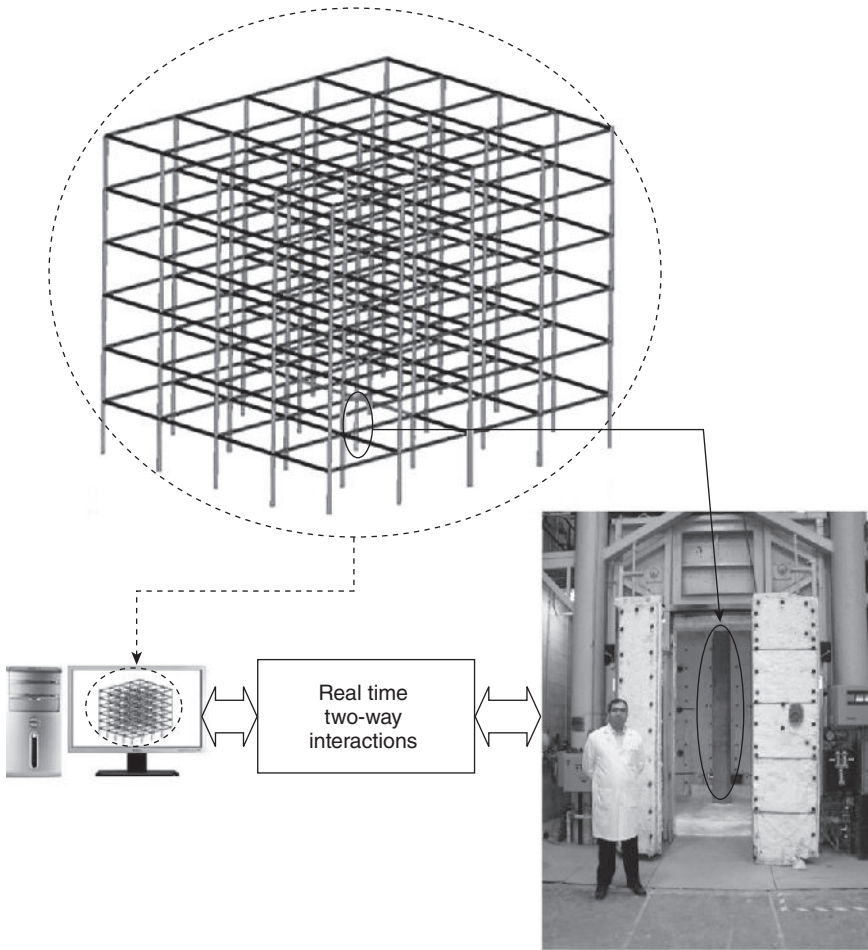
14.6 Lateral load resistance of a fire-damaged column using a hybrid method

Recently, a hybrid test procedure has been developed and implemented at the National Research Council Canada, for evaluating lateral load resistance of reinforced concrete columns after experiencing fire damage. Through this study, a column was tested as part of the six-storey concrete building using a hybrid testing technique. The test included a fire on the first floor of a six-storey building. Figure 14.12 shows the set-up of the hybrid testing technique for the fire test. In the hybrid method, the whole building is divided into two substructures; the test specimen and the model component. Then the fire performance of the whole structure is evaluated based on coupling the performance of the two substructures and by including their interactions in real time during the simulation. In this study, the hybrid method was performed for estimating fire resistance of the building. Six days later, after the entire structure had cooled to the ambient temperature, the building was subjected to lateral loads, determined based on a design seismic load, for estimating its residual lateral load capacity. The column's residual lateral load–deformation results obtained from this hybrid test were then compared with the results previously obtained for a column but with no fire damage. Considerable reductions were reported in both stiffness and deformation capacity of the column and the building due to the fire damage. More information on the hybrid test can be found in Mostafaei *et al.* (2011). Figure 14.13 shows an example of a lateral deformation response of the building after fire damage.

14.7 Conclusions and future trends

The overall results of three case studies on the vulnerability of reinforced concrete structures in fire and earthquake were provided in this chapter. The two main scenarios investigated included the effects of a fire following an earthquake on performance of the structures and the residual seismic capacity of the fire-damaged structures. The results from these cases studies highlighted that:

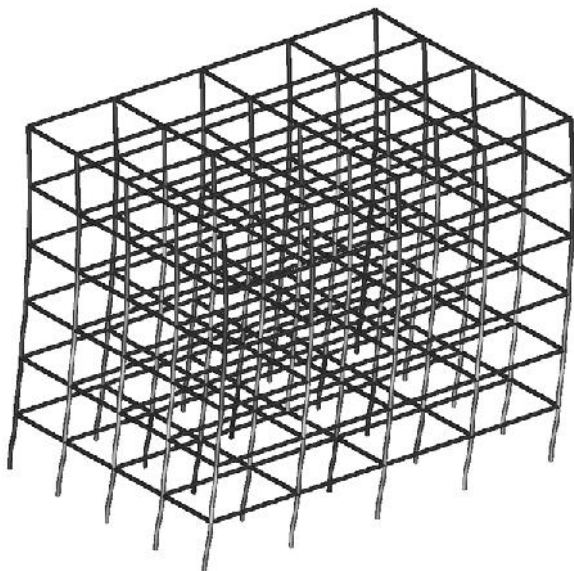
- seismic damage to structural elements and the fire protection systems would considerably reduce the fire resistance of the structure after an earthquake;
- fire damage to structures could reduce lateral load and deformation capacity of structures; and



14.12 Hybrid fire testing (HFT) method employed for a six-storey building, which involved two-way interactions between the computer simulation, on the left, and the physical test, on the right (Mostafaei *et al.* 2011).

- enhancing seismic resistance of fire protection systems and structural elements would enhance fire resistance of the structures in the event of a fire following an earthquake.

There is still a need for research and studies on the assessment, design and protection of structures in fire and earthquake. For a fire following an earthquake, new methods should be studied to reduce damage to fire protection systems of buildings, e.g. sprinklers systems and structural elements



14.13 An example of displacement response of a six-storey building after fire exposure, the darker the colour the larger the deformation.

of fire compartments. More efficient methods need to be developed to assess the fire damage of the building structure, and for the evaluation of their residual seismic resistance. Solutions are also required to conduct seismic retrofitting of fire-damaged buildings.

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Seismic risk models for aging and deteriorating buildings and civil infrastructure

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Abstract: The design, operation and maintenance of structures and infrastructure systems depend highly on their performance throughout their service life. In adverse environmental conditions structures deteriorate with time as a result of, for instance, corrosion and fatigue. Furthermore, in seismic regions structures are subjected to earthquakes of various intensities causing damage to accumulate further during their service life. Because the structural conditions change over time, modeling the deterioration process is central for reliability assessment and cost-based optimization. This chapter presents an overview of structural deterioration models (graceful and shock-based) and provides a discussion about the associated uncertainties. Furthermore, the chapter discusses the case of structures abandoned after failure and reconstructed successively after failure. The results highlight the importance of selecting the appropriate deteriorating model for the problem at hand. The importance of including deterioration within the lifetime assessment of structures is illustrated with several examples.

Key words: structural deterioration models, shock-based models, progressive deterioration, graceful deterioration models, stochastic models, life-cycle performance, reliability assessment.

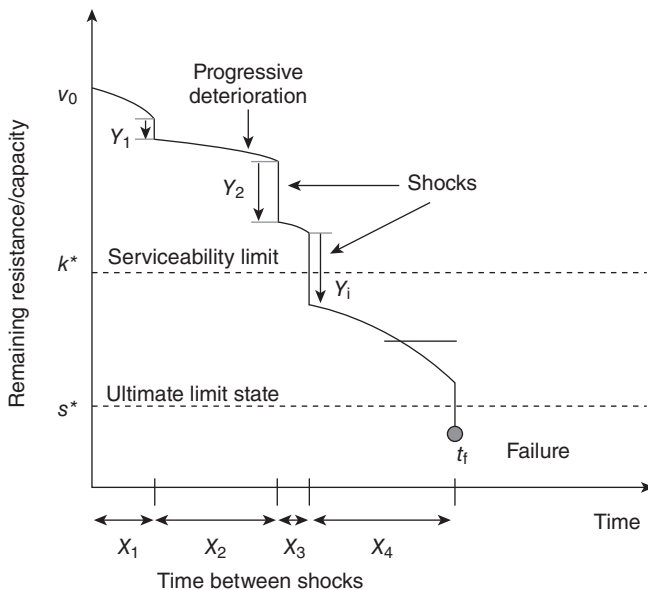
15.1 Introduction

Studying and modeling degradation of structures and critical infrastructure systems are essential for life-cycle analysis, reliability estimations and defining optimum maintenance and operation strategies. This is particularly important for structures located in aggressive environments (high/low temperature, humidity, etc.) or subjected to frequent extreme events (e.g., earthquakes). In practice, structural design does not usually take into account changes in the structural condition (e.g., deterioration) over time, underestimating the reliability and the operation costs. In order to understand damage accumulation mechanisms it is necessary to take into account the time-dependence of the structural properties as a result of the dynamic interaction with the external (e.g., environmental) demands. The main challenges in modeling deterioration are to manage the damage accumulation

mechanisms and the associated uncertainties, which are essential for effective decision making.

Structural deterioration is a combination of both progressive (e.g., corrosion, fatigue) and sudden (e.g., earthquakes) damaging events (Fig. 15.1). Progressive deterioration results from adverse environmental conditions such as high temperature, humidity and carbonation, chloride ingress, bio-deterioration, etc. The main effect of the surrounding environment on reinforced concrete (RC) structures, in particular, is the loss of effective cross-section of steel reinforcement due to corrosion, concrete cracking, loss of bond and spalling. Among all these phenomena, corrosion is probably the problem that has attracted most attention. According to Bhide (1999), about 173 000 bridges on the interstate system of the United States are structurally deficient or functionally obsolete due in part to corrosion. RC structures are susceptible to corrosion when they are placed in environments with high concentrations of chloride ions or carbon dioxide. Chloride ingress into a concrete matrix is dominated by a complex interaction between physical and chemical processes and although the details of these deterioration mechanisms are beyond the scope of the chapter, they are well described by, for instance, Vu and Stewart (2000), Duracrete (2000) and Bastidas *et al.* (2009).

On the other hand, deterioration caused by successive extreme events is mainly associated with the occurrence of earthquakes, hurricanes or blasts



15.1 Description of deterioration mechanisms.

(including both accidents and terrorists attacks). Damage accumulation after earthquakes has attracted a significant amount of interest, especially after the recent events in Chile and Japan. Frequently earthquake damage is evaluated in terms of the variation (e.g., increment) of the inter-storey drift, the loss of stiffness or the amount of energy dissipated. A large number of papers and reports have been produced regarding damage accumulation in structures in a single seismic shock due to repeated cyclic loadings (e.g., Jeong and Iwan, 1988; Ballio and Castiglioni, 1994). However, not very many papers are available discussing the problem of damage accumulation over time after the structure is subjected to successive earthquakes. Earthquake damage accumulation is in general difficult to assess and quantify. In other fields of engineering, and from a more mathematical perspective, extensive research has been carried out on models for shock degradation; for more details see Barlow and Proschan (1965), Aven and Jansen (1999), Nakagawa (1976), Feldman (1977) and Zuckerman (1977). These models have been shown to have features which can be used for better understanding of structural damage accumulation over the structure's lifetime.

The objective of this chapter is to present an overview of existing stochastic models to manage structural deterioration as a result of the individual or combined action of both progressive and extreme event degradation. The chapter emphasizes the importance of including damage accumulation when studying the life-cycle performance of structures and infrastructure systems. This chapter has been prepared as assemble of previous works published by the authors on topics related to stochastic modeling of damage accumulation. The reader can find some additional details in the following reference documents: Sánchez-Silva and Rackwitz (2004), Bastidas *et al.* (2010, 2011), Sánchez-Silva *et al.* (2011), Gosh *et al.* (2012) and Sánchez-Silva and Riascos-Ochoa (2012a,b). The chapter is structured as follows: in Section 15.2 we present an overall description of the deterioration process and the main characteristics of both progressive and shock-based mechanisms. In Sections 15.3 we focus primarily on shock-based damage processes while in Section 15.4 there is a discussion on progressive degradation models. Finally in Section 15.5 the combined effect of both progressive and shock-based mechanisms is discussed.

15.2 Structural degradation

15.2.1 Definition of degradation

Structural deterioration (i.e., damage) is defined as any change to the material or the geometric properties affecting the structural performance (e.g., capacity). The deterioration of the system is commonly measured with

respect to a physical parameter; for instance, in structures it can be the stiffness or the inter-story drift. In order to be general, the parameter that will be used to describe the system performance in this chapter is the structural capacity/resistance.

15.2.2 Progressive degradation

Progressive (graceful) degradation results in capacity/resistance continuously being removed from the structure at a rate that may, or may not, change (randomly) over time. Most progressive degradation models available in the literature assume that the *form* of the degradation process is known, but the parameters are uncertain. The solution to this problem conveys to a parameter estimation problem. Thus, if $V_p(t)$ is the state of the system at a given time t , which in practice, may be expressed in terms of, for example, remaining capacity, reliability, safety and durability, then these type of models have the following general form:

$$V_p(t) = \begin{cases} v_0 & 0 \leq t \leq t_e \\ v_0 - h(\mathbf{p}, t - t_e) & t > t_e \end{cases}, \quad [15.1]$$

where v_0 is the remaining life of the system at time $t = 0$ and t_e is a time before degradation initiation (e.g., time to corrosion initiation). The function h may take a linear, non-linear, exponential or any other form based on the appropriate selection of the vector parameter $\mathbf{p}$, which depends upon the problem at hand.

Progressive degradation can be handled also by evaluating variations in the degradation rate, i.e., $\delta_p(t)$, $t \geq 0$. Thus, the system state (e.g., remaining structural capacity) at time t can be expressed as:

$$V_p(t) = v_0 - \int_0^t \delta_p(\tau) d\tau \quad [15.2]$$

where the rate may be dependent or independent of time. Finally, the degradation function can be modeled also as a discrete process with random changes at fixed/random time intervals. If the distribution or the time between these changes is selected appropriately, it is possible to find a good approximation to the continuous solution. These models are used for shock-based deterioration processes and will be discussed in Section 15.4.

15.2.3 Degradation caused by shocks

Shocks can be defined as events that cause a significant change in a system's physical property in a small time interval. Then, shock-based degradation

occurs when a fixed amount of capacity/resistance is removed from the system at discrete points in time (Sánchez-Silva *et al.*, 2011). If Y_i is a random variable describing the degradation caused by shock i , the total degradation by time t is: $D_s(t) = \sum_{i=1}^{N_t} Y_i$; where N_t represents the number of shocks by time t . Note that in many practical applications the time between shocks is also random; therefore, N_t is also a random variable. Then, the remaining structural capacity/resistance by time t can be computed as:

$$V_s(t) = v_0 - D_s(t) = v_0 - \sum_{i=1}^{N_t} Y_i \quad [15.3]$$

Extensive research has been carried out on developing mathematical models for shock degradation. Additional details about these methods can be found in Barlow and Proshan (1965), Aven and Jansen (1999), Frangopol *et al.* (2004) and Sánchez-Silva *et al.* (2011).

15.2.4 General degradation model

Frequently, progressive and shock-based deterioration occur simultaneously (Fig. 15.1). Thus, for a structural component with initial capacity, ψ_0 , subject to both continuous and sudden damaging events acting independently, the remaining capacity/resistance by time t can be computed as (Sánchez-Silva *et al.*, 2011):

$$V(t) = v_0 - \int_0^t \delta_p(u) du - \sum_{i=1}^{N_t} Y_i \quad [15.4]$$

It is important to stress that in some cases both events are not independent and, therefore, the coupled effect should be taken into consideration.

15.3 Shock-based damage accumulation models

15.3.1 Damage accumulation due to successive earthquakes

Some of the early work on seismic damage accumulation of structures as a result of successive earthquakes can be found in Elnashai *et al.* (1998). Elnashai *et al.* showed that the ductility demand imposed on a structure, following multiple earthquake ground motions, is often several times higher than the ductility demand required by a single earthquake occurrence. Additionally, studies by Murià-Vila and Jaramillo (1998) revealed a significant reduction in lateral stiffness of a building founded in soft soil under repeated low magnitude earthquake excitations. The work by Amadio *et al.* (2003) focused primarily on the behavior of inelastic single degree of

freedom system under repeated earthquake ground motion. The study identified the effect of factors such as structural period, type of earthquake ground motion and level of available ductility on damage accumulation. Seismic damage accumulation can also be critical during the aftershocks after a large earthquake. In this regard, Luco *et al.* (2004) computed the ‘aftershock fragility’ of a structure with the probabilistic residual capacity of the damaged building following a main shock occurrence. Additionally Yeo and Cornell (2009a, 2009b, 2009c) proposed a decision-making framework that minimizes the expected life-cycle cost of structures subjected to aftershocks. However, these and other works do not capture completely the conditions that define damage accumulation along the service life of the structure. More research is still required to understand damage accumulation, which has shown to be critical to make better designs and to develop cost-effective maintenance plans.

15.3.2 Overview of existing shock-based damage accumulation models

The most common shock models can be constructed from the *Poisson process* (homogeneous and non-homogeneous). In this case, the time between shocks is exponentially distributed and changes in the state of the structure (deterioration) are assumed to be deterministic and of intensity one. Thus, every time an event occurs, the system deteriorates one unit. On the other hand, a *compound Poisson process* is a stochastic process, which takes into consideration the randomness of jumps. Similar to the non-homogeneous Poisson process, the *Gamma process* has also been used for modeling deterioration. A Gamma process also has independent non-negative increments, but shocks follow a Gamma distribution. While in a Poisson process there is a finite number of jumps in a finite time interval, in a Gamma process there are infinite numbers of jumps in a finite time interval (Noortwijk, 2009). For this reason, Gamma processes are suitable for describing continuous (graceful) deterioration (see Section 15.4.3).

Shock models based on *Markov processes* are also widely used for modeling deterioration (Barlow and Proshan, 1965). Markov models assume that the structure state at any point in time is characterized by a discrete space. Markov chains focus only on the transition between states at fixed time intervals and hold the Markov property. A model has the *Markov property* if the next state of the structure depends only on its current state and not on its history. On the other hand, *semi-Markov processes* allow for the time between transitions to be random. Semi-Markov processes can be discrete or continuous depending of the distribution of the time between a structure’s state changes. A special case of semi-Markov processes are continuous-time Markov processes in which the distribution of the time between

state changes is exponential; and, therefore, the Markov property holds (Howard, 2007). The third group of models is based on the full stochastic description of the process. In these models, time intervals are not necessarily exponential and shock distributions may be also arbitrary; e.g., see Sánchez-Silva *et al.* (2011). These models are described in the following sections.

15.3.3 Hazard rate-based stochastic model

Define a probability space $(\Omega, \mathfrak{F}, P)$ with respect to which all random variables that will be used in this model are measurable. Consider that the performance of the model throughout its lifetime is defined by a unique threshold, k^* . Then, the system will be abandoned or intervened every time the remaining capacity/resistence, $V(t)$, falls below k^* , i.e., $V(t) < k^*$ (Fig. 15.1). Furthermore, in a system that is in operation, failure (reconstruction) or preventive maintenance can be modeled as a first passage problem defined in terms of the instantaneous intervention rate. This is, the probability that if the system has survived for a time t (i.e., has not being intervened), it will not survive an additional time dt ; this is also commonly referred to as the *hazard function*. The hazard function, expressed as the rate $\lambda(t)$, is a renewal counting process:

$$\lambda(t) = \sum_{n \geq 0} \frac{f(t - T_n)}{1 - \int_0^{t - T_n} f(u) du} \cdot 1_{T_n < t \leq T_{n+1}} \quad [15.5]$$

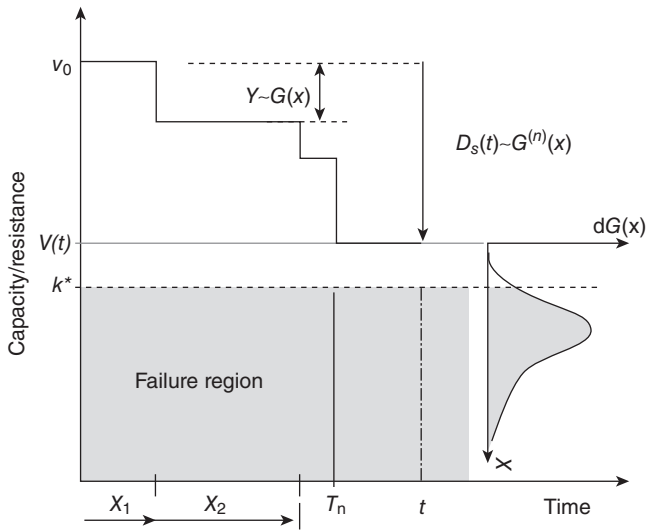
where T_n is the sum of times between shocks (Fig. 15.2). At this point, it is assumed that interventions occur only at shock times. The rate defined in equation 15.5 does not provide information about the remaining capacity/resistence of the system. Then, assuming that the magnitude of damage and the shocks' occurrence times are independent, the *instantaneous intervention rate* can be computed as (Sánchez-Silva *et al.*, 2011):

$$dP(t) = \lambda(t) P[D_s(t) > v_0 - k^*] dt \quad [15.6]$$

where $D_s(t)$ is the accumulated damage up to time t . Thus, if $dG(x)$ describes the probability of having a shock size (i.e., damage size) between x and $x + dx$, the instantaneous intervention rate after n shocks is:

$$dP(t, n) = \left[\int_{V(t, k^*, n)}^{\infty} \lambda(t) dG(x) \right] dt \quad [15.7]$$

where $V(t, k^*, n) = V(t, n) - k^*$ and $V(t, n)$ is the capacity of the system after n shocks. The complexity of the solution of equation 15.7 lies in computing the lower limit of the integral, which depends on the number of shocks n and the distribution of the sum of shock sizes (i.e., accumulated damage)



15.2 Description of the probability of failure for systems abandoned after first intervention.

(Fig. 15.2). Note that failure or serviceability assessments can be computed by exchanging s^* and k^* in equation 15.7 (see Fig. 15.1).

The distribution G in equation 15.7 describes the probability of having a certain damage level as a result of a shock (e.g., loss of structural capacity after an earthquake). This distribution is obtained after studying the dynamic interaction between the earthquake demand and the structural response. Commonly G is derived from the so-called fragility curves, which describe the probability that the system reaches a certain damage level in terms of a demand parameter. Several approaches to compute these curves have been proposed in the literature but their discussion is beyond the scope of this chapter (Hazu, 1997). Finally, it is important to stress that although the assumption of *iid* shocks might seem strong, it relaxes assumptions made by other models in which the structural damage, as a result of successive shocks, does not accumulate with time (Rackwitz, 2000).

15.3.4 Systems abandoned after failure

Instantaneous failure rate

This section considers the case of a structure exposed to successive shocks until the remaining life falls below a prescribed threshold value, i.e., s^* . Once this happens, the structure is abandoned; this is, the system is not reconstructed or intervened in any way. The instantaneous failure probabil-

ity can be computed by replacing k^* by s^* in equation 15.7. Equation 15.7 depends on the number of shocks that have occurred by time t , which is a random variable. Therefore, removing the condition on the number of shocks, the instantaneous failure rate can be rewritten as:

$$dP(t) = \sum_{n=0}^{\infty} \left[\int_{V(t, s^*, n)}^{\infty} \lambda(t) dG(x) \right] P(N = n, t) dt \quad [15.8]$$

where N is the number of shocks and $P(N = n, t)$ is the probability of having n shocks by time t . The lower limit of the integral in equation 15.8 is a function of the initial remaining life and the shock history; then,

$$V(t, s^*, n) = \int_0^{v_0 - s^*} (v_0 - s^* - x) dG^{(n)}(x) \quad [15.9]$$

where $dG^{(n)}$ is the n th Stieltjes convolution of $G(x)$ with itself. Then, the probability that an intervention is required before or at time t is given by:

$$P(\text{intervention before } t) = \Lambda(t) = \int_0^t dP(t) \quad [15.10]$$

Distribution of time to failure

Assuming independence between failures and disturbances, the distribution of the time to n th shock can be computed in terms of the time required for the remaining life to fall below s^* ; this is:

$$\begin{aligned} P[V(t) \leq s^*] &= \sum_{n=0}^{\infty} P(v_0 - s^* \leq D_s) P(n, t) \\ &= \sum_{n=0}^{\infty} \left[1 - \int_0^{v_0 - s^*} dG^{(n)}(x) \right] P(n, t) \end{aligned} \quad [15.11]$$

where $P(n, t) = P(N = n, t)$ is the probability of having n shocks by time t (Sánchez-Silva *et al.*, 2011).

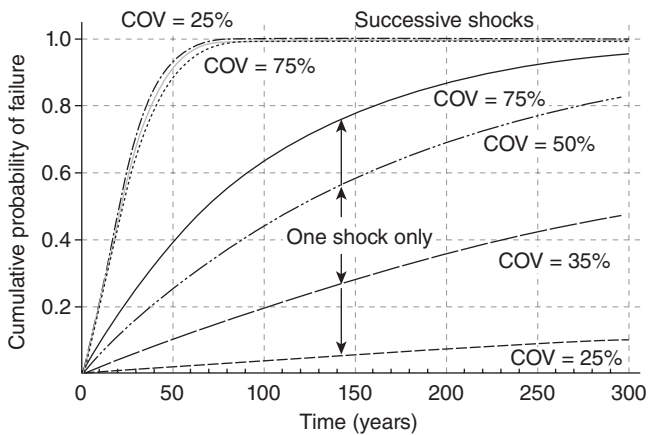
Illustrative example

Consider a structure that is put into service at time $t = 0$ with an initial capacity $v_0 = 100$ (capacity units); the minimum acceptable capacity is $k^* = 25$. The structure is located in a seismic region where earthquakes occur according to a Poisson process with rate $\lambda = 0.1$ and with *damaging intensity* that follows a lognormal distribution with $\mu = 40$. For comparison purposes, several coefficients of variation (i.e., COV) were selected: 25%, 35%, 50% and 75%.

Consider initially the case of a structure that does not deteriorate with time. In this case, the failure probability is computed as the probability that one shock causes it to fail. If the shock size is a random variable Y , with probability density dG , this failure probability is $p = 1 - G(v_0 - k^*)$. Now, if the structure is subjected to a train of earthquakes (shocks), the probability of failure in the i th shock (and not before) follows a geometric distribution, i.e., $p(i) = p(1-p)^{i-1}$ with $i = 1, 2, \dots$; and the probability of failure at the n th shock or before is: $\sum_{i=1}^n p(1-p)^{i-1}$. Conditioning out the number of shocks, the probability that the system fails at time t or before is calculated as:

$$P_f(t) = \sum_{n=1}^{\infty} \sum_{i=1}^n p(1-p)^{i-1} P(N=n, t) \quad [15.12]$$

The probability of failure as a function of time is shown in Fig. 15.3. For the particular case of $t = 50$ years, $P_f(50)$ takes the values the following values: 0.02, 0.11, 0.25, 0.40; corresponding to COV values of 25%, 35%, 50% and 75%. It is interesting to note that the probability of failure increases significantly with the COV of shock sizes (i.e., damage intensity). This is because a larger variation in shock sizes leads to a higher value of the probability of failure p . Furthermore, the COV has also an impact on the mean time to failure (MTTF). The MTTF for COV = 35%, 50% and 75% is 460, 171 and 99 years respectively. Note that for this particular case, the MTTF has an analytical solution; i.e., $MTTF = 1/(p\lambda)$. On the other hand, if damage accumulates, the probability of failure can be computed using equation 15.7. The probability of failure as function of time for this



15.3 Comparison of structural failure probability models.

case is also shown in Fig. 15.3. The results show clearly the importance of considering damage accumulation. Note that in this particular case, the probability of failure at any time increases with respect to the case of no damage accumulation. This implies a large underestimation of failure probability if damage accumulation is not considered.

In the experiments, it was observed that the effect of COV on the damage accumulation models is not significant compared with the first case. Although smaller values of COV result in larger probability of failure, differences are not significant. For example at $t = 50$, the probability of failure $P_f(t)$ is 0.93, 0.92, 0.91 and 0.88 for COVs of 25%, 35%, 50% and 75%, respectively.

For the case of damage accumulation, MTTF takes the values of 23.6, 24.3, 25.1 and 26.6 years for COV = 25%, 35%, 50% and 75%. It can be noticed that there is a substantial reduction in the structural lifetime when damage accumulation is considered. Note also that the structural lifetime increases with COV. However, in all cases considered, the differences are not as significant compared with the influence of other parameters such as the occurrence rate of the earthquakes and the parameters of the shock size distribution.

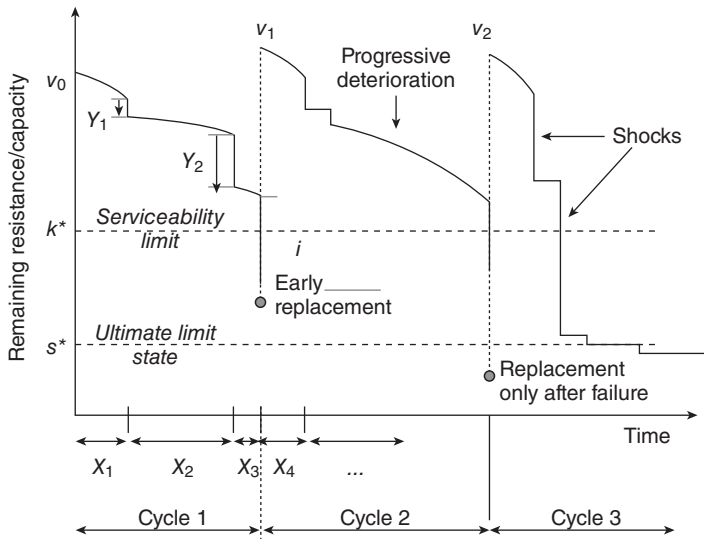
15.3.5 Successive reconstruction

General life-cycle models describe the performance (i.e., deterioration) of a system or a component throughout its lifetime. In a life-cycle model, once a structure is put in service damage starts accumulating as a result of progressive deterioration (e.g., corrosion, fatigue) or shocks (i.e., earthquakes) until the structure fails; it is then repaired or reconstructed and the process restarts. A sample path describing the performance of structural system throughout its lifetime is depicted in Fig. 15.4.

Shock-based damage accumulation

The lifetime of repairable systems consists of successive independent and stochastically identical cycles. A cycle starts with the system in a condition 'as good as new' and it deteriorates as a result of shocks; once the system fails, immediate reconstruction (or maintenance) is carried out and a new cycle starts. Let's define the counting process $\{\ell(t), t \geq 0\}$ as the number of interventions (i.e., early or full replacement) by time t . Then, a cycle is defined as the time between any two interventions; and $Z_{\ell(t)}$ denotes the $\ell(t)$ -th replacement time (initiation of a cycle). Then, the total amount of shocks in the $\ell(t)$ -th cycle can be computed as (Sánchez-Silva *et al.*, 2011):

$$Q^{\ell(t)}(t) = \begin{cases} \sum_{j=1}^{N(t)} Y_j & \ell(t) = 1 \\ \sum_{j=1}^{N(t)} Y_j - \sum_{j=1}^{N(Z_{\ell(t)-1})} Y_j & \ell(t) > 1 \end{cases} \quad [15.13]$$



15.4 Description of the degradation for regenerative process.

where the second term on the right hand side of equation 15.13, for $\ell(t) > 1$, is the history of demands since the last replacement. Note that the accumulation of damage is defined by assuming statistically identical cycles.

Then, the loss of remaining lifetime at a given cycle $\ell(t)$ is computed by subtracting the accumulated damage caused by shocks in the cycle from $v_{\ell(t)-1}$. Then,

$$V(t) = v_{\ell(t)-1} - Q^{\ell(t)}; \quad \ell(t) = 1, 2, \dots \quad [15.14]$$

In the $\ell(t)$ -th cycle, an intervention takes place once the remaining capacity/resistance falls below k^* (or s^*), i.e., $V_{\ell(t)}(t) < k^*$ (Fig. 15.3); and if the structure is not abandoned, it is put back in service immediately. In other words, intervention times are negligible and the 'off' times non-existent. If the structure is either reconstructed after failure or preventive maintenance is carried out, the instantaneous intervention rate (equation 15.7) can be rewritten as (Sánchez-Silva *et al.*, 2011):

$$\begin{aligned} dP(t) &= \lambda(t) P[Q^{\ell(t)}(t) > v_{\ell(t)-1} - k^*] dt \\ &= \left[\int_{V_{\ell(t)}(t, k^*, n)}^{\infty} \lambda(t) dG(x \cdot 1_{Z_{\ell(t)-1} < t \leq Z_{\ell(t)}}) \right] dt \end{aligned} \quad [15.15]$$

where $V_{\ell(t)}(t, k^*, n) = V_{\ell(t)}(t, n) - k^*$. Note that at the beginning of every cycle the remaining lifetime is reset to a random value $v_{\ell(t)-1}$.

Regenerative process

If the structure is reconstructed after failure or intervened as a result of preventive maintenance, the process regenerates (Fig. 15.4). Regenerative processes are only of interest in the case where damage accumulated with time until the system fails; otherwise, the problem can be managed using basic renewal theory (Rackwitz, 2000). When the regenerative nature of the system is taken into account, the solution depends on the cycle in which the system is. In order to condition out the cycle from this solution two approaches can be taken: (1) find the actual analytical distribution of the length of a cycle and the number of cycles by time t ; and (2) find a limiting solution (i.e., $t \rightarrow \infty$). In this chapter we will present only the latter approach.

Ordinary and delayed renewal systems differ in the initiation of the first cycle of the renewal. Contrary to ordinary renewals, in delayed renewal systems the origin does not coincide with the initiation of a cycle; therefore, the first renewal has a different distribution than the subsequent cycles. However, if the process has been running for a long time, the effects of the origin vanish as t goes to infinity, and the delayed and the ordinary process converge asymptotically although the transient behavior is different. Then, for most existing structures, which have been in operation for a long time, the asymptotic behavior can be written as:

$$\lim_{t \rightarrow 0} P[V(t + dt) \leq s^* | V(t) > s^*] = \frac{1}{E[L]} \left\{ \sum_{n=1}^{\infty} \left[\int_{V(t, s^*, n)}^{\infty} \lambda(t) dG(x) \right] P(n, t) \right\} \quad [15.16]$$

where $E[L]$ is the expected value of the length of a cycle. The length of one cycle is the expected time between interventions given that repair and reconstruction times are not significant with respect to the total life cycle. Based on renewal theory, it can be proved that $E[L]$, if all cycles are independent and identically distributed, is:

$$E[L | \mu_0] = E[X] \{m_G(v_0 - s^*) + 1\} \quad [15.17]$$

where the renewal function, m_G is computed as $m_G(t) = \sum_{i=1}^{\infty} G^i(t)$; for more details see Ross (1996). Note that the expected value of the cycle's length is conditioned on the remaining life at the beginning, i.e., v_0 .

15.4 Approximation to graceful deterioration

15.4.1 Progressive deterioration in RC structures

In RC structures there are several damage mechanisms that can be classified as progressive; among those, corrosion and fatigue are the most

common. Corrosion is a form of steel deterioration that consists of material disintegration as a result of chemical or electrochemical actions. Most metals corrode on contact with water (or moisture in the air), acids, bases, salts, and other solid and liquid chemicals. Metals will also corrode when exposed to gaseous materials like acid vapors, formaldehyde gas, ammonia gas, and sulfur containing gases (Jones, 1992). Depending on the case, corrosion can be concentrated locally to form a pit, or it can extend across a wide area to produce general wastage (Vu and Stewart, 2000; Martín-Pérez *et al.*, 2001; Val and Stewart, 2003, 2009). On the other hand, fatigue is the damage of a material resulting from repeated stress applications (e.g., cyclic loading). Fatigue is conditioned by many factors such as high temperature, i.e., creep-fatigue, and presence of aggressive environments, i.e., corrosion-fatigue (Schijve, 2003; Suresh, 1998). Coupled corrosion-fatigue deterioration results from the combined action of cycling stresses in corrosive environments. Localized corrosion leading to pitting may provide sites for fatigue crack initiation. Several experimental studies have shown that pitting corrosion has been responsible for the nucleation of fatigue cracks in a wide range of steels and aluminum alloys (Kondo, 1989; Ahn *et al.*, 1992; Chen and Duquette, 1992). In such studies, pits are usually found at the origin of the fracture surface.

15.4.2 Approximation to graceful deterioration

In modeling graceful (continuous) deterioration it is frequently not possible to evaluate the time-dependent nature of the degradation rate (Kiessler *et al.*, 2002) (equation 15.3). Thus, alternative models focus on evaluating progressive deterioration discretely. These models use stochastic processes in which the loss of remaining life (i.e., deterioration) occurs only at discrete, and frequently, deterministic times. Therefore, most of these models are limiting solutions of shock models. In this section two shock-based approaches to graceful deterioration are presented. In both cases, progressive deterioration is caused by random shock occurring at fixed time intervals.

15.4.3 Gamma process

If the Gamma process (Abdel-Hameed, 1975) is used for modeling graceful deterioration, it is assumed that shocks follow a Gamma distribution with identical scale parameter and occur at small, deterministic and equally spaced time intervals (Noortwijk 2009). A continuous-time stochastic process $\{X(t), t \geq 0\}$ is said to be a Gamma process if it has non-negative independent increments and $X(s) - X(t) \sim \text{Gamma} [v(s) - v(t), u]; \forall 0 \leq t < s$; where $\text{Gamma} [v(t), u]$ is the Gamma distribution function with param-

eters $v(t)$ and u . When using the Gamma process for modeling deterioration, it is assumed that the scale parameter remains constant throughout the process while the shape parameter is a right continuous, non-decreasing, real-valued function (Noortwijk, 2009).

15.4.4 Trend-dependent

In this approach, graceful deterioration is modeled by a stochastic process in which shocks are assumed to be independent and equally distributed; and the time between shocks is deterministic but not necessarily equally spaced (Sánchez-Silva *et al.*, 2011). Let's define B_i as the loss of capacity caused by a 'progressive deterioration shock' i , and assume that these shocks are independent and identically distributed with $W_B(x) = P(B \leq x)$. At a given time t and in a cycle $\ell(t)$, the accumulated damage $S(t)$ can be computed as:

$$S(t) = \sum_{j=0}^{e(t)} B_j \cdot \mathbf{1}_{\{Z_{\ell(t)} \leq t < Z_{\ell(t)+1}\}}, \quad i = 2, 3, \dots, \quad [15.18]$$

where $e(t)$ is the number of fictitious shocks, simulating progressive deterioration, that have occurred by time $t - Z_{\ell(t)}$. Note that $e(t)$ is not a random variable but a known value; i.e., $e(t) \in \mathbb{N}$. Then, the structural capacity in cycle $\ell(t)$ by time t becomes:

$$V(t, k^*) = v_{\ell(t)-1} - k^* - S(t) \quad [15.19]$$

which results by subtracting, from the structural capacity available (i.e., $v_{\ell(t)-1} - k^*$), the deterioration caused by progressive deterioration in the form of small shocks occurring at fixed times, i.e., $S(t)$.

Because shocks that simulate progressive deterioration are evaluated at fixed times, it is possible to simulate possible trends of the process (e.g., linear or exponential decay). Then, if the occurrence times are governed by any deterministic function $g(t)$, equation 15.19 becomes

$$V(t, k^*) = u_{\ell(t)-1} - k^* - S_g(t) \quad [15.20]$$

where $S_g(t)$ is the total damage caused by progressive deterioration as a result of jumps that will occur by time t , according to function $g(t)$. The number of jumps $e_g(t)$ is computed based on the selection of the time between shocks, i.e., Δt , which, in turn, is computed by solving:

$$g(t + \Delta t) - g(t) = c; \quad t > 0 \quad [15.21]$$

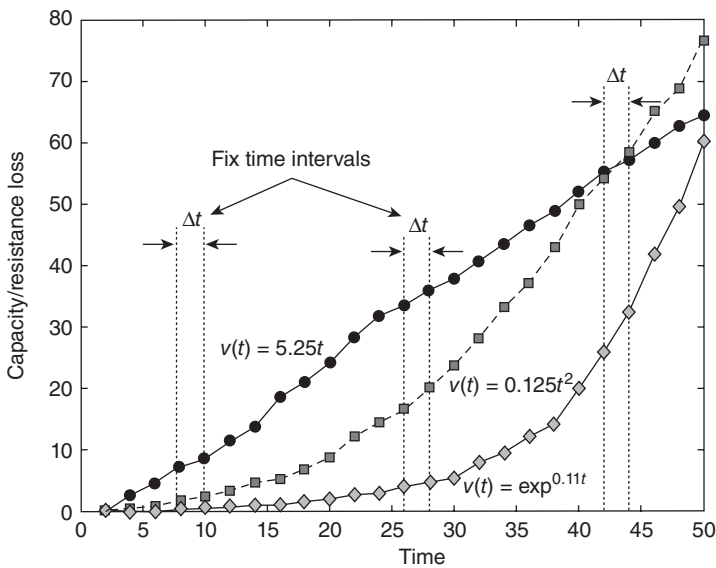
where c is a constant. For example, for $g(t) = t$, jumps are equally spaced, while for $g(t) = \exp(\alpha t)$, $\Delta(t)$ gets smaller as time increases. In this approach, shocks are assumed to be *iid* and, in principle, the shock distribution can

be selected arbitrarily. However, it is suggested that $E[B_i] = (v_0 - k^*)/e_{g(T)}$; where $e_{g(T)}$ is the number of shocks by time T (i.e., time-window for the study) according to the trend function $g(t)$.

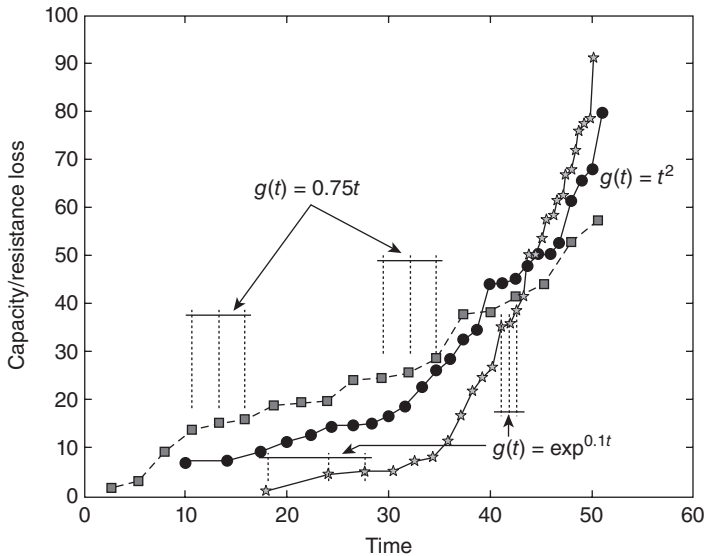
15.4.5 Comparison of graceful deterioration models

Consider a structure that deteriorates continuously with time; for example a concrete sewage pipe subjected to bacterial attack. The deterioration process can be approximated by a discrete shock-based approach according to the models presented in previous sections. Consider that the pipe is put into service at time $t = 0$ with an initial capacity $v_0 = 100$ (capacity units); the minimum acceptable capacity is $k^* = 25$. Three models for graceful deterioration that follow a Gamma process are presented in Fig. 15.5. The cases shown were constructed for a scale parameter and the following time-dependent shape parameter functions: $v(t) = 5.25t$, $v(t) = 0.125t^2$ and $v(t) = \exp(0.11t)$.

Consider now the case in which shocks are *iid* and occur at fixed time according to a function g . In Fig. 15.6 three sample paths of the deterioration history process are shown. Every process corresponds to a different *trend* function g . It can be clearly observed that the times at which the random increments are evaluated depend on the trend function. Thus, they



15.5 Gamma model for graceful deterioration (adapted from Sánchez-Silva and Riascos-Ochoa, 2012a,b).



15.6 Trend-dependent model for graceful deterioration (adapted from Sánchez-Silva and Riascos-Ochoa, 2012a,b).

are smaller as time increases for the exponential and quadratic functions. In Fig. 15.6 shocks were assumed to be exponentially distributed with mean $1/\lambda = (v_0 - k^*)/e_{g(T=50)}$.

Clearly both models depend on the selected parameters; i.e., shape function $v(t)$ (Gamma process) and trend function $g(t)$ (trend-based approach). In general, sensitivity analysis shows that the latter approach is less smooth as time increases for non-linear functions since there are more jumps in a smaller timespan. The only way to validate these models is by using actual data, which is usually difficult to find. However, if the trend of the deterioration process is somehow known, these two models can be used to obtain estimates of the MTTF.

15.5 Combined progressive and shock-based deterioration

15.5.1 Basic formulation

There is little evidence of a strong correlation between progressive deterioration and damage as a result of shocks (e.g., earthquakes), therefore, independence can be assumed in most cases. If progressive degradation has a continuous positive rate, $\delta(t) > 0$, the remaining life of the structure at time t can be computed as:

$$V_{\ell(t)}(t) = v_{\ell(t)-1} - Q^{\ell(t)} - \int_{Z_{\ell(t)-1}}^t \delta(\tau) d\tau \quad [15.22]$$

Progressive deterioration increases the probability of failure by reducing the shock size required to exceed the failure or intervention threshold value. The lower limit of the integral in equation 15.17 changes to:

$$V(t, n, s^*) = \int_0^{v_0 - s^* - g_d(t)} [v_0 - s^* - g_d(t) - x] dG^{(n)}(x) \quad [15.23]$$

Typical forms of $g_d(t)$ are linear, quadratic or exponential (Mori and Ellingwood, 1994; Pandey, 1998). More elaborate models have been proposed to include the randomness of progressive deterioration (Streicher *et al.*, 2008; Klutke and Yang, 2002). In general these can be classified into two broad categories, the random variable (RV) model and the stochastic process model. In the RV model, the probabilistic nature is included as random parameters of the deterministic function of deterioration $g_d(t)$. In the other hand, the stochastic process models, that include Markov chains, continuous-time Markov chains or the Gamma process model, incorporates the temporal uncertainty of the deterioration process and do not have a specific sample path associated (Pandey *et al.*, 2009).

15.5.2 Illustrative example

Consider a structure that is put into service at time $t = 0$ with an initial capacity $v_0 = 100$ and minimum acceptable capacity of $k^* = 0$. It will be assumed that structural deterioration results from the combined action of shocks and progressive degradation. Shocks occur according to a Poisson process with $\lambda = 0.1$ and with *damaging intensity* that follows a lognormal distribution with $\mu = 10$ and $\sigma = 5$. If damage accumulates due to shocks only, the structure's MTTF = 106 years with standard deviation of 36 years.

On the other hand, three deterministic models were used to represent graceful deterioration: $g_{p1}(t) = 2t$, $g_{p2}(t) = 0.04t^2$ and $g_{p3}(t) = 0.0008t^3$. All three models have in common that their time to failure is 50 years. In addition to these three models, a RV linear deterioration model $g_d(t) = Mt$ was also considered in the analysis. In this model M is a random variable that reflects the inherent variability of the process. M was assumed to be gamma distributed with shape and scale parameters: η and δ respectively. These parameters were chosen so that MTTF = 50 years; furthermore, three COV (0.3, 0.6 and 0.9) of the time to failure were also considered (Pandey *et al.*, 2009) to evaluate the sensitivity of the results to the variability in M . The parameters of the Gamma distribution for the three cases mentioned above are shown in Table 15.1.

Table 15.1 RV model parameters

	RV 1	RV 2	RV 3
Parameters	$\text{COV}_{\text{TF}} = 0.3$	$\text{COV}_{\text{TF}} = 0.6$	$\text{COV}_{\text{TF}} = 0.9$
Shape (η)	13.1111	4.7778	3.2346
Scale (δ)	0.1651	0.5294	0.8950

Table 15.2 Results for the combined damage: shocks + progressive deterioration

Case	Model	μ_{TF} (years)	σ_{TF} (years)	COV_{TF}
1	Shocks only	106.0	36.0	0.34
2	g_{p1}, g_{p2}, g_{p3} (*)	50.0		
3	Shocks + g_{p3}	42.1	4.2	0.1
4	Shocks + g_{p2}	38.2	12.1	0.32
5	Shocks + g_{p1}	33.9	6.7	0.20
6	RV 1 + Shocks	32.4	7.6	0.24
7	RV 2 + Shocks	29.9	9.6	0.30
8	RV 3 + Shocks	27.9	9.6	0.34

* Deterministic cases.

The combined effect of shocks and the various graceful deterioration models was compared by evaluating the mean, standard deviation and COV of the time to failure; the results are summarized in Table 15.2. It can be first observed in Table 15.2 that the results of the models are organized according to the MTTF. The first two rows show the basic cases; i.e., shocks only and deterministic degradation only. Note also that the combined effect of shocks and the deterministic graceful deterioration models (cases 3–5) leads to larger mean time to failure than the case of shocks and RV progressive deterioration (cases 6–8). When comparing cases 3–5 it is interesting to observe that the mean time to failure exhibits the following behavior: $\text{MTTF}_5 < \text{MTTF}_4 < \text{MTTF}_3$. The main reason for this is that the deterioration rate in, for instance, g_{p3} (case 3) is rather small at the beginning of the process compared with the rate of the rate of g_{p1} (case 5). By the time it starts growing faster, the system is already close to failure. This situation may change as the size of shocks become smaller; in this case the progressive deteriorating function will dominate the process.

In cases 6–8, it is noticed that the MTTF increases as the COV of the time to failure of the progressive deterioration model (Table 15.1) decreases. This means that larger variability produces smaller MTTF; this is, the failure

occurs faster. Note also that the COV of the time to failure decreases when shocks are included. For case 8, the COV of the time to failure of graceful deterioration model RV3 was originally 0.9; however, by combining this model with the effect of shocks, it decreases to a value of 0.34. This can be explained because the presence of shocks controls the dispersion.

15.6 Conclusions

Structures and infrastructure systems are permanently exposed to environmental demands and, depending upon the region, to extreme events such as earthquakes or hurricanes also. The dynamic interaction between structures and external demands yields to a loss of capacity with time; a process defined as structural deterioration. Structural deterioration is a combination of both progressive (e.g., corrosion, fatigue) and sudden (e.g., earthquakes) damaging events. Progressive deterioration results from adverse environmental conditions such as high temperature, humidity and carbonation, chloride ingress and biodeterioration. The latter results from the successive exposure to earthquakes of various magnitudes. Modeling structural deterioration has been shown to be of great importance for decisions about maintenance and reconstruction of critical infrastructure. This chapter presented an overview of structural deterioration models and describes existing stochastic models. Also a formulation for the combined effect of graceful and shock deterioration was presented. The central argument of this chapter is that life-cycle modeling is highly dependent on the deterioration mechanism; thus, the proper assessment of the deterioration is essential for optimizing the management of infrastructure systems.

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Stochastic modeling of deterioration in buildings and civil infrastructure

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Abstract: Infrastructure systems deteriorate while in service due to routine use and exposure to extreme conditions (e.g., earthquakes and corrosive environment). Deterioration is a serious concern as it can considerably reduce the structural capacity and consequently the reliability and service life of systems. Moreover, deterioration is a highly uncertain process and is often invisible. This chapter first presents the stochastic modeling of a general deterioration process. Then, as an illustration, the chapter conducts a stochastic analysis of the structural deterioration process of a typical single-column reinforced concrete (RC) highway bridge that is subject to deterioration due to corrosion and earthquakes. In particular, the illustration considers the deterioration of the bridge column due to loss of lateral stiffness, low-cycle fatigue of longitudinal reinforcement and loss of cross-sectional area of reinforcing steel. The presented method is general and can be applied to analyze other structural systems experiencing different deterioration mechanisms.

Key words: stochastic modeling, structural deterioration, reinforced concrete. (RC) bridges, infrastructure systems, earthquakes, corrosion

16.1 Introduction

Civil infrastructure systems are critical to the functioning of a country; they are expensive to build and their disruption often leads to severe consequences. Civil infrastructure systems are often subject to extreme environmental conditions causing them to deteriorate. In order to optimize the long-term reliability and life-cycle cost of infrastructure systems, we need to develop methods to model deterioration processes accounting for the relevant uncertainties. Moreover, since in the field of civil engineering, the interest in performance-based engineering is growing, we need such models to be able to predict the long-term performance of infrastructure systems.

In the area of reliability and performance of deteriorating systems, researchers have broadly conducted two types of studies. In the first type, studies primarily computed the reliability of systems at various stages of deterioration using laboratory test data and complex numerical modeling

of the system behavior (e.g., Stewart, 2001; Melchers, 2005; Choe *et al.*, 2009; Pillai *et al.*, 2010; Simon *et al.*, 2010). However, typically such studies do not completely model the time-dependence of a stochastic deterioration process and its interaction with the process of loading. Therefore, the results might not be adequate to conduct life-cycle analysis of a deteriorating system (e.g., predicting lifespan and life-cycle costs). The second type of studies focus on stochastic modeling of deterioration processes to conduct life-cycle analysis of systems (e.g., Esary *et al.*, 1973; Ellingwood and Mori, 1993; Mori and Ellingwood, 1994; Klutke and Yang, 2002; Noortwijk *et al.*, 2005; Wortman *et al.*, 2006; Kumar *et al.*, 2009; Sánchez-Silva *et al.*, 2011; Kumar *et al.*, 2012). However, owing to the complex nature of deterioration processes, there are challenges in certain aspects of modeling such as the dependence between deterioration and loads, the interaction between different failure modes of a system and non-stationary behavior of a damage accumulation process.

This chapter uses the stochastic semi-analytical approach (SSA), which is a general stochastic formulation originally proposed by Kumar *et al.* (2012), to model the deterioration of civil engineering systems and account for the effects of deterioration on their reliability over time. We use the SSA formulation to forecast the lifespan of a reinforced concrete (RC) bridge bent subject to earthquakes and a corrosive environment. The SSA formulation is selected here because it addresses the following important issues typical of other available models:

- modeling the effect of deterioration on both capacity and demand;
- modeling the dependence between the deterioration process and demand;
- modeling the combination of shock and the gradual deterioration process;
- accounting for different failure types; and
- having accurate and time-efficient solution strategies.

The rest of the chapter has four sections. In the next section, the chapter describes the characteristics of a general deterioration process. The following section summarizes the SSA formulation. Next, the chapter describes the methodology to model deterioration in RC bridges and presents a case study consisting of an RC bridge subject to deterioration due to earthquakes and corrosion. The last section summarizes the chapter and draws some conclusions.

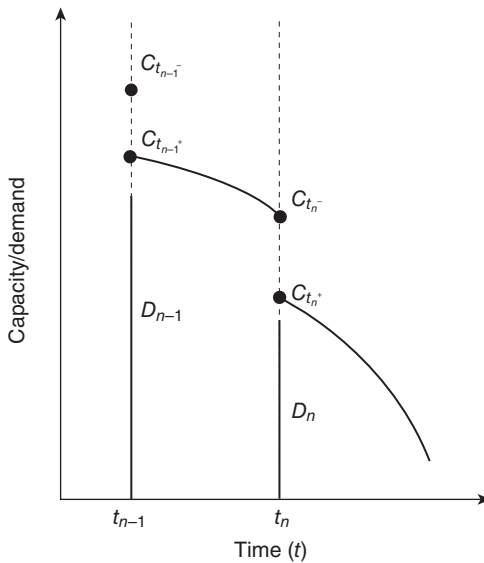
16.2 A general deterioration process

A deterioration process can be distinguished into two types: shock and gradual deterioration. A shock deterioration is an instantaneous change in a system's properties due to the action of external loads (e.g., sudden

deterioration of a bridge due to an earthquake). A gradual deterioration is the wear and decay of the system over time due to prolonged use (e.g., fatigue in machine parts during regular operation), aging, and exposure to unfavorable environment (e.g., corrosion of steel reinforcement bars in RC structures from exposure to chlorides). Figure 16.1 illustrates a general deterioration process in a system.

During the service life, a system is typically subject to a sequence of loads $\{S_{t_n}\} = \{S_{t_1}, S_{t_2}, \dots\}$ at time instances $\{t_n\} = \{t_1, t_2, \dots\}$. The sequence of loads $\{S_{t_n}\}$ induces a sequence of demands $\{D_{t_n}\} = \{D_{t_1}, D_{t_2}, \dots\}$, where $D_{t_n} = D(\mathbf{x}_{t_n^-}, S_{t_n})$, $\mathbf{x}_t$ represents the properties of the system at time t , and t_n^- is the time instant just before t_n . The capacity of the system at time t is written as $C_t = C(\mathbf{x}_t)$. Figure 16.1 shows that the system experiences shock deteriorations at t_{n-1} and t_n . The capacity C_t instantaneously changes from $C_{t_{n-1}^-}$ to $C_{t_{n-1}^+}$ and from $C_{t_n^-}$ to $C_{t_n^+}$ (t_i^+ is the time instant immediately after t_i). The figure also shows the process of gradual deterioration after the shocks at t_{n-1} and t_n in which C_t gradually decreases over time.

Typically, a system has multiple modes of failure (e.g., an RC column can fail in shear, deformation, or axial compression). For a given mode of failure k , failure can be of two types: excessive demand and excessive deterioration. In the former type, the demand is greater than the capacity corresponding to a particular performance level, i.e., $[(C_{t_n} - D_{t_n}) < 0]$. In this type, the failure occurs at the time of occurrence of the load. In the latter type, a system is considered as failed at a time t when the total deterioration W_t crosses a



16.1 A general deterioration process.

permissible value w_a . The value of w_a is such that if $W_t > w_a$ then an insignificantly small demand in the future will cause failure in mode k . The main difference between the two types of failure is that the former can take place only during the occurrence of a load whereas the latter does not require a load. It must be noted that both the failure types correspond to the same mode of failure. For example, a bridge column might fail due to excessive deformation demand during an earthquake. The column might also be considered as failed due to excessive deterioration in deformation capacity due to corrosion.

For a particular failure mode, the probability distribution of the number of shocks to failure n_F is given by the following equation:

$$P[n_F > n] = P\left[\bigcap_{i=1}^n (C_{t_i} - D_{t_i} \geq 0)\right] \quad [16.1]$$

where $P[A]$ is the probability of A and the symbol $\cap$ denotes the intersection of events. It should be noted here that the events $[(C_{t_i} - D_{t_i}) \geq 0]$ are not independent. Therefore,

$$P[n_F > n] \neq P[(C_{t_1} - D_{t_1} \geq 0)] \times P[(C_{t_2} - D_{t_2} \geq 0)] \times \dots \times P[(C_{t_n} - D_{t_n} \geq 0)] \quad [16.2]$$

The time to failure considering only failures due to excessive demand is given by t_{n_F} . Defining $N(t)$ as the number of loads or shocks in a timespan t , the distribution for t_{n_F} is given as follows:

$$P[t_{n_F} > t] = P[n_F > N(t)] \quad [16.3]$$

While in many engineering fields, $N(t)$ is commonly modeled using a Poisson process (Ang and Tang, 2007), the stochastic formulation used in this chapter is applicable for any random counting process. The distribution of time to failure t_F , considering excessive demand and excessive deterioration, is computed as follows:

$$P[t_F > t] = P[\{n_F > N(t)\} \cap W_t \leq w_a] \quad [16.4]$$

Now, for a system having k independent modes of failure, we can write,

$$P[t_F > t] = \prod_{i=1}^k P[t_{F,i} > t] \quad [16.5]$$

where, $t_{F,i}$ represents t_F for i th mode of failure only and the symbol Π denotes algebraic product. In general, for Eq. (16.4), closed-form solutions are not available and simulations are required. Often, such simulations are conducted based on the basic ‘hit and miss’ methods in which the entire

process is simulated a large number of times and the probability of failure is computed as the number of failures divided by the number of simulations. However, such methods are not suitable for computing low failure probabilities as is the case for well-designed engineering systems. Therefore, with reference to Kumar *et al.* (2012), the next section presents the SSA formulation that is computationally efficient and provides accurate results even for small failure probabilities.

16.3 Modeling of a general deterioration process using the stochastic semi-analytical approach (SSA)

The effects of deterioration on demand and capacity can be modeled by expressing the respective changes caused by deterioration using an additive form as adopted in Kumar *et al.* (2012). Alternatively, it is also possible to develop a multiplicative form by multiplying a factor to the demand or the capacity. However, it is noted that one form can be obtained from the other by using an exponential or logarithmic transformation depending on the starting model form. While the specific process of damage accumulation is modeled as a stationary process, the formulation presented here is more general and capable of modeling a non-stationary $\{D_{t_n}\}$ process. In addition, it accounts for the correlation between W_i and $\{D_{t_n}\}$.

16.3.1 Modeling the effect of deterioration on demands

In general, owing to deterioration, $\{D_{t_n}\}$ may not be a stationary process; i.e., $D_{t_1}, D_{t_2}, \dots$ are not statistically independent and identically distributed (SIID) random variables. The non-stationary nature of $\{D_{t_n}\}$ can be modeled as follows:

$$D_{t_n} = Y_{t_n} + \alpha_{t_n^-} \quad [16.6]$$

where $\{Y_{t_n}\} \in (-\infty, \infty)$ is a stationary process independent of the deterioration process and $\alpha_t = \alpha(\mathbf{x}_t)$ is a stochastic process that captures the effect of deterioration on $\{D_{t_n}\}$. The value of α_0 is arbitrary and in most cases may be assumed to be zero since at $t = 0$ deterioration has not yet occurred. Considering the case where $\mathbf{x}_{t_n} = \mathbf{x}_0$ (i.e., no deterioration), we have $D(\mathbf{x}_0, S_{t_n}) = Y_{t_n} + \alpha_0$ or $Y_{t_n} = D(\mathbf{x}_0, S_{t_n}) - \alpha_0$. Since $\{Y_{t_n}\}$ is a stationary process that is independent of the deterioration process, we always have $Y_{t_n} = D(\mathbf{x}_0, S_{t_n}) - \alpha_0$. Therefore, the cumulative distribution function (CDF) $F_Y(y)$ for Y_{t_n} is given by $P[D(\mathbf{x}_0, S_{t_n}) - \alpha_0 < y]$. Adopting an additive form, the total effect of deterioration is written as follows:

$$\alpha_{t_n^-} = \alpha_{t_{n-1}^-} + \Delta\alpha_{t_{n-1}^+}^{t_n^-} + \int_{t_{n-1}^+}^{t_n^-} \Delta\alpha^g(t) dt \quad [16.7]$$

where $\Delta\alpha_{t_{n-1}}^s = \alpha_{t_{n-1}}^+ - \alpha_{t_{n-1}}^-$ (shock deterioration), $\Delta\alpha^g(t) = (d\alpha_i/dt)|_{t \neq t_i}$. This implies that $\Delta\alpha^g(t)$ is the rate of change of α_i during gradual deterioration (or $t \neq t_i$). The value of $\Delta\alpha_{t_0}^s = 0$ because there is no shock in the time interval $[0, t_1^-]$. Also we have, $\alpha_{t_0} = \alpha_{t_1^-} - \int_0^{t_1^-} \Delta\alpha^g(t) dt = \alpha_0$. Now combining Eqs. (16.6) and (16.7), D_{t_n} is written as follows:

$$D_{t_n} = Y_{t_n} + \alpha_0 + \sum_{i=1}^{n-1} \Delta\alpha_{t_i}^s + R_\alpha(t_n^-) \quad [16.8]$$

where $R_\alpha(t) = \int_0^t \Delta\alpha^g(t) dt$.

16.3.2 Modeling the effect of deterioration on capacity

Adopting the previously used additive form, the effect of deterioration on capacity of a system is written as follows:

$$C_{t_n^-} = C_{t_{n-1}}^1 + \Delta C_{t_{n-1}}^s + \int_{t_{n-1}^+}^{t_n^-} \Delta C^g(t) dt \quad [16.9]$$

where $\Delta C_{t_{n-1}}^s = C_{t_{n-1}}^+ - C_{t_{n-1}}^-$ (shock deterioration), $\Delta C^g(t) = (dC_i/dt)|_{t \neq t_i}$. This implies that $\Delta C^g(t)$ is the rate of change of C_i during gradual deterioration (or $t \neq t_i$). As explained with respect to α_i , $\Delta C_{t_0}^s = 0$ and $C_{t_0} = C_0$. The capacity C_t is then written in terms of the total change as follows:

$$C_t = C_0 + \sum_{i=1}^{N(t^-)} \Delta C_{t_i}^s + R_C(t) \quad [16.10]$$

where $R_C(t) = \int_0^t \Delta C^g(t) dt$.

16.3.3 Formulation of failure

As discussed previously, for a given failure mode, failure can be of two types: excessive demand or excessive deterioration. The failure of the system due to excessive demand during n^{th} load occurs if $[(C_{t_n^-} - D_{t_n}) < 0]$ where $(C_{t_n^-} - D_{t_n})$ is given as follows:

$$C_{t_n^-} - D_{t_n} = (C_0 - \alpha_0) - Y_{t_n} + \sum_{i=1}^{n-1} (\Delta C_{t_i}^s - \Delta\alpha_{t_i}^s) + [R_C(t_n^-) - R_\alpha(t_n^-)] \quad [16.11]$$

Now normalizing with $u_0 = (C_0 - \alpha_0)$ and assuming that $(C_0 - \alpha_0) > 0$ (this assumption does not result in any loss of generality because α_0 is an arbitrary constant), the failure event is written as follows:

$$\hat{Y}_{t_n} + \sum_{i=1}^{n-1} \frac{(\Delta \alpha_{t_i}^s - \Delta C_{t_i}^s)}{u_0} + \frac{[R_\alpha(t_n^-) - R_C(t_n^-)]}{u_0} > 1 \quad [16.12]$$

where $\hat{Y}_{t_n} = Y_{t_n}/u_0$. Writing $(\Delta \alpha_{t_i}^s - \Delta C_{t_i}^s)/u_0 = Z_{t_i}$, $[R_\alpha(t) - R_C(t)]/u_0 = R(t)$ and defining g_{t_n} as

$$g_{t_n} = \hat{Y}_{t_n} + \sum_{i=1}^{n-1} Z_{t_i} + R(t_n^-) \quad [16.13]$$

the number of shocks until failure is given by $n_F = \min\{n : g_{t_n} > 1\}$.

Using the same definitions, the failure due to excessive deterioration occurs at time t if $W_t > w_a$ where, W_t is given as follows:

$$W_t = \sum_{i=1}^{N(t^-)} Z_{t_i} + R(t) \quad [16.14]$$

Note that $N(t^-)$ excludes the shock at t .

16.3.4 Failure time analysis

To estimate the time to failure, we need to model the processes $\{\hat{Y}_{t_n}\}$, $\{Z_{t_n}\}$ and $R(t)$. In doing so, it is important to model the correlation between $\{Z_{t_n}\}$ and $\{\hat{Y}_{t_n}\}$. The processes $\{Z_{t_n}\}$ and $\{\hat{Y}_{t_n}\}$ are correlated because they are both dependent on the loads $\{S_{t_n}\}$. The presented stochastic formulation has the flexibility to incorporate any kind of dependence between $\{Z_{t_n}\}$ and $\{\hat{Y}_{t_n}\}$ as long as the conditional cumulative distribution function (CDF) $F_{\hat{Y}|Z}(y|Z_{t_n}) = P[\hat{Y}_{t_n} \leq y | Z_{t_n}]$ can be computed and is independent of t_n . Since $\{\hat{Y}_{t_n}\}$ is a stationary process and the solution assumes $F_{\hat{Y}|Z}(y|Z_{t_n})$ is independent of t_n , it is implied that the solution is valid as long as $\{Z_{t_n}\}$ is a stationary process.

In the proposed formulation, any monotonic time-continuous random process can be used to model the process $R(t)$. The semi-analytical solution assumes that $R(t)$ is independent of $\{Z_{t_n}\}$ and $\{\hat{Y}_{t_n}\}$. However in general, the rate of gradual deterioration process can be altered during a shock (e.g., acceleration of corrosion of steel in a RC structure due to the opening of cracks during an earthquake). The semi-analytical solutions to estimate the distributions of n_F , t_{n_F} and t_F are explained next.

- *Probability distribution of n_F .* The probability that the failure has not occurred in the first n loads is given by

$$P(n_F > n) = P\left[\bigcap_{i=1}^n (W_{t_i} + \hat{Y}_{t_i} \leq 1)\right] \quad [16.15]$$

By conditioning on the values of $\{Z_{t_i}\}$, $\{t_i\}$ and $\{R(t_i)\}$ for $i \geq 1$, the failures due to each S_{t_i} can be treated as independent events. Therefore,

$$\begin{aligned}
 P[n_F > n | \{Z_{t_i}\}_{i \geq 1}, \{t_i\}_{i \geq 1}, \{R(t_i)\}_{i \geq 1}] \\
 = \prod_{i=1}^n P[\hat{Y}_{t_i} \leq 1 - W_{t_i} | \{Z_{t_i}\}_{i \geq 1}, \{t_i\}_{i \geq 1}, \{R(t_i)\}_{i \geq 1}]
 \end{aligned} \quad [16.16]$$

Now, taking the expectation of the expression in Eq. (16.16) over the distributions of $\{Z_{t_i}\}$, $\{t_i\}$ and $\{R(t_i)\}$, we obtain

$$P(n_F > n) = E \left[\prod_{i=1}^n P[\hat{Y}_{t_i} \leq 1 - W_{t_i} | \{Z_{t_i}\}_{i \geq 1}, \{t_i\}_{i \geq 1}, \{R(t_i)\}_{i \geq 1}] \right] \quad [16.17]$$

where $E[\cdot]$ is the expectation. Since $\hat{Y}_{t_i}$ does not depend on $\{t_i\}$ and $\{R(t_i)\}$, and depends only on Z_{t_i} , we have

$$P[\hat{Y}_{t_i} \leq 1 - W_{t_i} | \{Z_{t_i}\}_{i \geq 1}, \{t_i\}_{i \geq 1}, \{R(t_i)\}_{i \geq 1}] = F_{\hat{Y}|Z}(1 - W_{t_i} | Z_{t_i}) \quad [16.18]$$

From Eqs. (16.17) and (16.18) we have,

$$P(n_F > n) = E \left[\prod_{i=1}^n F_{\hat{Y}|Z}(1 - W_{t_i} | Z_{t_i}) \right] \quad [16.19]$$

It must be noted that in Eq. (16.19), W_{t_i} is a function of $\{t_i\}$ and $\{R(t_i)\}$ and the expectation is obtained over the distribution of $\{Z_{t_i}\}_{i \geq 1}$, $\{t_i\}_{i \geq 1}$ and $\{R(t_i)\}_{i \geq 1}$. The expression in Eq. (16.19) is general and as a special case, if $N(t)$ is a Poisson process with rate ν , then t_i has a Gamma distribution (Ang and Tang, 2007) with parameters ν and i where the mean is i/ν . In the case of no deterioration, $P(n_F > n) = (1 - P_f)^n$, where P_f is the probability of failure given the occurrence of a load.

- *Probability distribution of t_{n_F} .* Considering failures due to excessive demand only, the distribution for time to failure t_{n_F} is given by:

$$P(t_{n_F} > t) = E \left[\prod_{i=1}^{N(t)} P[\hat{Y}_{t_i} \leq 1 - W_{t_i} | \{Z_{t_i}\}_{i \geq 1}, \{t_i\}_{i \geq 1}, \{R(t_i)\}_{i \geq 1}] \right] \quad [16.20]$$

Therefore,

$$P(t_{n_F} > t) = E \left[\prod_{i=1}^{N(t)} F_{\hat{Y}|Z}(1 - W_{t_i} | Z_{t_i}) \right] \quad [16.21]$$

In particular, if $N(t)$ is a Poisson process with rate ν , then

$$P(t_{n_F} > t) = E \left[\sum_{n=0}^{\infty} \frac{(\nu t)^n e^{-\nu t}}{n!} \prod_{i=1}^n F_{\hat{Y}|Z}(1 - W_{t_i} | Z_{t_i}) \right] \quad [16.22]$$

- *Probability distribution of t_F .* Now considering the failures due to both excessive demand and excessive deterioration, the distribution for time to failure t_F is estimated as follows:

$$P(t_F > t) = P[n_F > N(t) \cap W_t < w_a] = E \left[1_{\{W_t \leq w_a\}} \prod_{i=1}^{N(t)} F_{\hat{Y}|Z}(1 - W_{t_i} | Z_{t_i}) \right] \quad [16.23]$$

where $1_{\{W_t \leq w_a\}} = 1$ if $W_t \leq w_a$ and 0 otherwise. If $N(t)$ is a Poisson process, then

$$P(t_F > t) = E \left[\sum_{n=0}^{\infty} \frac{(vt)^n e^{-vt}}{n!} 1_{\{W_t \leq w_a\}} \prod_{i=1}^n F_{\hat{Y}|Z}(1 - W_{t_i} | Z_{t_i}) \right] \quad [16.24]$$

If there is no deterioration then $t_F = t_{n_F}$ and

$$P(t_F > t) = E[P(t_{n_F} > t | N(t) = n)] = \sum_{n=0}^{\infty} (1 - P_f)^n P[N(t) = n] \quad [16.25]$$

These solutions for the cases with deterioration are referred to as semi-analytical, because the values of $E[\cdot]$ in Eqs. (16.17) to (16.24) have to be computed by simulating the processes $\{\hat{Y}_{t_i}\}$, $\{Z_{t_i}\}$ and $R(t)$. The computation of expectations in Eqs. (16.19), (16.21) and (16.23) is computationally more efficient than computing the expectation of a Bernoulli random variable (takes only values 0 and 1; Ang and Tang, 2007) which is normally practiced in a basic ‘hit and miss’ method to compute probabilities. This is because in the above-mentioned equations, the expectations are computed for continuous random variables that converge faster than the expectation of a Bernoulli random variable. The estimates of Eqs. (16.19), (16.21) and (16.23) are found to converge fast even for small failure probabilities of the order 10^{-6} .

As the solution requires simulations, the computation of the expectations may have simulation errors which must be reported. For assessing simulation errors, variance of the expectation estimated through simulations can be computed. This variance can be used to compute upper and lower bounds on the estimate. If $\bar{X}$ is the sample mean of X in the simulation, then $\text{Var}(\bar{X}) = \bar{V}(X)/m$, where $\bar{V}(X)$ is the sample variance of X and m is the number of simulations. Typically, $\bar{X} \pm 2\sqrt{\text{Var}(\bar{X})}$ can be used as upper and lower bounds for estimate of $\bar{X}$.

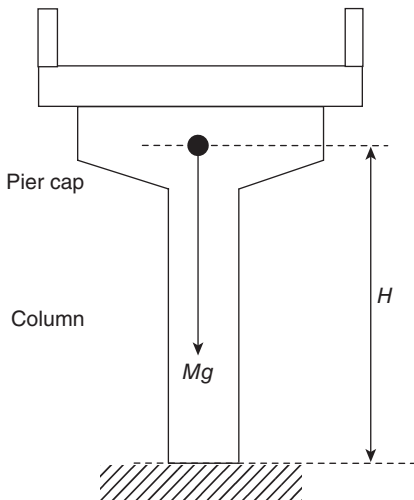
16.4 Stochastic modeling of deterioration in reinforced concrete (RC) bridges

As an illustration of the method described so far, this section models the effects of deterioration on RC bridge columns caused by the occurrence of earthquakes and corrosion of the reinforcing steel. For the purpose of this illustration, we analyze an RC highway bridge that is assumed to be located in San Francisco, CA, having the typical properties of a highway bridge

specified by Caltrans Seismic Design Specifications (Caltrans, 2006). The seismic characteristics of the bridge site, such as the probability of earthquakes and ground motion characteristics, used in this study are typical for San Francisco and are not specific to a particular site. San Francisco is selected because it is located in a seismically active region and due to its proximity to sea; structures are likely to experience corrosion.

16.4.1 Case study

Figure 16.2 shows a typical single-column bent used in RC highway bridges in California, where H is the column height and Mg is the weight of the bent where M is the mass and g is acceleration due to gravity. Table 16.1



16.2 A typical bridge bent (g = acceleration due to gravity).

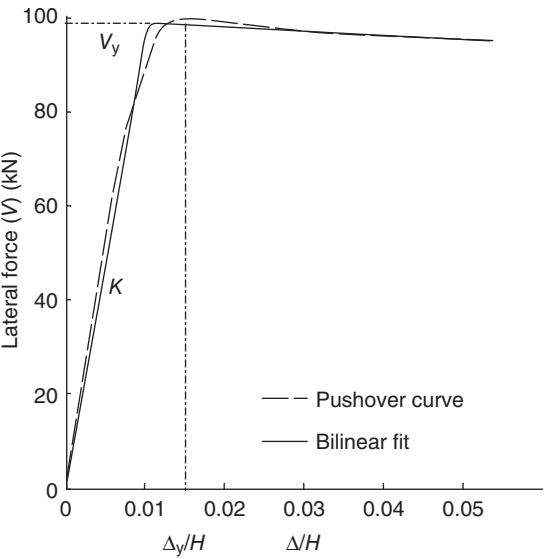
Table 16.1 Structural properties of the RC bridge column

Property	Value	Units
H	5000	mm
D_g	850	mm
H/D_g	5.88	
f'_c	27.56	MPa
f_y	551.00	MPa
f_{su}/f_y	1.58	–
ρ_l	2.20	%
ρ_s	0.13	%
$4P_u/\pi D_g^2 f'_c$	0.095	kN

gives the basic geometric and material properties for the example RC column. The column is designed to have properties typical of a bridge designed following Caltrans Seismic Design Specifications (Caltrans, 2006). In the table, in addition to the quantities already defined, D_g is the diameter of the column, f'_c is the compressive strength of concrete, f_y is the yield strength of reinforcing steel (same for longitudinal and transverse), f_{su} is the ultimate strength of steel, ρ_l is the ratio of the area of longitudinal steel to the gross cross-sectional area of the column, ρ_s is the volumetric ratio of confinement steel, and P_u is the axial load on the column. Table 16.2 shows the global structural properties for the bent computed using the pushover curve shown in Fig. 16.3, where V is the lateral force, Δ is the lateral displacement of the column top, K is the pre-yield stiffness, V_y is the shear force at yield, Δ_y is the displacement at yield, $M = P_u/g$ and $T_N = 2\pi\sqrt{M/K}$

Table 16.2 Global structural properties of the RC bridge bent

Property	Value	Units
T_N	0.71	s
Δ_y/H	0.01	—
V_y/Mg	0.30	—
M	151.6E03	kg



16.3 Pushover curve for the bent column.

is the natural period of vibration based on the single-degree-of-freedom (SDOF) idealization of the bent.

For this case study, we consider two modes of failure: deformation mode of the bent column (δ) and low-cycle fatigue of the longitudinal reinforcing steel (LC) in the column. In the deformation mode of failure, the lateral seismic loads generate excessive bending stresses in the RC column, causing the concrete to fail in compression. This is a critical mode of failure for seismically designed RC bridge columns owing to their dominant flexural behavior. The low-cycle fatigue mode of failure has also been reported as an important mode of failure for RC bridge columns subject to multiple earthquakes (e.g., El-Bahy *et al.*, 1999a,b; Kumar and Gardoni, 2012). In this mode of failure, the longitudinal steel undergoes low-cycle fatigue due to large strain reversals during cyclic loading. The low-cycle fatigue damage index DI increases with each load cycle and the longitudinal steel reinforcement bars rupture when $DI > 1.0$.

Along with the two modes of failure, we consider the following deterioration mechanisms that determine the time-dependence of the failure probability:

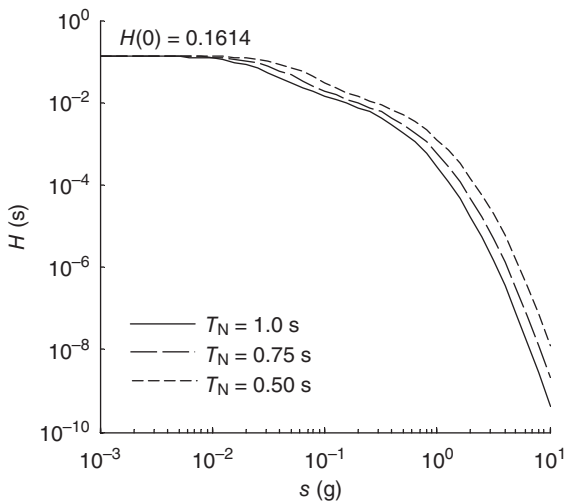
- Reduction in lateral stiffness due to the cyclic earthquake loads.
- Reduction in deformation capacity due to reduction in reinforcement area caused by corrosion.
- Reduction in lateral stiffness due to reduction in the reinforcement area caused by corrosion.

16.4.2 Seismic deformation demand and capacity

The seismic demand of a structure for a given ground motion can be evaluated by performing a time-history analysis. However, a time-history analysis can be computationally demanding and because future ground motions are not available, a large number of such analyses are required to compute the probability distributions of the demand quantities. As a result, probabilistic seismic demand models are better suited for computing such probability distributions. Following the general formulation proposed by Gardoni *et al.* (2002, 2003), probabilistic demand models can be developed akin to existing deterministic demand models and procedures commonly used in practice, but with additional correction terms that explicitly describe the inherent systematic and random errors inherent in the common models and procedures. Through the use of a set of ‘explanatory’ functions, terms that correct the bias in the existing models can be identified. A step-wise deletion process can then be used to construct parsimonious models by identifying which explanatory functions, from a set of potential ones, are more informative and which ones can be removed from the model with no significant

loss of information and accuracy. Systematic assessment of a measure of model quality can be made; thus, it is possible to differentiate between the accuracy of alternative candidate models. These explanatory functions provide the means for gaining insight into the underlying behavioral phenomena and selecting ground motion parameters that are most relevant to the seismic demands. The approach can take into account information gained from scientific/engineering laws, observational data from laboratory experiments, and data from simulated dynamic responses. The demand models are unbiased and properly account for all the predominant uncertainties, including model errors, arising from an inaccurate model form or missing variables, measurement errors, and statistical uncertainty. Additional examples of such probabilistic demand models can be found in, for example, Zhong *et al.* (2009) and Huang *et al.* (2010).

Following Gardoni *et al.* (2003) the deformation demand for RC bridges in terms of $\ln(\Delta/H)$, at time t_n is $D_\delta[\mathbf{x}_{t_n}, S_a(S_{t_n}, \mathbf{x}_{t_n}), \Theta_D]$, where $D_\delta(\mathbf{x}, S_a, \Theta_D)$ is the probabilistic deformation demand model, S_a is the pseudo spectral acceleration corresponding to T_N of the structure and 5% damping, $\mathbf{x}$ represents the structural properties, and Θ_D is a set of model parameters. To compute the probability distribution for $D_\delta(\mathbf{x}, S_a, \Theta_D)$ the probability distribution for S_a is needed. The CDF $F_{S_a}(s)$ for S_a for a given region and T_N is given by $F_{S_a}(s) = 1 - H(s)/H(0)$, where $H(s)$ is the regional seismic hazard curve for the given T_N . A seismic hazard curve defines the relation between a value of S_a and the annual probability of the value being exceeded. Figure 16.4 shows $H(s)$ for San Francisco for $T_N = 0.5$ s, 0.75 s and 1.0 s obtained



16.4 Seismic hazard curves for San Francisco, CA.

using *OpenSHA* (Field *et al.*, 2003). The figure shows that $H(0) = 0.1614$ independently from the value of T_N . This implies that the rate of occurrence of earthquakes ν for the region is 0.1614.

Using the probabilistic deformation capacity model by Choe *et al.* (2007), the capacity for deformation mode of the RC column in terms of $\ln(\Delta/H)$ at time t is given by $C_t = C_\delta(\mathbf{x}_t, \Theta_C)$, where $C_\delta(\mathbf{x}, \Theta_C)$ is the probabilistic deformation capacity model and Θ_C is a set of random model parameters. Note that $C_\delta(\mathbf{x}_0, \Theta_C)$ is a random variable due to the uncertainty in $\mathbf{x}_0$, Θ_C and random modeling error in the probabilistic model. In this illustration, the failure time analysis is conducted by conditioning on the value of $C_\delta(\mathbf{x}_0, \Theta_C)$. To account for the uncertainty in $C_\delta(\mathbf{x}_0, \Theta_C)$, estimates of the analysis can be multiplied by the probability density function (PDF) of $C_\delta(\mathbf{x}_0, \Theta_C)$ and the product can be integrated over the domain of $C_\delta(\mathbf{x}_0, \Theta_C)$. In this case, $C_\delta(\mathbf{x}_0, \Theta_C)$ is taken to be equal to the median capacity estimated following Choe *et al.* (2007). The median value of $C_\delta(\mathbf{x}_0, \Theta_C)$ is found to be -2.97 (i.e., drift capacity ≈ 0.05).

The probability of failure can be computed in the space of several deformation variables such as Δ (displacement), Δ/H (drift), or Δ/Δ_y (ductility). In addition, any monotonic transformation of these variables can also be used. Having a positive value for C_0 simplifies the modeling process because it allows setting α_0 equal to 0 in Eq. (16.11) and still satisfying the assumption made to write Eq. (16.12), i.e., $C_0 - \alpha_0 > 0$. Therefore, we choose $\ln(100\Delta/H)$ as the deformation variable which gives $C_0 = C_\delta(\mathbf{x}_0, \Theta_C) + \ln(100) = 1.64$. Following the described probabilistic demand and capacity models and using $\ln(100\Delta/H)$ as the deformation variable, we have $C_t = C_\delta(\mathbf{x}_t, \Theta_C) + \ln(100)$ and

$$D_{t_n} = D_\delta[\mathbf{x}_{t_n}, S_a(S_{t_n}, \mathbf{x}_{t_n}), \Theta_D] + \ln(100) \quad [16.26]$$

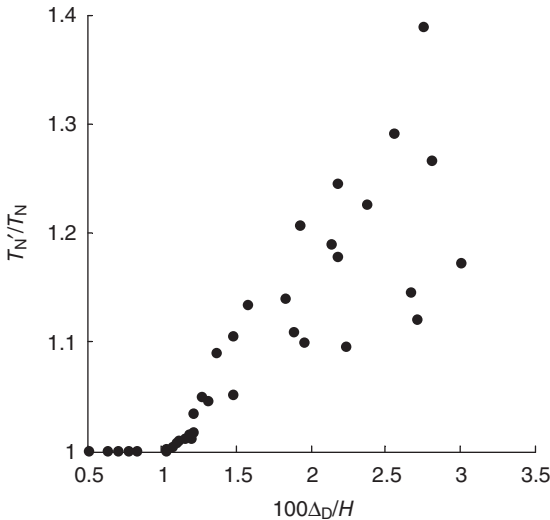
16.4.3 Stiffness deterioration due to earthquakes

The deterioration in the lateral stiffness due to earthquakes results in the change in the distribution of the deformation demand. This change can be modeled as the stochastic process $\{\Delta\alpha_{t_n}^s\}$ (see Eq. (16.6)). In this illustration, the shock deterioration in the deformation capacity $\{\Delta C_{t_n}^s\} = 0$ and therefore, for the failure mode δ , the total shock deterioration $Z_{t_n} = \Delta\alpha_{t_n}^s/u_0$. As seen in Eqs. (16.19), (16.21) and (16.23), to perform the failure time analysis, we need to compute the CDFs $F_Y(y)$, $F_Z(z)$, and $F_{Y|Z}(y|Z_{t_n})$. Now, assuming that $\alpha_0 = 0$ (as discussed earlier) we have $u_0 = C_0$, $Z_{t_n} = \Delta\alpha_{t_n}^s/C_0$, $\hat{Y}_{t_n} = D_\delta[\mathbf{x}_0, S_a(S_{t_n}, \mathbf{x}_0), \Theta_D]$ and $\hat{Y}_{t_n} = D_\delta[\mathbf{x}_0, S_a(S_{t_n}, \mathbf{x}_0), \Theta_D]/C_0$.

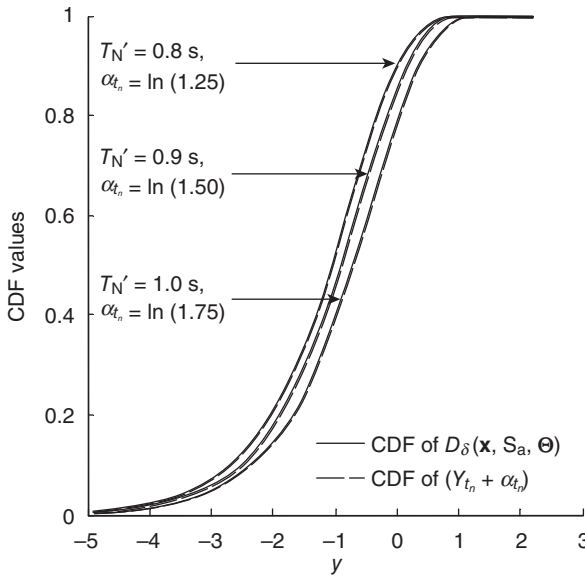
Now to compute $F_Z(z)$, and $F_{Y|Z}(y|Z_{t_n})$, we generate data representing the relation between $\hat{Y}_{t_n}$ and Z_{t_n} . For this purpose, we conduct nonlinear time-history analyses of the bent for ground motions selected to generate

stiffness deterioration data. We select the ground motions from Ground Motion Database (PEER, 2010) using the conditional mean spectrum (CMS) method (Baker, 2011). Accordingly, six ground motions each are selected corresponding to six S_a hazard levels (total 36 ground motions); 1% in 50 years, 2% in 50 years, 5% in 50 years, 10% in 50 years, 20% in 50 years and 50% in 50 years, so as to cover a wide range of intensity of ground motions. At the end of the analysis, we record the value of seismic drift demand given by the displacement demand Δ_D divided by H . We also compute the value of natural period T'_N of the deteriorated bent by conducting a free vibration test of the deteriorated bent. Figure 16.5 shows the data for T'_N/T_N versus $100\Delta_D/H$. It is observed that the $T'_N/T_N > 1$ for $100\Delta_D/H > 1$. This is because for $100\Delta_D/H < 1$ the column is in elastic state and receives no damage.

Since any change in the natural period results in a change in the distribution of the deformation demand, the shock deterioration $\Delta\alpha'_n$ can be computed as a function of T'_N/T_N . For this purpose, we compute the CDFs for $D_\delta(\mathbf{x}, S_a, \Theta_D)$ for the SDOF system in Table 16.2 by replacing T_N with T'_N but keeping the values of M and V_y constant so as to represent a deterioration process caused purely by stiffness deterioration. The values of T'_N are arbitrarily chosen as 0.8 s, 0.9 s and 1.0 s. Figure 16.6 shows that the CDFs corresponding to $T'_N = 0.8$ s, 0.9 s and 1.0 s are the same as the CDF for $Y_{t_n} + \alpha'_n$ with values of $\alpha'_n = \ln(1.25)$, $\ln(1.5)$ and $\ln(1.75)$ respectively. To an extent, this result supports the choice of $\ln(100\Delta/H)$ as the deformation



16.5 Data regarding Δ_D/H and T'_N/T_N obtained from nonlinear time history analyses.

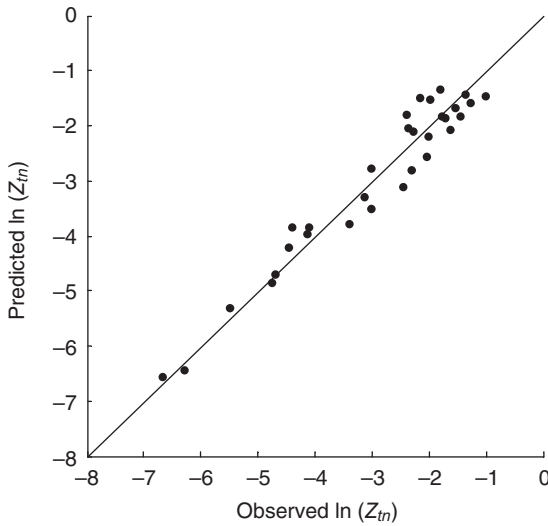

 16.6 Comparison of CDFs of $D_\delta(\mathbf{x}, S_a, \Theta)$ and $(Y_{t_n} + \alpha_{t_n})$.

variable to satisfy the additive form in Eq. (16.6). From the relation between α_{t_n} and T_N'/T_N , and recalling that $\alpha_0 = 0$, we can write $\Delta\alpha_{t_n}^s = \ln[1 + 1.75(T_N'/T_N - 1)]$. From the data in Fig. 16.5, we compute $\hat{Y}_{t_n} = \ln(100\Delta_D/H)/C_0$ and $Z_{t_n} = \Delta\alpha_{t_n}^s/C_0$. Now, we develop a regression model $\zeta(y)$ such that $Z_{t_n} = \zeta(\hat{Y}_{t_n})$. Linear regression analysis with the following model form is performed to develop the model:

$$\ln[\zeta(y)] = \begin{cases} \theta_{z\delta 1} \ln(y) + \theta_{z\delta 2} + \sigma_{z\delta} \varepsilon_{\delta z} & y > 0 \\ 0 & y \leq 0 \end{cases} \quad [16.27]$$

where $\theta_{z\delta 1}$, $\theta_{z\delta 2}$ and $\theta_{z\delta}$ are the model parameters and $\varepsilon_{\delta z}$ is a standard normal random variable. The values of $\theta_{z\delta 1}$, $\theta_{z\delta 2}$ and $\theta_{z\delta}$ are found to be 1.332, -1.477 and 0.345 respectively. Figure 16.7 shows the plot of predicted versus observed values of $\ln(Z_{t_n})$ for the developed regression model. Next, we need to develop models for $F_z(z)$ and $F_{y|z}(y|Z_{t_n})$. The CDF $F_z(z) = P[\zeta(\hat{Y}_{t_n}) \leq z]$ and the conditional CDF $F_{y|z}(y|Z_{t_n})$ is computed as follows:

$$F_{\hat{Y}|Z}(y|Z_{t_n}) = \begin{cases} P(\hat{Y}_{t_n} < y | Z_{t_n} = 0) & z = 0, y \leq 0 \\ 1 & z = 0, y > 0 \\ P(\hat{Y}_{t_n} < y | Z_{t_n} = z, Z_{t_n} > 0) & z > 0, y > 0 \\ 0 & \text{otherwise} \end{cases} \quad [16.28]$$



16.7 Plot showing observed versus predicted values of $\ln(Z_{tn})$ using linear regression.

where,

$$P(\hat{Y}_{tn} < y | Z_{tn} = 0) = P(\hat{Y}_{tn} < y | \hat{Y}_{tn} < 0) = \frac{F_{\hat{Y}}(y)}{F_{\hat{Y}}(0)} \quad [16.29]$$

The probability $P(\hat{Y}_{tn} < y | Z_{tn} = z, Z_{tn} > 0)$ can be found by developing a Gaussian copula (Nelsen, 2006) for the distributions $F_{\hat{Y}|Z_{tn}>0}(y) = P(\hat{Y}_{tn} < y | \hat{Y}_{tn} > 0)$ and $F_{Z|Z_{tn}>0}(z) = P(Z_{tn} < z | Z_{tn} > 0)$ (from Eq. (16.27), the event $Z_{tn} > 0$ is the same as the event $Y_{tn} > 0$). Now, we introduce the functions: $G_{\hat{Y}}(\cdot) = \Phi^{-1}[F_{\hat{Y}|Z_{tn}>0}(\cdot)]$ and $G_Z(\cdot) = \Phi^{-1}[F_{Z|Z_{tn}>0}(\cdot)]$. Then, we simulate a random variable $\hat{Y}^*$ that has CDF $F_{\hat{Y}|Z_{tn}>0}(y)$ and compute $Z^* = \zeta(\hat{Y}^*)$. This implies that $U_{\hat{Y}} = G_{\hat{Y}}(\hat{Y})$ and $U_Z = G_Z(Z^*)$ are standard normal random variables. We then fit a bivariate standard normal distribution on the data $(U_{\hat{Y}}, U_Z)$. Now, we can compute $P(\hat{Y}_{tn} < y | Z_{tn} = z, Z_{tn} > 0)$ in the space of $U_{\hat{Y}}$ and U_Z as follows:

$$P(\hat{Y}_{tn} < y | Z_{tn} = z, Z_{tn} > 0) = P[U_{\hat{Y}} < G_{\hat{Y}}(y) | U_Z = G_Z(z)] \quad [16.30]$$

The quantity on the left hand side of the equation can be easily obtained from a bivariate standard normal distribution. The correlation coefficient $\rho_{U_{\hat{Y}}U_Z}$ between $(U_{\hat{Y}}, U_Z)$ for the given is found to be 0.45.

16.4.4 Effect of corrosion

It is shown by Choe *et al.* (2008) that the reduction in the diameter of reinforcement bars due to corrosion is given by:

$$d_b(t|T_{\text{corr}}) = \begin{cases} d_{b0} & t \leq T_{\text{corr}} \\ d_{b0} - 1.0508(1 - w/c)^{-1.64} (t - T_{\text{corr}})^{0.71} & T_{\text{corr}} < t \leq T_f \\ 0 & t > T_f \end{cases} \quad [16.31]$$

where, $d_b(t|T_{\text{corr}})$ = diameter of bar at t and T_{corr} = the corrosion initiation time given by:

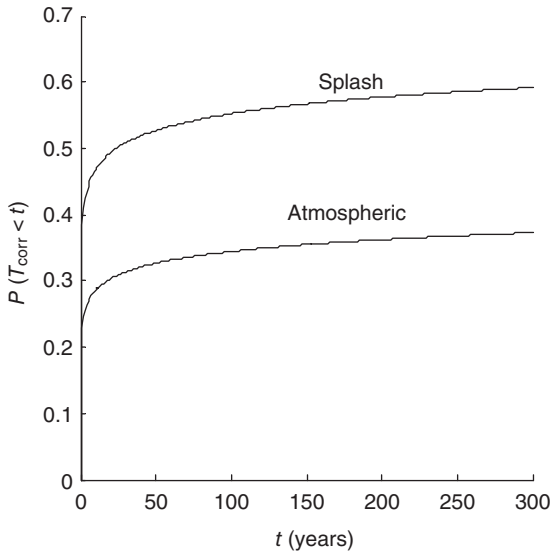
$$T_{\text{corr}} = X_I \left\{ \frac{d_c^2}{4k_e k_t k_c D_c (t_{\text{ref}})^q} \left[\text{erf}^{-1} \left(1 - \frac{C_{\text{cr}}}{C_{\text{cs}}} \right) \right]^2 \right\}^{1/(1-q)} \quad [16.32]$$

where d_c = reinforcement cover depth, k_e = environmental factor, k_t = factor representing the influence of test methods to determine the empirical diffusion coefficient D_c , k_c = parameter that accounts for the influence of curing, t_{ref} = reference period for D_0 , X_I = model uncertainty coefficient to account for the idealization by Fick's second law, $C_{\text{cs}} = A_{\text{cs}}(w/b) + \varepsilon_{\text{cs}}$ is the chloride concentration on the surface, w/b = water to binder ratio, A_{cs} and ε_{cs} are the model parameters, C_{cr} = critical chloride concentration, and $\text{erf}(\cdot)$ is the error function. In Eq. (16.32), $T_{\text{corr}} < \infty$ for $C_s \geq C_{\text{cr}}$ and $T_{\text{corr}} = \infty$ for $C_s < C_{\text{cr}}$. Since C_{cr} and C_{cs} are independent random variables, $P(T_{\text{corr}} = \infty) > 0$.

Table 16.3 lists the values of the model parameters for two levels of exposure to chlorides: atmospheric and splash. The atmospheric exposure level refers to the condition where moisture in the atmosphere contains chloride ions. The splash exposure level refers to cyclic spraying of salt water (containing chlorides) on the structure's surface. The values shown

Table 16.3 Model parameters for prediction of T_{corr}

	Mean	Standard deviation	Mean	Standard deviation	
Variable	Atmospheric		Splash		Distribution
D_c	220.9 (mm ² /yr)		801.01 (mm ² /yr)	801.01 (mm ² /yr)	Normal
q	0.362	0.245	0.362	0.245	Beta
k_e	0.362	0.245	0.362	0.245	Gamma
k_c	0.8	0.1	0.8	0.1	Beta
k_t	0.832	0.024	0.832	0.024	Normal
X_I	1	0.05	0.265	0.045	Lognormal
A_{cs}	2.565	0.356	7.758	1.36	Normal
ε_{cs}	0	0.405	0	1.105	Normal
C_{cr}	0.80	0.10	0.80	0.10	Normal

16.8 Cumulative distribution function for T_{corr} .

in the table correspond to water-binder ratio of 0.4 for concrete. Figure 16.8 shows the CDFs for the variable T_{corr} for atmospheric and splash conditions. The CDFs for T_{corr} are found to be asymptotic to a value less than 1.0 because there is a positive probability that $T_{\text{corr}} = \infty$.

Using Eq. (16.31), C_t is computed by using $d_b(t | T_{\text{corr}})$ in computing $C_\delta(\mathbf{x}_t, \Theta_C)$. We use the reduced cross-sectional area for longitudinal and transverse steel to compute $C_\delta(\mathbf{x}_t, \Theta_C)$. It is found that:

$$R_C(t) = C_0 \left[1.4 \times 10^{-6} (t - T_{\text{corr}}) - 3.516 \times 10^{-4} (t - T_{\text{corr}})^2 \right] \quad [16.33]$$

To assess the effect of corrosion on seismic deformation demand, we compute the natural period T'_N of the deteriorated structure as a function of time t . This is computed by conducting pushover analysis with reduced steel area at different values of t . It is found that for the bent $T'_N = 5.0 \times 10^{-4} (t - T_{\text{corr}}) + T_N$. As observed in the case of stiffness degradation due to earthquakes, the change in natural period can be used to compute α_t . The type of deterioration process (i.e., corrosion or earthquakes) does not affect the relation between α_t and T'_N/T_N as long as the deteriorated state is the same (in this case T'_N/T_N). Therefore, the previously developed relation $\alpha_t = \ln[1 + 1.75(T'_N/T_N - 1)]$ can be used with the difference that in this case α_n is replaced with α_t . Now, we obtain

$$R_\alpha(t) = \ln[1 + 12.5 \times 10^{-4} (t - T_{\text{corr}})] / C_0 \quad [16.34]$$

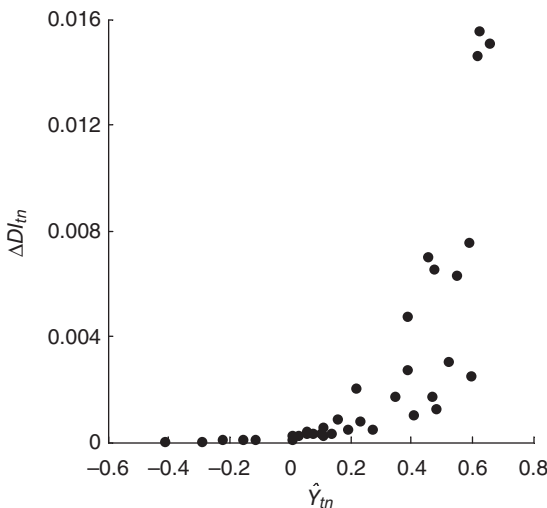
Using the expressions for $R_C(t)$ and $R_a(t)$, we obtain

$$R(t) = 3.516 \times 10^{-4} (t - T_{\text{corr}})^2 - 3.443 \times 10^{-4} (t - T_{\text{corr}}) + 0.61 \ln[1 + 12.5 \times 10^{-4} (t - T_{\text{corr}})] \quad [16.35]$$

16.4.5 Modeling low-cycle fatigue damage due to earthquakes

As mentioned earlier, the damage index DI accumulates with every passing damaging earthquake and, a longitudinal reinforcement bar is expected to fail due to low-cycle fatigue when $DI > 1.0$. Therefore, writing incremental damage due to S_{t_n} as ΔDI_{t_n} , we need to compute the distribution of ΔDI_{t_n} to compute n_F and t_F . In this case, the computation of distribution of n_F simplifies to $P(n_F > n) = P\left(\sum_{i=1}^n \Delta DI_{t_i} < 1.0\right)$ and for that of t_F to

$P(t_F > t) = P\left(\sum_{i=1}^{N(t)} \Delta DI_{t_i} < 1.0\right)$. In order to compute the CDF $F_{\Delta DI}(d)$ for ΔDI_{t_n} , we first develop a function $\Delta DI_{t_n} = \Delta DI(\hat{Y}_{t_n})$ and then compute $F_{\Delta DI}(d) = P(\Delta DI_{t_n} < d) = P[\Delta DI(\hat{Y}_{t_n}) < d]$. To develop $\Delta DI(\hat{Y}_{t_n})$, we perform a regression analysis using the results of time-history analyses conducted earlier. First, we compute ΔDI_{t_n} for each ground motion by analyzing the strain cycles induced in the reinforcement bars by the ground motion and following the methodology in Brown and Kunnath (2004) to compute low-cycle fatigue. Figure 16.9 shows the data generated for ΔDI_{t_n} and $\hat{Y}_{t_n}$ ($\hat{Y}_{t_n}$ for each



16.9 Plot showing data regarding ΔDI_{t_n} and $\hat{Y}_{t_n}$.

ground motion was computed in Section 16.4.3 to model the stiffness deterioration). Next, we perform a linear regression analysis of the data in logarithmic space as follows:

$$\ln(\Delta DI_{in}) = \begin{cases} \theta_{ZLC1} \ln(\hat{Y}_{in}) + \theta_{ZLC2} + \sigma_{ZLC} \epsilon_{ZLC} & \hat{Y}_{in} > 0 \\ 0 & \hat{Y}_{in} \leq 0 \end{cases} \quad [16.36]$$

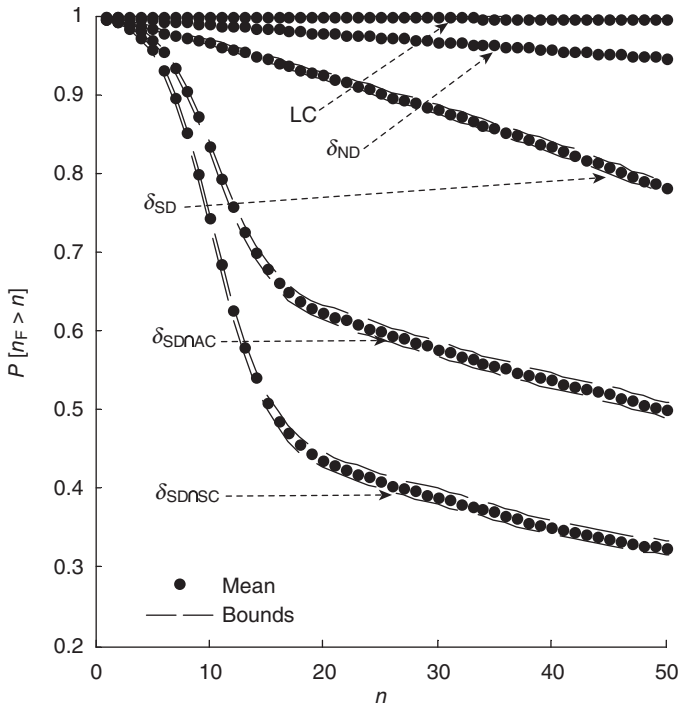
where we obtain $\theta_{ZLC1} = -6.0$, $\theta_{ZLC2} = -8.45$, and $\sigma_{ZLC} = 0.54$, and ϵ_{ZLC} is a standard normal random variable.

16.4.6 Analysis, results and discussion

Deterioration for different combinations of modes of failures, deterioration mechanisms, and environmental conditions are analyzed. Depicting ND as no deterioration, SD for seismic stiffness deterioration, AC for atmospheric level corrosion and SC for splash level corrosion, we analyze the cases δ_{SD} , $\delta_{(SD \cap AC)}$, $\delta_{(SD \cap SC)}$ and LC , where k_A denotes failure in mode k considering deterioration scenario A . For the analysis, it is assumed that $w_a = -\log(0.1)/C_0 = 1.40$. This value is adopted for w_a because if $W_i > 1.40$ then a small deformation demand of approximately $> 0.1\%$ drift (i.e., 1/10th of drift at yield) is sufficient to collapse the structure. The accuracy in the value of w_a is not of significance for the presented case because it is found that t_F is not significantly sensitive to w_a . Figure 16.10 shows the plots for $P(n_F > n)$ for the three mentioned scenarios. It is found that $P(n_F > n)$ follows the order; $LC > \delta_{ND} > \delta_{SD} > \delta_{(SD \cap AC)} > \delta_{(SD \cap SC)}$. This is expected because the column is likely to deteriorate faster when subject to both corrosion and seismic stiffness deterioration than seismic stiffness deterioration alone. Furthermore splash level exposure causes faster deterioration than atmospheric level because corrosion is expected to start earlier under splash level exposure. Similarly, Fig. 16.11 shows the plots for $P(t_F > t)$ for the above mentioned cases. It is found that $P(t_F > t)$ follows the order as previously observed i.e., $LC > \delta_{ND} > \delta_{SD} > \delta_{(SD \cap AC)} > \delta_{(SD \cap SC)}$, and that the slope of $P(t_F > t)$ decreases significantly around $t = 75$ years. This happens due to the shape of the CDF of T_{corr} (see Fig. 16.8) where the slope decreases significantly indicating that most of the deterioration is likely to occur for $t < 75$ years. This indicates that for these cases, the CDF for T_{corr} primarily controls $P(t_F > t)$.

16.5 Conclusions

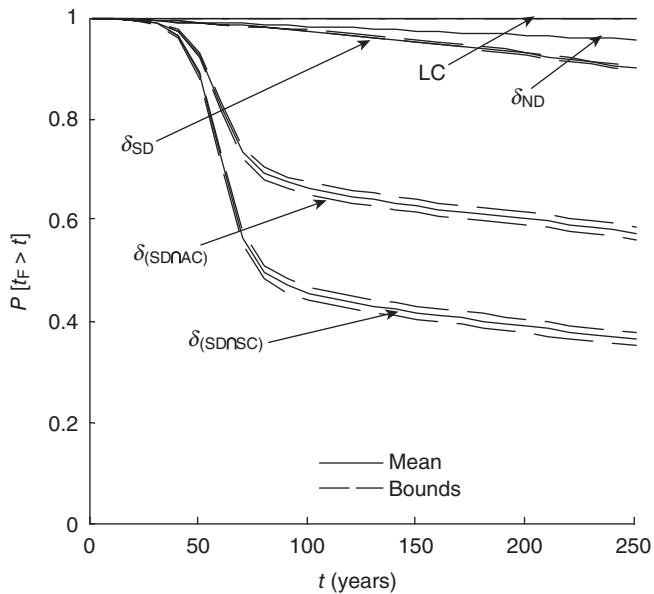
Structural deterioration of infrastructure systems is a serious concern because it might decrease the lifespan and reliability of the systems. In addition, the prediction of structural deterioration can be helpful in optimizing design of infrastructure systems accounting for their life cycle. In



16.10 Probability distribution for n_F considering effects of corrosion and earthquakes.

particular, with the advent of performance-based engineering methodology, models to predict life-cycle performance of infrastructure systems are needed. In this chapter, deterioration in a typical RC highway bridge bent subject to earthquakes and corrosion of reinforcement is modeled. For this purpose we used a stochastic formulation that combines shock deterioration due to earthquakes and gradual deterioration due to corrosion. Three deterioration mechanisms are focused upon: loss of lateral stiffness of the bridge columns due to earthquakes, loss of lateral stiffness of the bridge columns due to corrosion, and loss of deformation capacity due to corrosion and low-cycle fatigue of longitudinal reinforcement.

The results show that deterioration due to corrosion of reinforcement and seismic stiffness deterioration have a significant effect of the lifespan of the bridge bent. The effect of low-cycle fatigue damage is found to be insignificant on the expected lifespan of the bridge bent. The results also show that it is important to study the combined effect of corrosion and seismic damage. The obtained results are specific for the scenarios that are similar to the analyzed one (i.e., type of bent, regional seismicity, and



16.11 Probability distribution for t_F considering the effects of corrosion and earthquakes.

corrosive environment) and thus observations/conclusions made in this study should not be directly extended to other deterioration scenarios. Infrastructure systems are complex systems and modeling their life cycle in the presence of multiple sophisticated and interacting deterioration processes is highly challenging. However, the benefits of continuing research in this area are also considerable. In order to advance the state-of-the-art, work is required to model the interaction between various deterioration mechanisms, modeling non stationary deterioration processes and developing robust frameworks to incorporate laboratory data in the models.

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Risk assessment and management of civil infrastructure networks: a systems approach

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Abstract: Infrastructure networks are complex systems due to the large number of components that interact in a nonlinear way. Detecting and understanding the properties of such systems is of paramount importance to make effective decisions about risk management and sustainable development. This chapter presents a systems approach to risk management and risk-based decision making in infrastructure networks. In the proposed approach, the internal structure of a network is detected via pattern recognition (clustering), and structured information is used to enhance conceptual and computational analyses of reliability, vulnerability, damage propagation, and resource allocation. The approach can be applied to network analysis of complex infrastructure systems subjected to extreme events, such as earthquakes.

Key words: infrastructure, transportation networks, systems thinking, risk assessment, decision-making, optimization.

17.1 Introduction

This chapter provides a framework for risk-based decision-making in complex infrastructure networks and a set of applications, including reliability, vulnerability, and damage propagation analyses, as well as optimal resource allocation. The discussion is presented within the context of systems thinking, which considers properties and relationships of networks' components at different levels of abstraction, and emphasizes the role of the decision-maker in the problem. In such an approach, network features are extracted by means of clustering methods and used to gain insight on system properties and simplify the computations associated with risk assessment and management of infrastructure networks.

Systems can be viewed as a way of structuring information. The term *system* (in Greek: *sustema-ma*) denotes a set of connected things or parts or an organized body of things. Its broad interpretation makes it a concept that can be used in several domains (e.g., biology, politics, and engineering). Most, if not all, systems allow a description in terms of networks of interacting components (nodes) and their relationships (links).

A network is a representation of a set of interacting elements. The last decades of the twentieth century brought significant development in the field of network modelling; the modelling and analysis of biological, physical, and social systems, as well as the availability of computational resources facilitated the formalization of network research (Watts, 2004). Many problems are treated from a network perspective, e.g., the Internet network, transportation, and communication (Gómez *et al.*, 2011a,b,c,d), as well as less evident ones, such as protein chains (Barzel and Biham, 2009) and percolation theory (Stauffer and Aharony, 1992). Network theory condenses models, measures, and algorithms that are useful to describe and quantify the form (topology) and performance (flow and dynamics) of networks.

Section 17.2 provides basic notions of systems thinking and network modelling upon which the proposed framework is constructed. It describes how systems can be described as the recursive embedding of subsystems in a hierarchical fashion and offers a basic theoretical background, such as network representation and computation of network performance indices. Section 17.3 provides an introduction into network clustering as a pattern recognition technique that facilitates the hierarchical decomposition of infrastructure networks into sub-networks. Section 17.4 deals with risk assessment of infrastructure networks, focusing on strategies to exploit benefits of the proposed hierarchical model for the assessment of reliability, vulnerability, and damage propagation. Finally, Section 17.5 discusses relevant decision-making aspects in infrastructure systems and proposes optimization strategies for resource allocation, based on the proposed hierarchical model.

17.2 Systems and networks

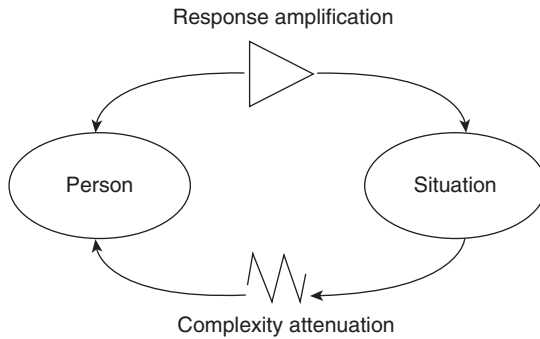
17.2.1 Systems thinking

The rise of cybernetics (Wiener, 1948; Ashby, 1964) contributed to the efforts towards a unified theory of systems through the analysis of shared properties of systems across several disciplines and domains. Further theories and applications were developed based on such knowledge, including organizational cybernetics and applied systems thinking (Beer, 1967; Checkland, 1981; Blockley and Godfrey, 2000; Senge, 2006; Mingers and White, 2009). Systems thinking (Ackoff, 1962; Churchman, 1963) emerged as an oppositional paradigm to reductionist tendencies (Looijen, 1999) in the first half of the twentieth century. Systems thinking not only studies the parts of a system but also the relationships between them, thus incorporating concepts such as wholes, parts, hierarchy of subsystems, and emergent properties.

A system is described in terms of a purpose, which identifies it as a whole; e.g., the nervous system as a unit is distinguished from other systems within an organism. The parts of a system can also be considered as a whole in terms of different purposes; e.g., a neuron, as part of the nervous system, is a whole for the purpose of perception. The latter corresponds to the concept of *holon*: an element which is part and whole at the same time. The way in which systems are embedded into more complex systems to accomplish more complex purposes leads to a hierarchical structure. The concept of hierarchy usually implies relationships of command, but in this context, it will be used to describe recursion of systems and subsystems. Emergent properties are those that can be observed in the whole, but not in its parts in isolation; e.g., cognition is a property of the nervous system, not of neurons.

A system can be defined as a set of interacting elements exhibiting closure and synergy: closure implies that the network of elements is perfectly distinguishable from its environment, while synergy implies that the whole is more than the sum of the parts (Checkland, 1981; Blockley and Godfrey, 2000). The definition of a system is tied to an observer, who ascribes a purpose to it: 'everything is said by an observer' (Maturana, 1987). There is a boundary that separates the system and its environment. Moreover, the system may interchange energy, matter, and information with its environment. Therefore, the system interacts with the environment/observer. In the context of this chapter, this means that the definition and analysis of systems must be tied to decision-makers' interests.

The British cybernetician Ross Ashby (1964) defined *variety* as the number of possible states that a system can take. This concept accounts for potentiality, whereas *complexity* accounts for actuality. For example, a traffic light (having three lights with two states, on/off) might take $2^3 = 8$ different states (variety), but its complexity is lower (at most 4 states) because not all permutations are part of the system's behaviour that is relevant to observers. Furthermore, uncertainty complicates the definition and treatment of these issues. Ashby's Law of Requisite Variety implies that only complexity absorbs complexity. Thus, a decision maker cannot cope with complexity of a system (e.g., infrastructure) that exceeds his/her own complexity (understood as the capacity to deal with the problem). It is necessary to apply complexity attenuators and response amplifiers (Beer, 1967) in order to deal with complex problems appropriately (Fig. 17.1). For instance, an organization may attenuate the complexity of customer service by using automated phone-call response to filter requests that do not need personalized attention; on the other hand, response amplification could be achieved by publishing online help information about common customer requests. The proposed systems approach uses clustering methods to filter/aggregate information from networks, which acts both as an attenuator of complexity



17.1 Complexity attenuation.

(data) for a decision-maker and a response amplifier, since decisions made for a specific cluster spread to all its constituents.

17.2.2 Network modelling

Graph theory provides a natural way of modelling networks (Brandes and Erlebach, 2005; Bondy, 2008). A graph $G(V, E)$ is an abstract data structure, which consists of a set of nodes (vertices) $V = \{v_1, v_2, \dots, v_n\}$ and a set of connecting edges (also called links or arcs) $E = \{e_1, e_2, \dots, e_m\}$ whose end-points are elements of V . Vertices can represent any kind of elements (e.g., molecules, facilities, people), whereas links represent relationships between them (e.g., attraction, distance/flow, or similarity/friendship).

The network structure (i.e., how elements are connected) is defined either by the incidence matrix of a graph, or its adjacency matrix. The incidence matrix B has n rows and m columns indicating relationships between nodes and edges. Then, if nodes i and j are connected by link k , the elements B_{ik} and B_{jk} are taken as 1, or 0 otherwise. The adjacency matrix, on the other hand, is a useful alternative to describe how vertices are connected, and is denoted by a matrix $A_{n \times n}$ such that $a_{ij} = 1$ if nodes v_i and v_j are connected, and $a_{ij} = 0$ otherwise.

A graph is said to be directed if A_{ij} and A_{ji} are different, i.e., the relationship from i to j is different from that from j to i . Note that if G is an undirected graph, its adjacency matrix is symmetrical. Weighted graphs can be used to represent infrastructure networks, where edge values are other than binary, capturing characteristics, such as cost and capacity between pairs of nodes. Commonly, more than one attribute is necessary to describe a real-world system; in this case, a multi-graph, with more than one link per pair of nodes, is used.

The purpose of a network can be described by a multi-attribute function g . In most cases, g is described in terms of form or flow. Models that focus on the form of the network concentrate on changes in its topological structure (i.e., the way in which elements are connected) and address connectivity of the system. Models that focus on flow deal with cost and efficiency of the system. Flow in a general sense describes commodity transport through a network, such as traffic in transportation systems. Although form and flow are linked concepts, their relationship might be difficult to generalize and assess explicitly.

Performance measures are used to assess the state of individual links and nodes in a network (i.e., local indices) or the state of the whole network (i.e., global indices). A list and description of basic indicator is presented next (Brandes and Erlebach, 2005), whilst other widely used indices in the context of infrastructure can be found elsewhere (Garrison and Marble, 1962; Dueñas-Orsorio *et al.*, 2007).

- *Local indices:* Local network measures are concerned with network links or nodes, emphasizing their relative topological characteristics and focusing on indicators of individual nodes, set of nodes or pre-specified paths. For instance:
 - Distance: shortest-path between two nodes, i.e., the sum of attributes in the links of the path (e.g., cost).
 - Degree: number of neighbours of a node (links going inside and/or outside a node).
 - Betweenness: nodes/links that occur on many shortest-paths have higher betweenness than those that do not.
- *Global indices:* Global measures provide a single metric that summarizes the characteristics of the entire network and can be used to compare different networks. Such indices are conceptually straightforward but computationally demanding (e.g., the maximum number of network cycles of different lengths). For instance, the clustering coefficient accounts for the probability that nodes a and b are connected, given that they share a common neighbour c . Most local indices can be taken to a global level by taking the average, maximum, and/or minimum (depending on the type of measure) over all nodes/links, paths, or pairs of nodes (e.g., average shortest-path).

These measures are neutral in nature but decision makers can introduce their attitudes towards risk by specifying acceptable thresholds with respect to the probability of an index exceeding a certain level. Moreover, statistical analyses can be made about the distribution of metrics such as distance, or degree, throughout the network to determine the frequency with which different values occur. For instance, a common measure is the degree distribution of a network, which indicates the probability that a randomly

chosen node has degree k . The k -connectivity of a graph is a popular global measure that indicates the number of independent paths between source and target nodes, i.e., it indicates the minimum number of nodes that need to be removed to disconnect the network.

Network modelling poses significant computational challenges. With the exception of small networks, or certain geometric characteristics (Duenas-Osorio and Rojo, 2011), analytical and/or numerical closed-form solutions of connectivity and flow efficiency cannot be easily obtained. The identification of all possible failure scenarios is commonly known as an NP-hard (nondeterministic polynomial time hard) problem due to the enormous number of possible states (e.g., a network with n on-off elements has 2^n possible states). In other words, it is unlikely that there is a general algorithm that can compute all possible cases under reasonable time constraints.

Several approaches have been proposed to deal with this problem, for example, Konak *et al.* (2004) divided alternative approaches in three groups: (1) state-based techniques, which ‘aim to generate state vector more efficiently [...] or to introduce negative correlation between sampled state vectors’; (2) ‘sample space of arc permutations instead of the original sample space S [...], which [...] are particularly efficient if all arcs have identical reliability’; and (3), bounding techniques, which includes ‘stratified and important sampling variance reduction techniques’ (Konak *et al.*, 2004).

The proposed approach is motivated by the possibility of detecting topological features (i.e., a cluster-based hierarchy) of complex systems to addresses decision-makers’ needs by allowing the use of simplified, surrogate network models (see Section 17.3). The process of network decomposition allows redefining any network problem in terms of communities and is not intended to be an approach limited to reliability computation, but to general network problem solving.

17.3 Hierarchical representation of networks

17.3.1 Basic notions on clustering

Clustering is a pattern recognition technique that can classify a set of unlabelled data into multiple classes; unlabelled means that the algorithm is not previously trained to recognize classes (i.e., unsupervised learning). The objective of network clustering methods is to generate a partition of the network into k subgroups; essentially, it seeks to maximize intra-cluster density and inter-cluster sparsity. Each sub-group includes a centroid, which is representative of the group, and constitutes a zone referred to as a Voronoi region, which is determined by a convex set containing the nodes

for which such centroid is the nearest according to a pre-specified measure of distance/similarity. Thus, the network representation is given by the union of the Voronoi regions, each including a centroid and its associated elements (Filippone *et al.*, 2008).

Conceptually, most approaches to clustering are based on developing a similarity measure m_{ij} between pairs of vertices (v_i, v_j) and involve an iterative process of vertex grouping up to a point where a maximum similarity value is achieved. In infrastructure systems, similarity is commonly defined as a measure of distance between two vertices; two cases in point are the Euclidean distance or the Pearson correlation between columns (or rows) of the adjacency matrix (Wasserman and Faust, 1994). The selection of a similarity function defines the clustering process. Therefore, the same network may produce different clusters depending upon the similarity function used.

Because of the complexity of the clustering problem, exact optimization is not practical. Commonly, evolutionary computation and other heuristics are used. The evaluation of clustering performance is a difficult task since an expected solution or correct partitioning is usually unavailable. Consequently, most often the quality of clustering is measured based on the structure of the partitioning and relational characteristics of the network. Many approaches seek to measure the balance of density within clusters versus sparsity between clusters.

There is no standard approach for network clustering; it depends on the context, type of network, objective, and computational cost. Clustering methods can be classified into partitional (Newman, 2004; Newman and Leicht, 2007) and hierarchical methods, which in turn can be divided into top-down and bottom-up (Peng and Xia, 2005), depending on whether they construct hierarchies by dividing the whole graph into partitions or by merging individual elements until the graph is complete.

Although clustering is unsupervised in nature, some methods may frequently require a priori information; thus, there is a distinction between supervised and unsupervised algorithms. Some examples of the former are the NJW-Method (Ng *et al.*, 2001) and the k -means algorithm (McQueen, 1967), which require information about the number of clusters in advance. On the other hand, a well-known unsupervised method is the Markov clustering algorithm, MCL (van Dongen, 2000). Furthermore, more elaborate clustering methods include kernel-based methods (Graepel *et al.*, 1998; Scholkopf *et al.*, 1998; Inokuchi and Miyamoto, 2004) and spectral methods (Meila and Shi, 2001; Ng *et al.*, 2001; Verma and Meila, 2003). The basic idea behind kernel-based methods is to implicitly map data to high-dimensional spaces where the classification can be done more easily, whereas spectral methods rely on eigenvalue analysis to perform partitionings.

17.3.2 Hierarchical model of infrastructure networks

Building a hierarchy that represents network communities at different levels is the core idea behind the proposed approach. This is carried out by a process of successive clustering of the network. Therefore, a recursive implementation of clustering is necessary to construct a hierarchy. Moreover, variations in parameters may be necessary from level to level in order to achieve different levels of detail and cluster sizes.

A clustering $C = \{C_1, \dots, C_k\}$ of a graph G is a partition of the node-set V into non-empty (disjoint) subsets C_i such that V is given by the union of them. The set $E(C_i, C_j)$ represents the set of edges with origin in cluster C_i and destination in cluster C_j ; $E(C_i)$ is the short notation for $E(C_i, C_i)$. Then, $E(C)$ is the set of intra-cluster edges $E(C)$, thus $E/E(C)$ (node-set without), is the set of inter-cluster edges (Brandes and Erlebach, 2005). A clustering is a refinement of other if it is a partitioning of higher resolution, or a coarsening in the opposite case. Cases where the number of clusters k is equal to 1 or the number of nodes n are referred to as a trivial clusterings. A chain of clusterings is called a hierarchy. Multi-level network clustering provides structured organization of a system, which enhances knowledge of network properties.

Aiming to provide a model for a highly complex system, it is proposed that networks are partitioned recursively based on pertinent criteria (Gómez *et al.*, 2011a). Each level of recursion (i.e., each different partitioning of the network) denotes a level of abstraction. The hierarchical clustering leads to a system representation in the form of a tree, also called a dendrogram (from Greek *dendron* ‘tree’ and *gramma* ‘drawing’). The concept of hierarchical representation of a system has been widely discussed for hard (e.g., infrastructure) and soft (e.g., sociological) systems (Blockley and Godfrey, 2000; Scott 2000).

- *Fictitious nodes*: Clusters at all levels define fictitious nodes. Let $V_i^{(lv)}$ be the i th node at the hierarchical level lv . Then, at the top of the hierarchy (i.e., $lv = 1$), the network is interpreted as a single unit (i.e., one fictitious node), which consists of all actual nodes v_i and all links between them: $V_1^{(1)} = \{v_1, v_2, \dots, v_n, E\}$, where E is the set of links of the graph $G(V, E)$. In the second level ($lv = 2$), the network is described by d fictitious nodes: $V_1^{(2)}, V_2^{(2)}, \dots, V_d^{(2)}$, with $V_1^{(2)}, V_2^{(2)}, \dots, V_d^{(2)}$ being a subset of $V_1^{(1)}$. Notice that the n actual nodes, contained in the fictitious node $V_1^{(1)}$ at the first level, are distributed among the d fictitious nodes at the second level.
- *Fictitious links*: A fictitious link $E_{ij}^{(lv)}$, which connects $V_i^{(lv)}$ and $V_j^{(lv)}$, describes how clusters at a hierarchical level lv are connected to each other, and is constructed as the parallel arrangement of all actual links

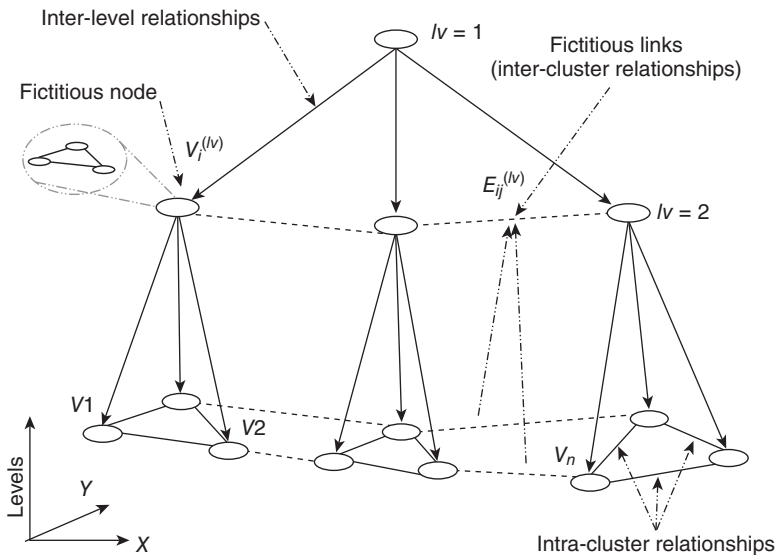
between each pair of fictitious nodes. At the bottom level ($lv = L$), both fictitious and real networks are the same, and so are nodes and links (Gómez *et al.*, 2011a).

- **Fictitious networks:** A fictitious network is defined in terms of fictitious nodes and links at each level. Let $\Lambda^{(lv)}$ be the set of fictitious nodes at level lv . Then, $\Lambda^{(1)} = V_1^{(1)} = \{v_1, v_2, \dots, v_n\}$ and $\Lambda^{(2)} = \{V_1^{(2)}, V_2^{(2)}, \dots, V_d^{(2)}\}$. Similarly, $E^{(lv)}$ is the set of fictitious links $E_{ij}^{(lv)}$ at level lv . The fictitious network at level lv describes a system at such level of abstraction and has an associated graph $G^{(lv)}(\Lambda^{(lv)}, E^{(lv)})$. The adjacency matrix $A^{(lv)}$ denotes whether fictitious nodes $V_i^{(lv)}$ and $V_j^{(lv)}$ are connected through a fictitious link in $E^{(lv)}$.

Each of the L levels in a hierarchy represents a complete description of the network. Each level is comprised of $k_{lv} > k_{lv-1}$ fictitious nodes and is a refinement of its immediately superior level (i.e., it is a partitioning of higher resolution). Note that $k_1 = 1$ and $k_L = n$; thus, the associated partitionings are trivial clusterings. The hierarchical representation of a network consists of a set of graphs $G^{(lv)}(\Lambda^{(lv)}, E^{(lv)})$ with $lv = 1, 2, \dots, L$, representing fictitious networks.

In addition to the definition of fictitious nodes and links, three types of relationships can be observed within the hierarchy. Figure 17.2 summarizes the hierarchical representation of infrastructure networks.

- **Inter-cluster relationships:** These are relationships among fictitious nodes, i.e., fictitious links.



17.2 Elements in the hierarchical model.

- *Intra-cluster relationships*: These are relationships among actual network nodes within a specific fictitious node, which are represented by the sub-graph $G[V_i^{(lv)}, E(V_i^{(lv)})]$ containing only the elements from the actual network that lie within $V_i^{(lv)}$.
- *Inter-level relationships*: These are relationships between fictitious nodes at adjacent levels of the hierarchy and denote how fictitious nodes (clusters) are embedded into others.

Each level in the hierarchy provides a different degree of detail. Therefore, a problem can be significantly simplified and emergent properties can be detected (Gómez *et al.*, 2011a); in fact, the system should not be taken to a level of detail beyond which the relevance of information is more valuable than its specificity. The main advantages of using fictitious networks are:

- Enhanced insight into emergent properties of the network at different levels of resolution, which is important for risk analysis and decision making (e.g., detecting critical elements).
- The computational burden of network analyses can be substantially reduced by balancing the specificity and relevance of information.

17.4 Risk assessment of infrastructure networks

Risk is related to the effect of uncertainty on the performance of a system of interest. Moreover, hazards to the system may lead to vulnerability when compromising critical elements. Risk analysis is essential for decision making: rational and structured analyses of available information provide key evidence for effective decisions. Furthermore, this type of analyses is closely tied to the socioeconomic background of involved individuals and organizations.

Risk assessment requires the identification and valuation of future scenarios; then, all results are estimates of probabilities of occurrence of events and their consequences. Managing uncertainty is vital in this process (Knight, 2009), which includes lack of knowledge about the problem and randomness. Even if all scenarios are known, the combinatorial problem would be excessively large and difficult to evaluate. Finally, risk assessment is essential for risk management tasks, such as risk mitigation/reduction. The following sections discuss the use of the proposed systems approach and hierarchical model for the following problems:

- reliability assessment;
- vulnerability analysis;
- damage propagation analysis.

17.4.1 Reliability assessment

Reliability analysis provides evidence that can be used to make decisions as to which actions should be taken by ensuring that network performance is kept above a certain acceptable level (Konak *et al.*, 2004; Murray *et al.*, 2007). Reliability is often calculated as the complement of the failure probability by evaluating a multivariate integral of probability density functions of relevant random variables over the failure space. Both probability functions and failure space are extremely difficult to obtain for most realistic networks. Therefore, an analytical solution is not available. In this section, the basic reliability problem for network systems is presented.

Let $\beta(X_j)$ be a measure of performance under state $X_j = \{y_1, \dots, y_n, z_1, \dots, z_m\}$; with $y_i = 1$ if node i is in operation (z for the case of edges) and 0 otherwise. Let Ω be the set of all possible scenarios (i.e., failure and full operation), then, the system reliability under state X_j is defined as the probability of the performance measure $\beta(X_j)$ exceeding a threshold Ψ , i.e., $P(\beta(X_j) > \Psi)$.

A comprehensive reliability analysis requires testing the network for every possible failure scenario. Define a scenario as the vector state of the system $X = \{x_1, x_2, \dots, x_n\}$ in which x_i corresponds to the state of element i (i.e., full operation or failure of either a node or link). Then, if the set of independent scenarios that causes the system to fail is Φ_F , the probability of failure P_f can be computed as:

$$P_f = \sum_{X_j \in \Phi_F} \left(\prod_{x_i \notin H_j} (1 - p_i) \prod_{x_i \in H_j} p_i \right) \quad [17.1]$$

where H_j is the set of x_i in X_j such that $x_i = 1$ (component survives), and p_i is the failure probability of the network element x_i . Note that the term in parentheses is the probability of occurrence of the j -th failure scenario. Equation 17.1 is generic and assumes that elements fail independently, which is not necessarily true in practical cases; more rigorous formulations including correlation can be found elsewhere (Song and Der Kiureghian (2003)).

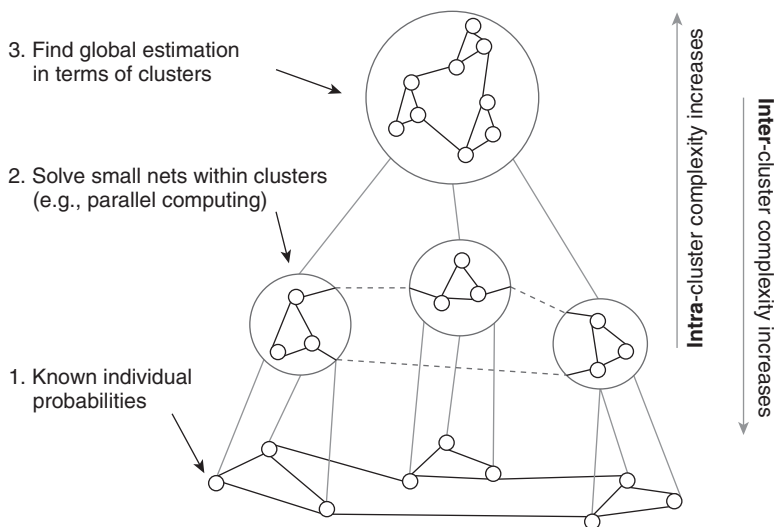
Computing network reliability is a difficult problem for which there are two basic approaches: exact or approximate methods. Exact methods can be further classified into those focused on the enumeration of all minimal paths or cut sets and those based on network partitioning. Some exact methods for computing reliability are the inclusion-exclusion formula and other enumeration methods (Bell and Ilida, 1997), optimization approaches (Ahuja *et al.*, 1993), network reduction methods (Todinov, 2007), and closed form recursive approaches for specific topologies (Duenas-Osorio and Rojo, 2011).

Simulation algorithms provide alternative methods. Crude Monte Carlo is a popular choice but too expensive for large networks, in particular, when failure probabilities are very small. Better estimates can be obtained from:

(i) block sampling techniques, such as the dagger sampling method (Kumamoto *et al.*, 1980); (ii) variance reduction methods like importance sampling, which can be further improved with random processes including Markov chains (Au, 2004), and (iii) statistical learning methods, such as neural networks or support vector machines (Hurtado, 2007). Finally, recent developments on subset simulation have become popular in reliability analysis, which facilitates the computation of reliability in high dimensions (Au and Beck, 2001) and assessment of seismic risk (Au and Beck, 2003).

The connectivity failure probability problem in a network can be done by estimating the probability that two nodes remain connected for all possible failure scenarios. The computational burden of this combinatorial problem (exponential on the number of elements) might be reduced by using fictitious networks as discussed in Gómez *et al.* (2011a). The idea is to solve the failure probability problem for each individual fictitious node and link at a level of abstraction. Then, the reliability problem for the fictitious network is solved. Figure 17.3 illustrates the general process for estimating reliability at fictitious networks. The selection of the level is an optimization problem that must take computational cost and relevance for decision-making (i.e., complexity/relevance ratio) into account. At higher levels, complexity results from solving big fictitious nodes, whereas at lower levels complexity results from solving the fictitious network.

Consider a fictitious network at level lv obtained after a hierarchical decomposition of a network. Then, the fictitious link $E_u^{(lv)}$ connecting fictitious vertices $V_i^{(lv)}$ and $V_j^{(lv)}$ is constituted by $N_u^{(lv)}$ actual links e_{pq} that connect



17.3 Methodology for hierarchical reliability assessment.

all actual vertices v_p in $V_i^{(lv)}$ and v_q in $V_j^{(lv)}$. Therefore, the probability of failure of a fictitious edge u at level lv is given by the product of individual failure probabilities of links e_{pq} since they comprise parallel arrangements. Once the fictitious node probabilities are estimated, global reliability calculations on the corresponding fictitious network can be made. At intermediate levels of description, the computational cost is still much lower than the cost of evaluating the entire network, even if network reliability computations are required at every fictitious node. The cost of such efficiencies is the loss in accuracy that results from assuming that elements within different clusters fail independently, which is not necessarily true.

17.4.2 Vulnerability analysis

A global measure of network vulnerability evaluates the change in performance when the network is exposed to a set of damaging scenarios $d_i \in D$ (D is the set of all possible failure scenarios) (Latora and Marchiori, 2005; Bell *et al.*, 2008; Schuchmann, 2010). Then, if $F(S)$ defines a performance measure of the system S , the relative drop in the system's performance for a given scenario d_i $V(S|d_i)$, can be computed as follows:

$$V(S|d_i) = \frac{F(S) - F(S|d_i)}{F(S)} \quad [17.2]$$

where $F(S|d_i)$ describes the system's performance measure given the occurrence of damaging scenario d_i . Because a comprehensive vulnerability analysis requires the assessment of all d_i in D , the overall vulnerability can be calculated as $V(S) = \max_i V(S|d_i)$ (Latora and Marchiori, 2005), which is computationally unfeasible for complex networks. Connectivity loss (or increase in shortest-paths) can be used in a similar way (Albert and Barabasi, 2002; Gong *et al.*, 2008; Barzel and Biham, 2009). Further discussion on available approaches to assess vulnerability can be found in Gómez *et al.* (2011b).

A network vulnerability analysis includes the following tasks: (1) system identification and characterization; (2) definition of evaluation criteria (e.g., form and strength); (3) identification of possible failure scenarios; (4) assessment of potential losses per scenario; and (5) evaluation of vulnerability indices.

The proposed analysis focuses on identifying critical links for connectivity (i.e., importance) while taking into consideration failure probability throughout the hierarchy. Fictitious nodes at high hierarchical levels consist of macroscopic network components which represent areas of major importance in a system, e.g., ports that should maintain connectivity with industrial areas. The clustering process maximizes density within clusters and

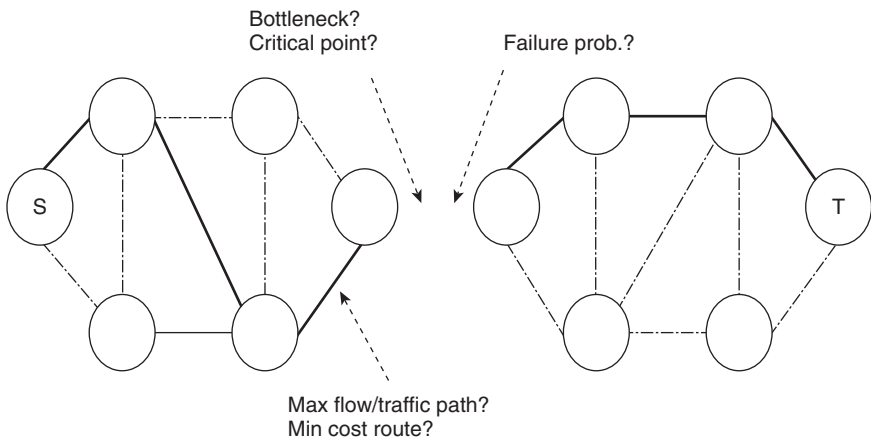
sparsity between clusters, then, low redundancy (i.e., few alternate paths) can be expected between fictitious nodes; thus, connectivity relies on a few elements of high betweenness. Figure 17.4 illustrates the fact that clustering can be used to detect bottlenecks and other critical elements.

The hierarchical model is used to make analyses more efficient by reducing the number of scenarios to be evaluated at the upper levels. Critical elements for the network performance are detected as emergent properties at different levels of abstraction, i.e., how individual elements participate in the connections between fictitious nodes at different levels of abstraction. Thus, in topological terms, links found in fictitious links at higher levels will be considered to have higher criticality. Figure 17.5 shows how the concept is applied throughout the hierarchy.

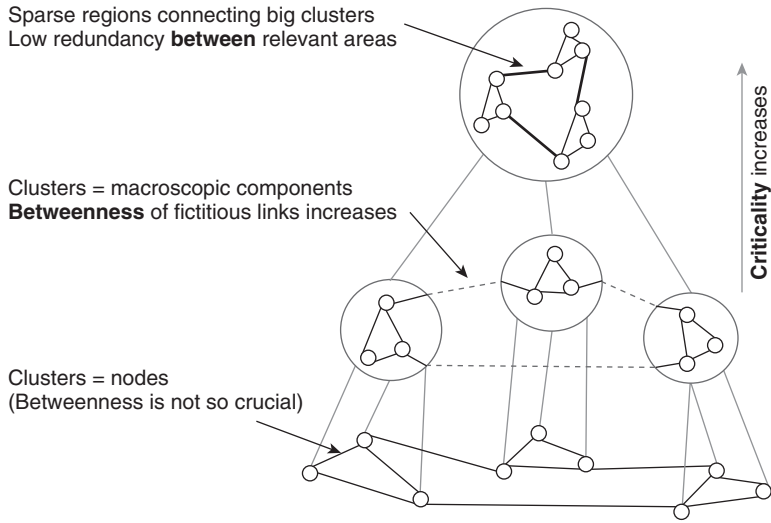
Two factors, $F_{\text{form}}(e_j)$ and $F_{\text{strength}}(e_j)$, are introduced to evaluate how the form (i.e., topological importance according to connectivity) and the strength (i.e., failure probability) contribute to the vulnerability of actual links e_j , respectively.

The impact of a damaging scenario is measured in terms of connectivity loss (or increase in shortest-path). The Dijkstra algorithm (Brandes and Erlebach, 2005) for finding shortest-paths within networks (before and after link removal) can be used to evaluate a measure of the impact of damage in the network. The factor $F_{\text{form}}(e_j)$ describes the importance of the edge j at level lv and is computed as:

$$F_{\text{form}}(e_j) = \sum_{s,t \in V} \frac{D^{(j)}(s,t) - D(s,t)}{D(s,t)} \quad [17.3]$$



17.4 Principle of maximum sparsity between clusters.



17.5 Overall strategy for vulnerability detection.

where $D(s, t)$ is the shortest path between nodes s and t , whilst $D^{(j)}(s, t)$ denotes the same index after removing link j .

The factor $F_{\text{strength}}(e_j)$ measures the contribution of an actual link to the failure probability of the fictitious link at level lv . Because fictitious links are parallel arrangements of actual links, $F_{\text{strength}}(e_j)$ can be computed as:

$$F_{\text{strength}}(e_j) = 1 - \frac{\prod_{i \in \Phi \setminus \{e_j\}} P_f(e_i) - \prod_{i \in \Phi} P_f(e_i)}{\prod_{i \in \Phi \setminus \{e_j\}} P_f(e_i)} \quad [17.4]$$

where Φ is the set of edges that make up the fictitious link, $\Phi \setminus e_j$ denotes the removal of link j and $P_f(e_i)$ is the failure probability of the i th actual link. The factor $F_{\text{strength}}(e_j)$ is larger for elements with higher failure probabilities. In summary, $F_{\text{strength}}(e_j)$ describes the contribution of the actual edge e_j to the probability of failure of the fictitious link to which it belongs.

The proposed vulnerability analysis focuses on identifying critical links at different levels. An index that evaluates the contribution of an edge to the system vulnerability can be defined as:

$$V^{(lv)}(e_j) = I^{(lv)}[e_j] \cdot c_{lv} \cdot [F_{\text{form}}(e_j) \cdot F_{\text{strength}}(e_j)] \quad [17.5]$$

where $I^{(lv)}[e_j]$ is an indicator function that denotes whether the actual node e_j belongs to a fictitious link at level lv . Note that only edges that belong to fictitious links will have an impact on the network vulnerability. The weighting factor c_{lv} is a level-dependent coefficient based on global descriptors of

the network, and may depend on different factors. The product in Equation 17.5 provides a qualitative metric that describes the relative importance of network components based on the proportional contribution of the factors. All factors in Equation 17.5 have been selected so that they are independent and under the assumption that they fully describe the vulnerability space under uncertain decisions. Three possible indices are proposed to build a coefficient that assigns higher weights to higher levels:

$$c_{lv} = \left(\frac{L - lv + 1}{L} \right) \left(\frac{n|\Lambda^{(lv)}| + 1}{n} \right) \left(\frac{1}{K^{(lv)}} \right) \quad [17.6]$$

where L denotes the total number of hierarchy levels, n is the total number of network nodes; $|\Lambda^{(lv)}|$ is the cardinality of the set of fictitious nodes at level lv ; and $K^{(lv)}$ is the k -connectivity of the fictitious network at level lv (Brandes and Erlebach, 2005). The first factor accounts for the fact that more complex subsystems rely on fewer actual links as moving up in the hierarchy; other weighting functions can be used depending upon the problem. The second factor defines a degree of clustering at level lv ; i.e., the ratio of fictitious nodes with respect to the actual number nodes. Finally, the third factor takes into consideration the k -connectivity of fictitious networks at every level. Note that all three factors are defined within the interval $[0, 1]$ and decrease as the network description becomes closer to the actual network.

The proposed index $V^{(lv)}(e_j)$ can be used for ranking the edges according to their relative contribution to the vulnerability at each hierarchical level. The final result is a ranking of links with decreasing contribution to vulnerability (either globally or by level). Thus a global index for a link can be computed as: $\max_{lv} V^{(lv)}(e_j)$. The proposed approach detects critical links as those with high betweenness (which belong to many shortest-paths, with low redundancy) and high failure probability. This value provides valuable information for decision making and risk management, e.g., resource allocation for maintenance, replacement, etc.

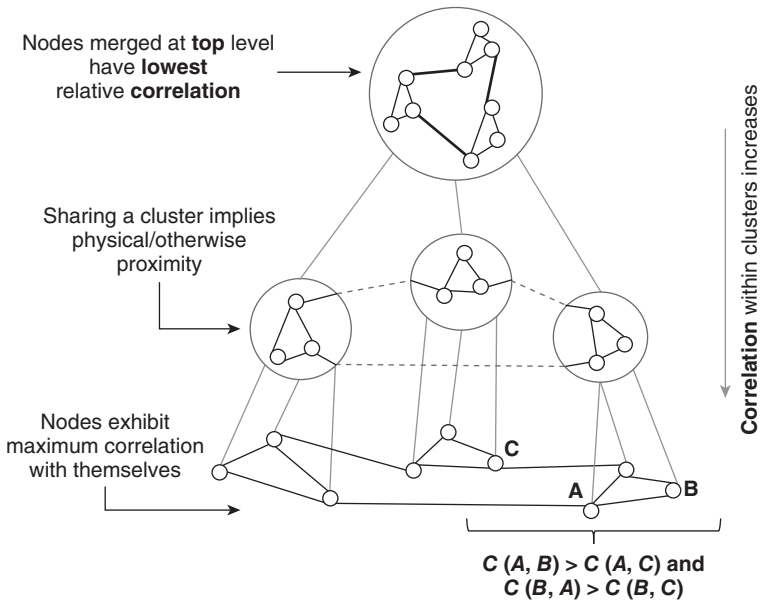
17.4.3 Damage propagation analysis

Damage propagation is critical for the design, analysis and operation of infrastructure systems. Networks are comprised of connected/related elements, increasing the likelihood that elements are affected by others; e.g., due to physical proximity to a source of damage (e.g., earthquake fault), or to redistribution of flow after the removal of elements. Damage propagation depends on a certain level of correlation among network elements. The proposed hierarchical model is constructed by grouping elements at different levels of abstraction. The hierarchy itself can be understood as a measure

of correlation obtained by means of computational intelligence. Then, elements that are merged into small fictitious nodes (lower hierarchy levels) are more correlated than those merged into larger fictitious nodes (higher hierarchy levels). Correlation does not necessarily imply causality, but in this context, it allows to estimate overall effects of local events, which is useful as an input to recovery planning. Figure 17.6 illustrates this idea.

The hierarchy-based correlation requires constructing a matrix C such that its ij th component denotes the relationship between elements i and j according to the hierarchical representation of the network. The matrix C can be used to estimate the extent of the impact of a failure scenario on every node as $I = C * e$, where the correlation matrix C serves as a mapping from a direct damage vector (failure scenario) e containing nodes affected by an event, onto a vector I that denotes how every other node in the network is affected. Therefore, I can be understood as an impact index on elements due to a damaging event and its propagation.

There are many ways to construct the matrix C . For instance, correlation functions may include functional relationships of the network; Barzel and Biham (2009) proposed topological and functional criteria to assess correlation. In general terms, the correlation matrix based on hierarchical description of the system can be obtained as $C_{ij} = h(lv, \Theta)$, where h is a function of the level lv in the hierarchy at which the decisions will be made and other



17.6 Hierarchy-based correlation function.

network attributes Θ . The function h has its maximum at the lowest level in the hierarchy and decays as the level moves up. Although decay rules are case-specific, polynomial and linear decay shapes seem to be reasonable. One practical way to define the correlation matrix is:

$$C_{ij} = 1 - \frac{D(v_i, v_j)}{\max_{pq} [D(v_p, v_q)]}; \text{ with } p, q = 1 \dots |\mathbf{V}_w^{(lv)}| \forall i, j \in [1, n] \quad [17.7]$$

where $\mathbf{V}_w^{(lv)}$ denotes the first cluster (bottom-up) at which the elements i and j of the original network intersect (i.e., share the same cluster). Note that the level lv is considered indirectly through the cardinality of cluster $|\mathbf{V}_w^{(lv)}|$, which is larger for smaller lv .

17.5 Optimal resource allocation in infrastructure networks

Engineering decision making involves a technical domain, which is concerned with actual system performance, and a socioeconomic domain that accounts for interests, circumstances and consequences for stakeholders. Questions such as: who makes the decision? for whom? and in what context? need to be clearly defined. Decision-making is the process of evidence interpretation in order to make a decision that maximizes the expected outcome (e.g., performance, utility, well-being). Although many methods exist for supporting decision making, the final result depends on the completeness and quality of the evidence. Different approaches to optimization are considered as the means to achieve the objectives established by a rational decision-maker. The evidence for making decisions on infrastructure networks can be provided and enhanced by the hierarchical model.

17.5.1 Optimization-based decision-making

Optimization methods seek to find variable values that optimize a multi-variate objective function under a set of constraints. Constraints define a search space, also known as feasible region within which the solution must be enclosed. Among optimization methods (Luenberger, 2003), linear programming is widely used because of its ease for implementation and because of its greater stability and convergence in comparison with other methods (e.g., nonlinear gradient methods).

For the case of large-scale problems, such as resource allocation at a global scale, a variety of techniques have been developed based on either exact or heuristic methods. The column generation method (Gondzio and Sarkissian, 1997) and Dantzig–Wolfe decomposition (Vanderbeck and Savelsbergh, 2006) are useful exact approaches for problems with a large

number of variables or constraints, respectively. There also exist powerful heuristics for many different types of problems, within which evolutionary and other bio-inspired techniques have gained momentum in the last decade (Reeves, 1993; Bonabeau *et al.*, 1999; Lee and El-Sharkawi, 2008). In particular, Chang (2010) has used evolutionary algorithms for global level optimization of supply chains.

Most practical problems have several (usually) conflicting objectives. Multi-objective optimization (Coello *et al.*, 2006) deals with such kinds of problems by finding a Pareto front, which defines a region in which neither of the objectives can be improved without degrading another. An alternative option is to fix one of the objectives as a constraint, i.e., attend at least P people or spend at most D dollars and then perform sensitivity analyses on P and/or D to unravel and evaluate tradeoffs. This way, stakeholders get to know a set of equivalent choices and their consequences on the objective.

Researchers have defined several archetype problems that encapsulate common features that appear in most applications (e.g., the travelling salesman problem). The capacitated facility location problem (CFLP; Levi *et al.*, 2012) is a specific case, in which a set of facilities/service-providers is to be installed to satisfy the demand from a set of users/clients, under a restricted capacity. The basic assignment problem is proved to be NP-hard (Khan, 2003), and the decision about the number of alternatives to assign is highly combinatorial. Therefore, formal methods of optimization may not suffice for practical applications.

17.5.2 Hierarchy-based optimization

The hierarchical model provides information about cluster structures of a network at several levels of abstraction. Nodes within these clusters are characterized for being relatively close to each other, and having high connectivity with respect to the whole network. Additionally, cluster centroids can be thought of as hubs, with enhanced centrality and connectivity within their corresponding clusters. This suggests that resources can be assigned to fictitious nodes instead of actual nodes, reducing complexity. Then, if a certain resource is to be allocated at a fictitious node, it can be located at the centroid, which can provide the whole cluster with access to the service more effectively than any other node in the cluster. In this sense, a variety of resources of different characteristics (cost, capacity, and accessibility) can be assigned coherently with the multi-scale intrinsic structure of the network.

This strategy seeks to simplify the optimization process of resource allocation. This is achieved by using the hierarchical model instead of the original network. The underlying principle is that a more complete data structure simplifies the algorithm due to a pre-processing of the network topology.

In the case under consideration, an optimal supply network can be designed, by considering: (1) whether a facility of a specified type h is located at each node i (hard capacity); (2) the capacity of a facility is neither fixed nor free, in contrast, there is a discrete set of alternatives with different characteristics that can be installed at any node in order to optimally satisfy the demand; and (3) the total number of facilities is not fixed, so it might take values between 1 (i.e., totally centralized) and the number of nodes n (i.e., totally decentralized) for each alternative according to feasibility and optimality. With the exposed conditions, the basic assignment problem has proved to be NP-hard (Khan, 2003) because the number of possible solutions is highly combinatorial. Therefore, crude exact optimization methods may not suffice for practical applications.

It is proposed that the hierarchical representation of networks enhances the approach to the optimum resource allocation problem: first, it achieves the quality and suitability of the solution to the network's topology; and second, it improves the computational efficiency of the optimization program. The optimization problem is defined in terms of an objective function and a set of constraints. The hierarchical model is incorporated in two forms:

- The objective function is formulated in terms of fictitious nodes rather than actual nodes. Centroids are used to map the obtained solution to the actual network. This condition alone reduces the amount of decision variables (i.e., only nodes that are centroids are candidates). The trivial partitionings (i.e., the top and bottom levels) are not considered, because they only produce trivial solutions that increase the computational complexity without benefits.
- The hierarchical description of the network reduces the space of feasible solutions by imposing additional constraints that force the solution to fit the multi-level clustering structure of the network. Thus, special parameters and functions that encapsulate relevant information from the hierarchy need to be introduced to the optimization problem. In this sense, the mathematical model not only takes into account conventional technical/physical constraints but also considers the system's internal structure, given by the centroids and fictitious elements.

The objective function takes into consideration the costs associated with both construction of facilities and users' cost to access the service; the latter is measured in terms of distance from individual users to its corresponding facility. Two sets of constraints (operational and hierarchical ones) limit the solution space. The formulation of the objective function is then stated as a cost minimization problem as:

$$\min \left(\sum_{i=1}^M Cb_i + \sum_{j=1}^n Ca_j \right) \quad [17.8]$$

subject to:

$$\Gamma = \{\gamma_g \in \mathbf{R}^n : \gamma_g = f[G(V, E), \Theta]\} \quad [17.9]$$

$$\kappa = \{\kappa_k \in \mathbf{R}^n : \kappa_k = f[T(G), R(G)]\} \quad [17.10]$$

where Cb_i is the cost of the i th facility (out of M facilities) and Ca_j is the cost for the j th user (out of n users); and Γ and κ are the sets of operational and hierarchical constraints, respectively. The first set is associated with issues, such as specific costs, and maximum allowed distances and capacities, whereas the second one deals with topological features, such as centroid mapping and conditions within the hierarchy. For this reason, the constraints γ_g are (linear) functions of the graph itself (G) and a set of attributes Θ (e.g., costs, distances, capacities), whilst the constraints κ_k depend on the tree representing the hierarchical structure of the graph ($T(G)$) and the distribution of centroids ($R(G)$). Notice that an essential constraint is that users are assigned only to installed support centres. See Gómez *et al.* (2011d) for details on the hierarchical constraints.

17.5.3 Illustrative example

Consider a variation of the CFLP. The objective is to provide a support network for victims after a major earthquake disaster. A transportation network with 64 nodes (urban regions) and a diameter of approximately 1500 km is generated by using the Delaunay triangulation algorithm (Khanban *et al.*, 2002). The population associated with every node was assigned based on a random function. Earthquake scenarios (i.e., assistance demand) are generated randomly.

The aim of the model is to minimize the cost of installing support centres throughout the network such that the demand is satisfied at every node. Additionally, the designed support network must cover a percentage λ (e.g., 20%) of the population, even if the demand is below that level; this is enforced so that a minimum level of assistance is guaranteed everywhere for the sake of robustness.

Then, a solution is designed for a specific disaster, whose performance is measured in terms of the assistance deficit when faced with new disasters; assistance deficit is computed as the difference between the demand (population affected) and the availability of services provided (i.e., available capacity of the support designed network).

The decision-making problem focuses on allocating two types of centres with different capacities: type-1 centres have greater capacity than type-2 centres. Additionally, in order to complete the support network, type-2 centres must be connected to a type-1 centre within a distance d_1 , and users must be connected to at least one centre within a distance d_2 ; these distances

are fixed around 350 km so that large clusters are not neglected due to distance constraints.

The cost is divided into the cost of building and operating support centres, and the user's cost for accessing the service; the latter is quantified proportionally to the distance travelled by a person and reflects both the actual transportation cost and the impact of travelling (i.e., time delay for the service). Therefore, the objective function from Equation 17.8 becomes:

$$\min \sum_{h \in H} \sum_{q \in Q} c_h x_{hq} + p_a \sum_{i \in V} \sum_{j \in V} w_{ij} d_{ij} \quad [17.11]$$

where c_h is the cost of installing a support centre of type h ; p_a denotes the number of people to be accommodated; and d_{ij} is the distance between support centres. The terms in Equation 17.11 account for the aforementioned parts of the cost:

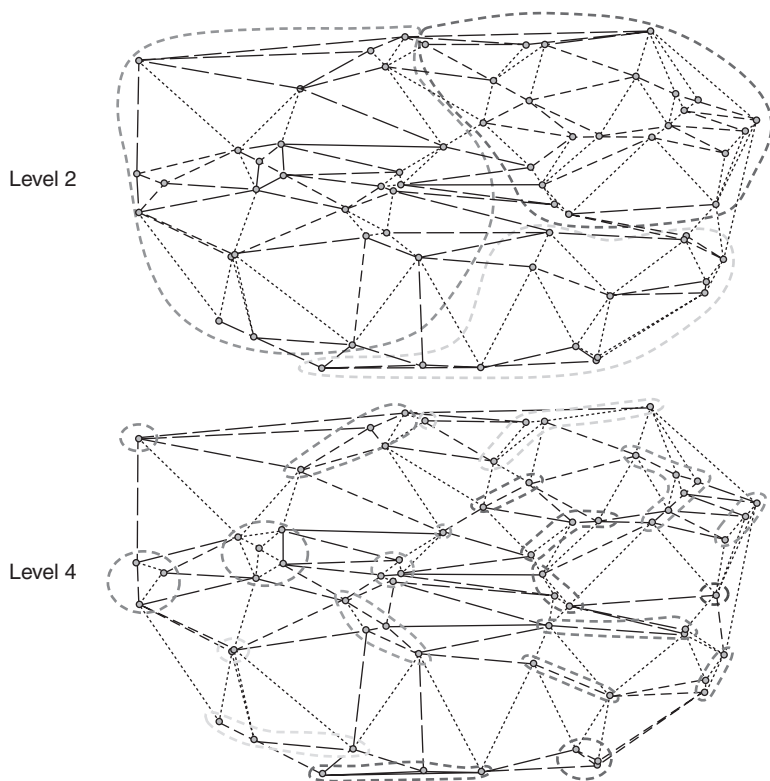
- Installation/operation cost: This is related to the decision variable x_{hq} that denotes whether alternative h is placed at cluster (fictitious node) q .
- User-access cost: This is linked to the decision variable w_{ij} that represents whether user i is attended at a centre at node j .

A recursive clustering process is carried out in order to find the intrinsic structure of the network. The algorithm leads to a five-level hierarchy. The second and fourth levels of the hierarchical decomposition of the proposed network are shown in Fig. 17.7.

A robustness-oriented simulation is carried out, aiming to estimate the performance of the designed network due to unforeseen damaging events. For this purpose, a set of S disasters are simulated and then S solutions for the resource allocation are obtained by running the optimization algorithm. For each solution, P additional disastrous events are generated independently and the assistance deficit (AD) of the network is computed as the amount of people unattended for each event. Monte Carlo simulation was used; however, more powerful alternatives can be used. Figure 17.8 shows the strategy for generating disastrous events, as well as the obtained solutions with a heuristic variation of the linear programming and the hierarchical approach.

Qualitatively, the solution for the proposed approach exhibits greater decentralization; i.e., the clustering structure of the network is reflected and groups with different scales appear in the solution according to optimality. This fact is important, because it enhances properties, such as robustness and scalability in the solution.

The formulation of the linear program with the proposed approach helps reduce the computational burden, i.e., fewer decision variables are considered by using only the centroids as candidate solutions. Moreover,

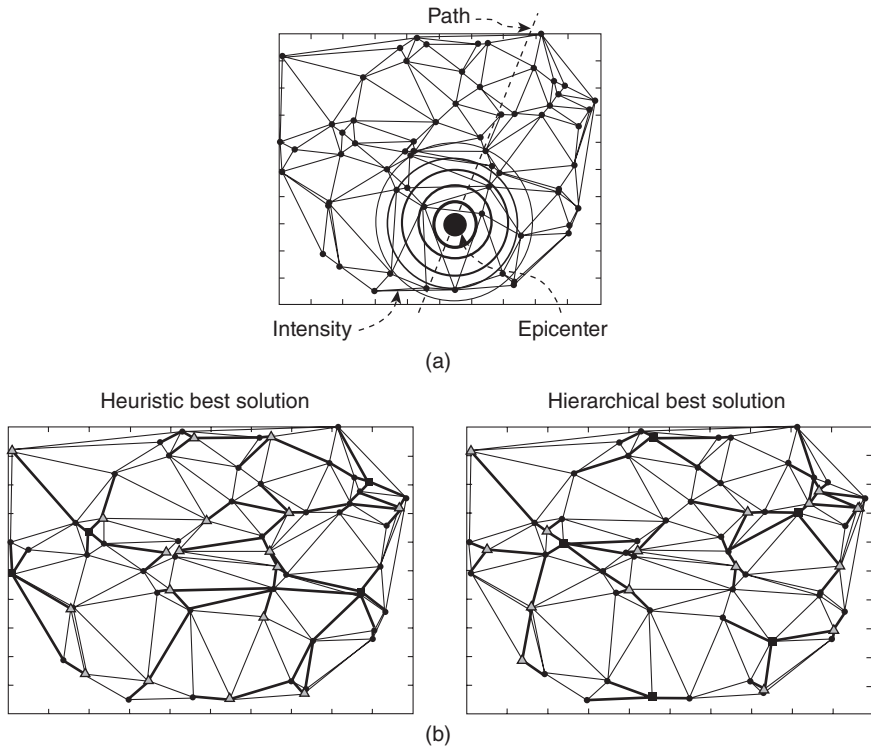


17.7 Second and fourth levels of the hierarchical decomposition of the proposed network.

the additional hierarchical information leads to a much narrower solution space.

17.6 Conclusions

This chapter presents the main advances and challenges in modelling and assessing performance of infrastructure networks in the context of risk-based decision making. A systems approach was introduced to deal with the computational complexity of managing infrastructure systems. The proposed approach successively decomposes networks into clusters and constructs a hierarchical model of the network, extracting relevant properties of their topology that are useful for simplifying assessment and optimization of infrastructure networks. The hierarchical model responds to the organizational scenario of decision-making problems in managing infrastructure, i.e., people making decisions at different levels, about different



17.8 (a) Strategy for generation of disasters and (b) designed network with the heuristic (left) and hierarchical (right) approaches.

specific problems, and with different resources. In this sense, the proposed framework provides a model for decision making with an appropriate level of decentralization as well as useful tools for assessment performance (e.g., reliability, vulnerability, and damage propagation) and decision support (e.g., optimization) in infrastructure networks. This area has great potential for further development; in particular, bridging technical considerations and other contextual aspects of decisions is challenging and is worthy of future investigations.

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Seismic vulnerability analysis of a complex interconnected civil infrastructure

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Abstract: This chapter presents a computational framework for probabilistic seismic performance assessment of the network of interdependent infrastructural systems that supports the functions of our society. An increasing number of large-scale failures (effects), initiated by relatively small natural or man-made disturbances (causes), have brought unforeseen disruption to infrastructures in recent years. The disproportion between causes and effects highlights an emergent chaotic behaviour, traceable in the infrastructure complexity and level of interconnectedness. In order to quantify this complexity and emulate the behaviour of the infrastructure as it emerges from that of its components and their interaction, an object-oriented model is developed.

Key words: object-oriented modelling, system of systems, uncertainty modelling, inter- and intra-dependencies.

18.1 Introduction and definitions

This chapter presents a computational framework for the performance assessment of an infrastructure. Though the focus is on its performance in the event of an earthquake, the framework can be generalised and extended for multiple hazards. The infrastructure can be defined as a network of distinct man-made systems and processes that function collaboratively and synergistically to produce and distribute a continuous flow of essential goods and services (PCCIP, 1997).

From a system-theoretic point of view, the infrastructure is a system of systems (SOS) (Rinaldi, 2004), a super-system containing all other systems (buildings, lifelines, critical facilities, etc.) and constitutes the physical layer supporting the life of our society. In this sense, its reliable operation is essential to our safety, health and all socio-economic activities. This is the reason why it is commonly termed the critical infrastructure (CI) (PCCIP, 1997; Rinaldi *et al.*, 2001; Macaulay, 2008).

Such continuous reliable operation has been taken for granted by most people (Macaulay, 2008). However, the unforeseen disruption brought by an increasing number of large-scale failures, initiated by natural or

man-made disturbances, shows contrary evidence. Just to mention an example close to the authors, on 28 September 2003, 56 million people in Italy experienced the most serious blackout in 70 years, which lasted almost 12 hours. It was caused by storm-induced damage to a single line supplying power from Switzerland to Italy. According to reports, the blackout spread to the entire country in less than 4 seconds.

One distinguishing feature of these failures is that in many cases the size of the triggering event was minor, certainly not commensurate with that of the consequences. This disproportion between causes and effects highlights an emergent chaotic behaviour of the infrastructure. The causes of this behaviour are due to the complexity and level of interconnectedness that characterise the SOS: the sheer number of different systems and of different components within them (themselves in turn, being quite often part of a complex system), as well as the dependencies among them, inflate the possible states to large numbers, and pose a challenge to modelling.

It can be argued whether trying to make quantitative predictions on the performance of the infrastructure makes any sense. As Macaulay (2008) put it, 'if [infrastructure] analysis reaches the level of forecasting accuracy obtained by weather forecasting, we might be doing well!' Taking this optimistic view, the following sections will outline a general object-oriented framework for the modelling and simulation of an infrastructure. Before introducing the model, the next two sections define its scope and goal within their possible ranges, and the type of model, analysis, and interactions, respectively.

18.2 Time, space and stakeholder dimensions of the problem

The assessment of the capability of a system to reliably carry out its function(s) under normal as well as disturbed operating conditions is termed its performance assessment. This task requires a model and metrics defined on its response. With specific reference to the infrastructure, the formulations of the models as well as metrics are not unique and depend at least upon the spatial extent of the study region, the time horizon of the observation window, the purpose of the assessment and its end-user(s).

As far as the time dimension is concerned, three aspects are of interest: the first two are associated with the analysis time-frame and the analyst's position on the time axis. With reference to the time-frame, typically, three situations are considered:

- Short term (hours, days, weeks): in the immediate aftermath of the event the damaged infrastructure operates in a *state of emergency*.

- Mid-term (weeks, months): the infrastructure progressively returns to a new, different state of normal functionality (*recovery*).
- Long term (months, years): the infrastructure is possibly upgraded/retrofitted with available resources to mitigate the risk from the next event (*reconstruction*).

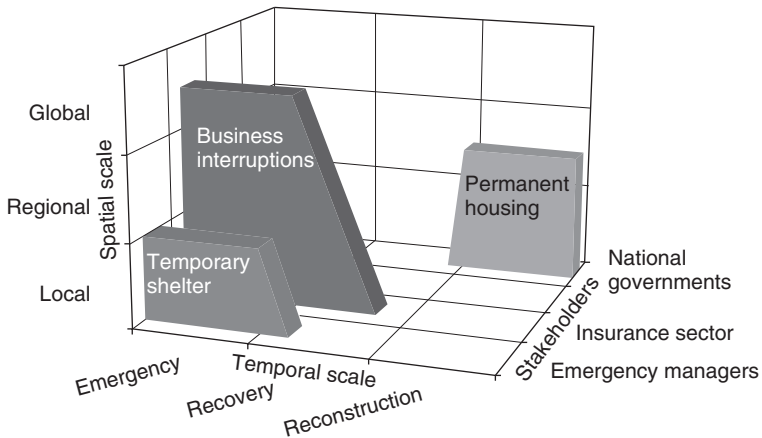
As far as the analyst's position on the time axis relative to the time-frame is concerned, again three cases are considered.

- Before the time-frame: the goal is *forecasting the impact* in order to prepare mitigation measures.
- Within the time-frame: the goal is *providing managers with a real-time decision support system* (DSS), which updates the infrastructure state based on the continuously incoming flow of information.
- After the time-frame: the goal is *validating models* against past events.

It is apparent how the goal of the analysis is strongly related to the analyst position. It can also be observed how the second case is certainly more challenging with respect to the first and third, where the information basis can be considered as stationary.

The third and final time-related aspect has to do with the dynamic nature of the infrastructure, i.e. with the fact that the impact of a disturbance evolves with the elapsed time since the occurrence of a disturbance. Hence, with respect to time, one important choice is whether the model is a *dynamic* or a *static* one. In a *static* model things are not really static but, rather, they are stationary, meaning for instance that flow equations are formulated for stationary conditions. This modelling choice of course has the potential to impact the way different components within each system, as well as different systems, interact, and, ultimately, how damage consequences propagate within the infrastructure. Intermediate strategies are possible. For instance, systems can be basically modelled as static (stationary flows), but consideration of the different *characteristic time* of each system is used to remove the assumption of instantaneous damage propagation. A similar approach is explored by Hernandez-Fajardo and Dueñas-Orsorio (2011).

The spatial dimension of the problem is, as already mentioned, a further important factor in the analysis, which is not independent of the described time-related factors. Indeed, the spatial dimension evolves with time. For instance, the spatial extent of interest is initially limited mainly to the local strongly affected area (short term), and is then expanded to include later an increasingly wide area covering adjacent regions, up to the national or global scale (in the economic recovery phase) to contract again to the national/local scales (in the reconstruction phase, and when long-term risk mitigation actions are undertaken). In some cases, however, this widening of scope can happen rapidly due to the propagation of the effects through



18.1 The three dimensions in an infrastructure vulnerability study.

the systems (e.g. cascading failures). Finally, it can be observed how different stakeholders/end-users have an interest in each different combination of time-frames and space-scales. These aspects are all illustrated in Fig. 18.1.

One distinguishing feature of available studies is that, typically, they address either the emergency phase (short-term, urban/regional scale, emergency managers; e.g. Franchin *et al.*, 2006b), or the economic recovery phase (medium to long-term, regional/national scale, politicians/planners/re-insurance industry; e.g. Karaca, 2005). The model described in the following focuses on pre-event (before the time-frame) performance assessment for the immediate aftermath of the event (short term). The target beneficiaries are emergency managers. The developed model facilitates the forecasting of the expected impact, and thus is valuable for planning and implementing risk mitigation measures at the urban/regional scale.

18.3 Model, analysis type and interactions

From a spatial/topological point of view the component systems of the infrastructure can be classified into three categories:

- Point-like (critical facilities): single-site facilities (hospitals, power-plants) whose importance for the functionality of the infrastructure justifies detailed description and analysis of their functionality and vulnerability.
- Line-like (utilities, lifelines): distributed systems comprising a number of vulnerable point-like sub-systems in their nodes, as well as vulnerable segments, and characterised by their flow-transmission function.

- Area-like (buildings): a special category specifically intended to model large portfolios of residential, office and commercial buildings (making up the largest proportion of the built environment). For the most common analysis resolutions, due to computational and information constraints, they cannot be modelled individually.

While, quite obviously, systems belonging to the second category can be described as networks or graphs, the remaining systems are also amenable to such a description. Further, systems exhibit dependencies that can also be modelled as edges (often called dependency edges or interface links) of a larger super-graph, connecting nodes belonging to different component systems. Network or graph theory is therefore a fundamental layer upon which the model is founded.

Nonetheless, graph theory can only go so far as the connectivity or integrity of the network is analysed, while the aim herein is to analyse functionality/serviceability in addition to connectivity and network integrity. Graph theory thus needs to be complemented. The second building block of the model is object-oriented modelling (OOM). Under this modelling paradigm, the SOS is described as a set of interacting objects, characterised in terms of attributes and methods. Objects are instances (concrete realisations) of classes (abstract templates for all objects with the same set of properties and methods) (Booch *et al.*, 2007).

One of the major advantages of OOM is the possibility of including physical laws and hence simulating the functioning of the system beyond its connectivity. The second advantage is that it allows the behaviour of the infrastructure to be emulated as it emerges from that of the individual objects and their interaction. The third is that the atomisation of the system into parts allows dealing with the complexity of the infrastructure. As far as the dependencies are concerned, Rinaldi *et al.* (2001) identified four general types: physical, cyber, logical and geographic/geo-spatial. Of these four categories, it has been argued that the functional and geographic ones are important (Johansson and Hassel, 2011).

The terms inter-dependence and intra-dependence are sometimes used to indicate the relation between components belonging to distinct systems or to the same system, respectively. In the following, dependence or inter-dependence are interchangeably employed to indicate the former, which in principle requires ad-hoc modelling with dedicated *dependency edges*, while the *within-system* relations, normally of physical/functional type, are already included within each system model in the form of components. Their presence is reflected in the formulation of the flow equations and their effect emerges in the corresponding solution. It should be noted that the dependency edges can be either uni- or bi-directional with respect to the dependence. A super-graph is always a directed one.

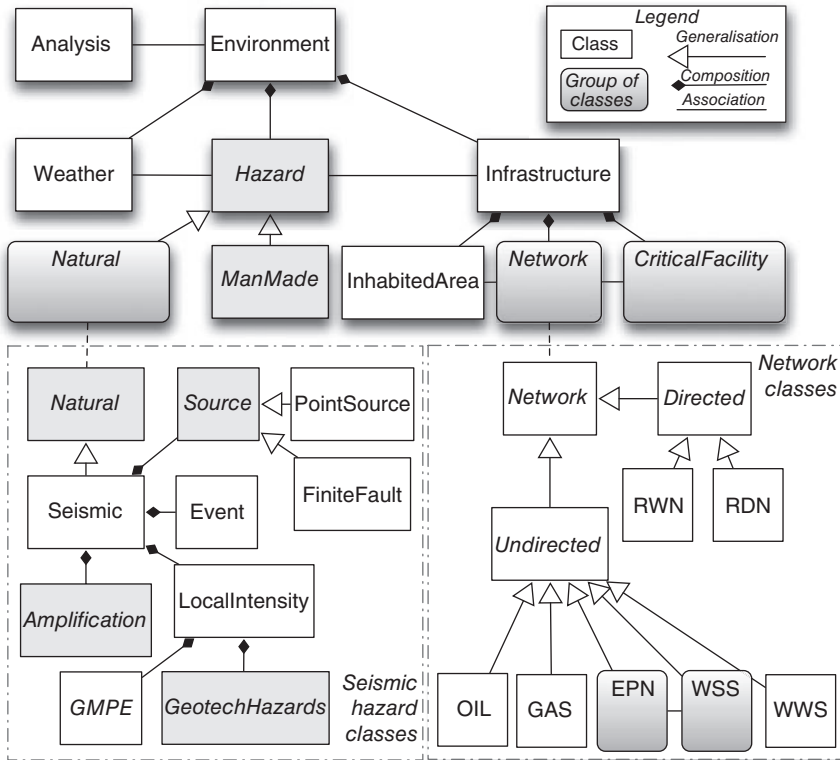
The final result is aimed to be a fairly comprehensive model that ideally includes all component systems and the relations between their components, as well as the capability to perform their functional analysis. Such a model has been termed a hybrid model (Nan and Kröger, 2011), and could be implemented in a distributed architecture with domain-specific simulators (in charge of performing the disciplinary functional analysis of each system) contained in an application layer. A communication or integration layer manages interactions between the simulators. The results shown later with reference to a simple example have been implemented and simulated using MATLAB® (The Mathworks, 2010) environment. The systems added are buildings, electric power networks (EPN), water supply systems (WSS) and road networks (RDN); further systems will be added. Only physical/functional and geographic dependencies are considered.

18.4 Object-oriented model (OOM) of the infrastructure and hazards

Figure 18.2 illustrates the class diagram describing the main classes defined to assess vulnerability of an Infrastructure subjected to multiple man-made and natural hazards. Only class names, without their attributes and methods, are shown for simplicity. Readers not familiar with the Unified Modelling Language (UML) graphical notation should consult Booch *et al.* (2007). In this, as well as in the following Fig. 18.5, classes are represented by rectangles (and classes grouped together for clarity, are shown with rectangles with rounded vertices) and lines connecting rectangles indicate a relationship between classes. Three types of relationships are used: the *generalisation* which means that a class is the generalisation of those below, i.e. an abstraction of their common features (and the class is called abstract), the *composition* which means that the class is made up of component classes, and the *association* which means that classes can call each other's methods.

As shown in the top portion of the figure, an *Analysis* is carried out within the *Environment* that is where *Hazards* act upon the *Infrastructure*, under certain *Weather* conditions. The *Infrastructure* class is the composition of three classes, the two abstract classes *Network* (line-like systems) and *CriticalFacility* (point-like systems), and the *InhabitedArea* class (area-like systems).

Networks can be either directed or undirected, and hence the *Network* class is composed of two abstract sub-classes *Directed* and *Undirected*. The former includes the Road Network (*RDN*) and the Railway Network (*RWN*), while the latter includes the Electric Power Network (*EPN*), Oil



18.2 The high-level class diagram of the OO model. In accordance with the UML standard, the names of abstract classes are in italic.

and Gas transmission and distribution networks (*OIL*, *GAS*), Water Supply System (*WSS*), Waste Water System (*WWS*).

The implemented *EPN*, *WSS* and *RDN* classes allow capacitive (flow) analysis on the respective networks (Vanzi, 1996; Wang *et al.*, 2010; see also Chapter 24). They include fragility models for vulnerable components (pipes, pumps, transmission and distribution stations, etc.) and direct physical interaction to model cascading failures (Dueñas-Orsorio and Vemur, 2009). The *Seismic* class and sub-classes implement a distributed seismic hazard model that accounts for the spatial correlation among intensity measures (IMs) at different sites (Jayaram and Baker, 2009) (see also Chapter 3), as well as for the probabilistic dependence between different intensity measures at the same site (Baker and Cornell, 2006; Bazzurro and Cornell, 2002). Further, the model can predict the full spectrum of IMs needed in a vulnerability study of an infrastructure, from ground shaking to geotechnical hazards (see Section 18.5.4).

18.4.1 Modelling interactions

As shown in Fig. 18.2, a number of association relationships between the component classes and sub-classes of the infrastructure describe the interactions between distinct systems. Among all possible interactions, the figure shows only those that have been modelled so far. For instance, associations exist among the *InhabitedArea*, *Network* and *CriticalFacility* classes, but also between the *WSS* and *EPN* classes. The former associations model interactions such as the evaluation of water, power and traffic demands starting from population and activities in the urban area, or the determination of accessibility of damaged buildings or hospitals for the purpose of emergency rescue. The latter are better described in Section 18.5.2.

In a more detailed, lower-level class diagram, such as that in Fig. 18.5, the above associations, modelling *between-system* interactions, are shown between classes corresponding to the actual components that interact with each other (e.g. interaction between a water pump and an electric substation through power feeding). At this lower-level *within-system* interactions, i.e. the interaction between elements within the same system, are also visible through association relationships amongst the sub-classes of each system.

As already discussed in Section 18.3, dependencies should be modelled with dedicated dependency edges in a super-graph. Correspondingly, a class should be defined for these objects. The attributes of these edges would be two pointers to the objects they connect and either one or two parameters (in case of a bi-directional relation) representing the strength of the dependence that can be either deterministic or probabilistic. In the latter case, the parameter is the probability that the dependent component reacts to an action of the controlling one (Dueñas-Osorio *et al.*, 2007). From a computational standpoint, it is straightforward and much more convenient to model these edges collectively, rather to include the pointer and dependence-strength parameter as attributes of the dependent object. This is the reason why dependency edges do not appear in Fig. 18.2 as a separate class but, rather, as associations between existing classes.

18.5 Description of the main classes

18.5.1 The 'InhabitedArea' class

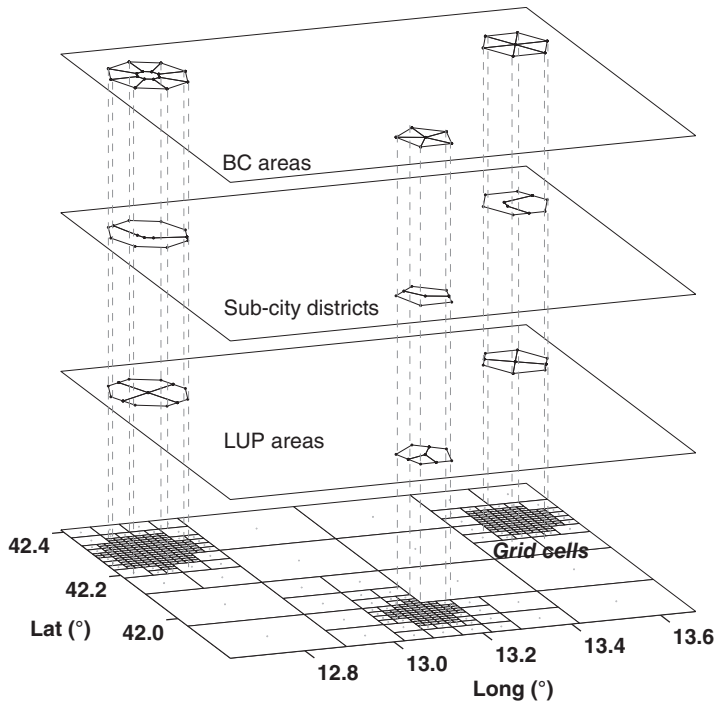
Buildings, residential, commercial or industrial, make up the largest proportion of the built environment and are the places where we live, work, consume goods and services. Buildings and the people within them are the reason for the existence of all other systems. The developed model on one hand recognises this central role of buildings, while on the other hand, it

compromises by adopting a statistical approach in their description. The very large number of buildings in a study of regional but even urban scale, prevents in all but exceptional cases a detailed individual analysis of all buildings. Hence, buildings are modelled as 'groups' (of size depending on the availability of data and on the refinement of the analysis, and varying from a single block to a larger extent of the urban territory) with information supplied at the group level, in terms of percentage of each building typology within the group, with associated fragility models, and socio-economic characteristics, such as population, employment, income, education, etc.

The above information is usually fragmented in several databases. For instance, in Europe physical data on the buildings and socio-economic data on urban areas can be retrieved from Building Census (BC) and European Urban Audit (EUA), respectively, both maintained by EUROSTAT. Information on usage is usually provided by a land use plan (LUP), maintained from a local source like the municipality.

It is noted that this situation poses a problem of 'mesh compatibility', since each source usually adopts a different meshing of the territory. The 'raster' approach followed in the presented model is to generate a set of mutually exclusive and collectively exhaustive cells of regular geometry (with a variable adaptive density) that cover the study region, as shown for instance in Fig. 18.3. Data from each source are then projected onto the grid cells using an area ratio. Therefore, by design, as opposed to the districts in the source databases, the cells are not internally homogeneous in terms of, for example, buildings and socio-economic indicators. All cells are characterised in terms of physical and socio-economic parameters that describe the total number of buildings, as well as its distribution among different types, together with fragility, total population and its distribution in terms of income, education, etc.

Coming to the central role of buildings in the SOS, each cell has a number of dependency edges connecting it with nodes in all other systems. Technically, as already mentioned, each cell 'knows' (maintains a pointer to) its reference nodes, e.g. an EPN substation feeding power to it, a WSS node supplying water, a RDN junction where incoming and outgoing traffic transits into the transportation system, etc. Conversely, nodes in these networks maintain a list of pointers to all 'tributary cells'. This dependence, represented in Plate I (between pages 452 and 453), is one of the main features of the model. The set of 'tributary cells' is employed to automatically assemble demands in the relevant nodes, in a way that is consistent across all systems with the basic demographic and socio-economic data (note that the user is left free to assign demands directly in the relevant nodes to consider various situations). Of course, this approach requires availability of models for each demand quantity, good or service, as a function of, at least, population size (examples are described in the next section). Further, this



18.3 Polygons on different layers indicate the geographical units used to specify buildings, demographic and other socio-economic data. The figure also shows the automatically generated grid. The hypothetical region with three cities is the same used later in the example application (see Section 18.8).

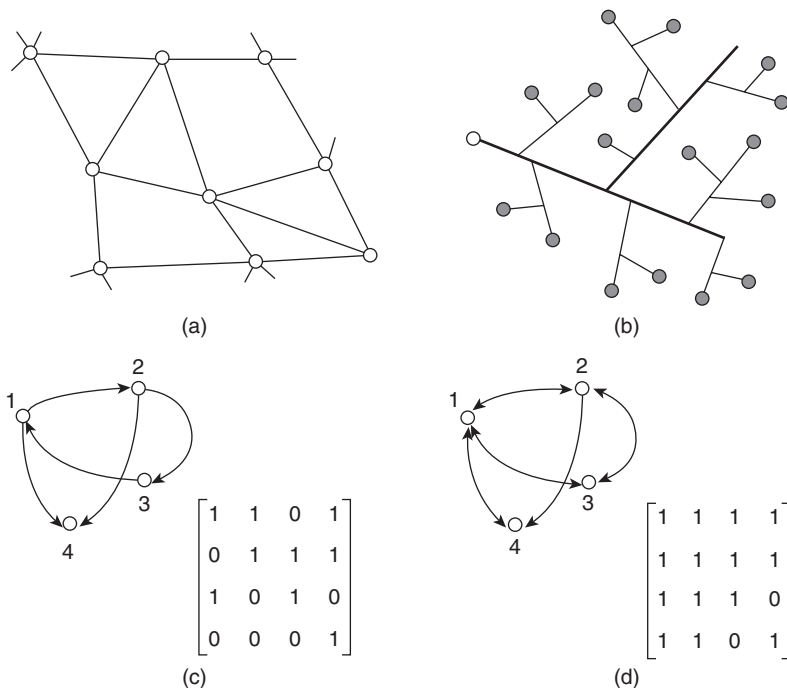
mechanism makes the meshing refinement of each system within the Infrastructure independent of each other. This set-up has an influence on the accuracy of the analysis results, affected by large differences in the meshing refinement. This is a compromise that is tolerated in the model due to the inhomogeneous quality of available data.

The *InhabitedArea* class has no sub-classes, and each object instantiated from this class describes a single cell of the grid (the grid is automatically generated to cover the entire study region and re-meshing takes place to decrease the cell size in correspondence of urban areas). A detailed description of this class can be found in Cavalieri *et al.* (2012), which illustrates in particular the methods employed to predict physical damage and quantitative estimates of emergency-management relevant parameters, such as *displaced population* and *casualties* (demands on the shelter and the regional health-care system models).

18.5.2 The 'Network' abstract class and sub-classes

Common connectivity model

The importance of the *Network* class described in this section cannot be understated, as network-like systems make up the majority of the infrastructure. From a modelling point of view, there is scope for significant synergy, since these systems share many common features. In particular, all networks are the same at the connectivity-level, consisting of a collection of *nodes* and *edges*, i.e. a graph, usually with two distinctively different connectivity patterns depending on the scale. A typical form is a grid-like redundant topology at large scale (transmission), with tree-like structures radiating at a lower scale (distribution), as shown in Fig. 18.4a and b, respectively. Were *integrity* the only concern, a single system *structural* model could suffice, with differences limited to the component models. These systems, however, differ in terms of the physical equations governing the *flow* of the conveyed good/commodity/service. This in turn translates into different *functional* models.



18.4 Topological features of network-like systems: (a) a grid-like system; (b) a tree-like system; (c) a directed graph and corresponding adjacency matrix; (d) an undirected graph.

The above considerations have led to a model that aggregates all connectivity-related methods at the highest possible level, depending on the network, and has a repeating template for the lower portion of each network, so that in most cases, the same methods are present even though with slightly different implementations for each particular system.

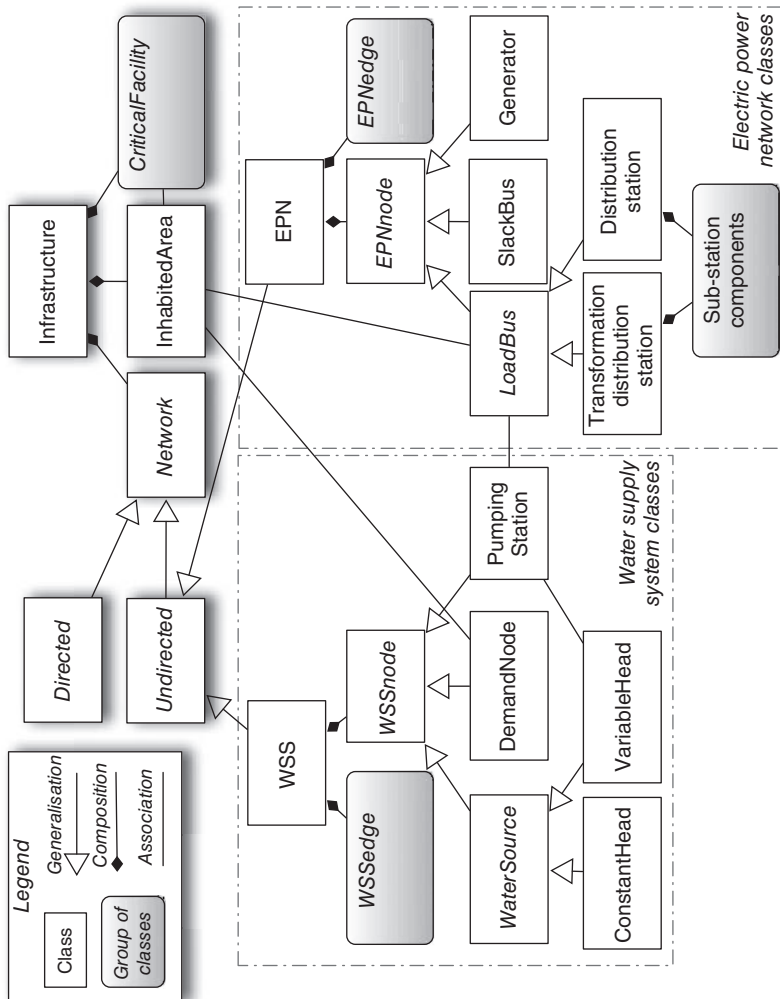
Figure 18.5 shows the class diagram of the *Network* class and sub-classes in the current model. The *Network* class is an abstract one. From a computational standpoint, it is convenient to group these systems into *directed* (Fig. 18.4c) and *undirected* (Fig. 18.4d) ones, since most graph-theoretic algorithms exist in specialised, optimised forms based on this distinction. Typically utility networks are of the undirected type, i.e. they have a symmetrical adjacency matrix whereby the existence of an edge from node i to node j implies the opposite, with the actual direction of flow in each edge determined by the solution of the flow equations in the functional network model (the flow model, as opposed to the structural model consisting of a graph). Transportation systems, on the other hand, are typically directed ones.

Examples of the main attributes and methods include, for instance, at the *Network* class level, the two lists of pointers to all *Nodes* and *Edges* in the network, that store the network topology. A further important attribute is *States*. This variable is present in all classes in the object-oriented model and is used to record the object state in each simulation run (see Section 18.7.3). Among the methods, there are auxiliary methods, that include algorithms to convert between alternative existing graph representations, such as from the list of edges to the incidence or adjacency matrices, or search methods, that include the search for a minimum path, given a set of generalised weights on the edges, or for a sub-network satisfying given property requirements.

At the lower level of the *Directed* and *Undirected* abstract classes, there are other methods that, as anticipated, exist in alternative forms depending on the network type. An example is the method *isConnected* (which determines whether there is at least a path between any two nodes in the graph) of the *Undirected* class, which has two counterparts in the *Directed* one, since in this case the concept of connection is replaced by those of strong and weak connections.

It should be underlined how all these methods are of great importance beyond the basic integrity analysis of each system. Indeed, rapid knowledge of the state of fragmentation of a network can avoid assembling and solving flow equations for which a feasible solution does not exist. The common connectivity model extends to the template for each concrete class at the lower level. All are the composition of two classes, one for nodes and the other for edges, both of the abstract type.

Classes describing nodes in each concrete network model store the geographical *position* (latitude, longitude and altitude) of the node and whether



18.5 The class diagram of the *Network* class and sub-classes, down to components of the EPN and WSS.

the component located at the site of the node is vulnerable to seismic hazard. More appropriately, it determines if a vulnerability model for the component exists and can be adopted in the model. When the component is vulnerable, its fragility model is a function of a scalar or vector IM that needs to be specified, and is influenced by site amplification (see Section 18.5.4). The latter is described by a model that will take as an input a V_{s30} value (a proxy for site class) stored in the corresponding node object as an attribute of the class. Classes describing edges in each network store the edge *connectivity*, its *centroid* (computed from the connected nodes' positions), and parameters related to seismic vulnerability of the edge (pipe, line, road segment, etc.). The next sections provide more details on two specific networks, in terms of the implemented functional model and the vulnerability models of the components.

Model of a WSS

Water supply systems are networks whose edges and nodes are pressure pipes and either pipe junctions, water sources or end-users, respectively. Their function is to provide end-users with potable water with a sufficient pressure level. A WSS can be decomposed into hierarchically arranged tiers. The first layer collects all pipes in the main distribution which (within urban areas) follow the main roads and convey the main water flow. This is usually designed with a redundant grid-like topology to ensure reliable connection of the sub-components. The latter follows lower-order roads and have a more vulnerable tree-like topology (but are made of more easily repairable smaller diameter pipes and serve smaller demands, thus limiting the impact of service interruption).

The class diagram for a WSS is shown in Fig. 18.5 together with that of an EPN. Both *WSSedge* and *WSSnode* are abstract classes. Edges can be either pipes or tunnels. Nodes can be demand nodes, pumping stations, or water sources. The latter can be either constant head sources (such as a river or a dam) or variable head sources (such as a tank). The distinction has an influence on the solution of the flow equations.

Examples of the main attributes and methods include, for instance, at the WSS level, the *waterEquipment* attribute, that stores the water daily equipment per person in the study region, a parameter employed by the *computeDemand* method to estimate demands in the demand nodes starting from the population in the tributary cells. Two more methods are the *computeFlow* and the *discretiseEdge*, which assemble and solve the flow equations in the functional model, and automatically subdivide edges in smaller length segments to ensure a predefined maximum segment length, respectively.

As discussed also in Section 18.7, vulnerable components in a system can be of two types: point-like and line-like components. The former usually

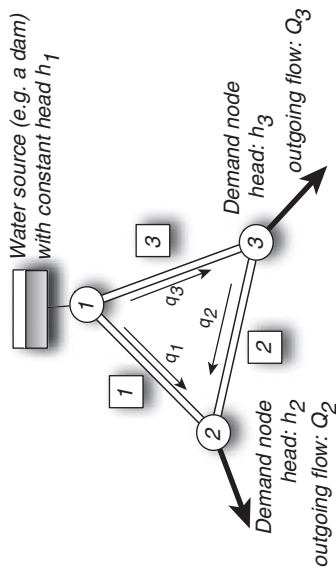
have fragility models in the form of a set of fragility functions, each describing the probability of attaining or exceeding a pre-defined limit-state or performance-level as a function of a scalar or vector IM. In contrast, line-like components commonly have fragility models in the form of a Poisson distribution of damage per unit length, with rate that is again a function of a scalar or vector IM. Within a WSS, pipes have a line-like fragility model, while other components, such as dams (Lupoi and Callari, 2011), tanks or pumping stations, have point-like fragility models. For long pipes, going through different soil conditions and with varying distance from the epicentre, whether the IM and hence the rate of damages (ruptures and/or leakages) is sampled at a single point (the centroid) or at many points along the line (the centroids of the automatically generated smaller segments), clearly makes a difference in the predicted damage distribution. Other attributes of pipes include the depth (derived from that of the connected nodes), the diameter, the roughness, the numbers of breaks and leaks, and the leakage area (equal to the total area if at least a break is present).

The functional model for this network consists of the $N + E$ nonlinear flow equations (Houghtalen *et al.*, 2009):

$$\begin{cases} \mathbf{A}_N^T \mathbf{q} - \mathbf{Q}(\mathbf{h}_N) = \mathbf{0} \\ \mathbf{R}|\mathbf{q}| + (\mathbf{A}_N \mathbf{h}_N + \mathbf{A}_S \mathbf{h}_S) = \mathbf{0} \end{cases} \quad [18.1]$$

where N , E and S are the number of internal (non-source) nodes, of edges and of sources, respectively. The first N equations express flow balance at the internal nodes (sum of incoming and outgoing flows equal either zero or the demands in the end-user nodes), while the next E equations express the flow resistance of the edges.

The $E \times N$ and $E \times S$ matrices $\mathbf{A}_N$ and $\mathbf{A}_S$ are sub-matrices of the $E \times (N + S)$ matrix $\mathbf{A}$ which contains 0, 1 and -1 terms as a function of the network connectivity. The $N \times 1$ and $S \times 1$ vectors $\mathbf{h}_N$ and $\mathbf{h}_S$ are the corresponding partitions of the $(N + S) \times 1$ vector $\mathbf{h}$ collecting the N unknown heads in the internal nodes and the S known heads in the water-source nodes. The $E \times 1$ vector $\mathbf{q}$ collects the unknown flows in the E links, and $\mathbf{R}$ is the $E \times E$ diagonal matrix of resistance, with terms $r_i = u_i L_i$, where $u_i = \beta D^{-5}$ (according to Darcy's law) and L_i is the i th link length. Figure 18.6 (top) shows a sample WSS, with three nodes and three edges, and the corresponding equations. As shown in the figure there are five equations in the five unknowns. Two equations express the balance in demand nodes 2 and 3. The next three equations express the dissipation of energy during flow through pipes 1 to 3. The interested reader should consult with recent textbooks such as Houghtalen *et al.* (2009) or Swame and Sharma (2008), for an in-depth treatment of water systems analysis.



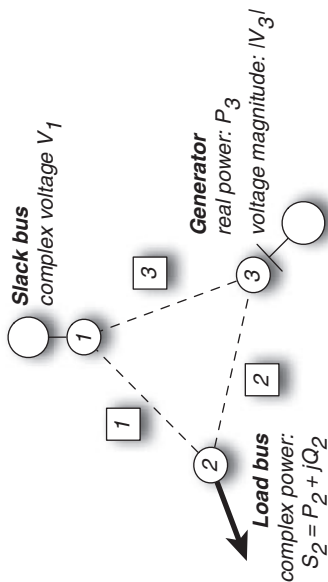
(a)

Node 2: $q_1 + q_2 - Q_2 = 0$
 Node 3: $-q_2 + q_3 - Q_3 = 0$
 Pipe 1: $-h_1 + h_2 + r_1 |q_1| q_1 = 0$
 Pipe 2: $-h_3 + h_2 + r_2 |q_2| q_2 = 0$
 Pipe 3: $-h_1 + h_3 + r_3 |q_3| q_3 = 0$

Known terms: h_1 and D_i, L_i, β_i ($i = 1, 2, 3$)

Unknowns: flows q_1, q_2, q_3 and heads h_2, h_3

(b)



(c)

Node 2: $P_2 = |V_2| [|V_1| (G_{21} \cos \theta_{21} + B_{21} \sin \theta_{21}) + |V_2| G_{22} + |V_3| (G_{23} \cos \theta_{23} + B_{23} \sin \theta_{23})]$
 $Q_2 = |V_2| [|V_1| (G_{21} \sin \theta_{21} - B_{21} \cos \theta_{21}) + |V_2| (-B_{22}) + |V_3| (G_{23} \sin \theta_{23} - B_{23} \cos \theta_{23})]$
 Node 3: $P_3 = |V_3| [|V_1| (G_{31} \cos \theta_{31} + B_{31} \sin \theta_{31}) + |V_2| (G_{32} \cos \theta_{32} + B_{32} \sin \theta_{32}) + |V_3| G_{33}]$

Known terms: $\left\{ \begin{array}{l} \text{complex voltage } V_i \text{ and power } S_2 \\ \text{real power } P_3 \text{ and voltage magnitude } |V_3| \\ \text{and line impedances } z_i \text{ (} i = 1, 2, 3 \text{)} \end{array} \right.$

Unknowns: voltage magnitude $|V_2|$ and phase angles δ_2, δ_3

(d)

18.6 Elementary WSS (top) and (EPN) bottom, and the corresponding flow equations.

It can be observed that in the above set of equations the end-user demands $\mathbf{Q}(\mathbf{h}_N)$ are written as functions of unknown heads in the internal nodes. The solution of the system in this form is called ‘head-driven’, and is preferred for the perturbed seismic conditions where satisfaction of prescribed demands is not guaranteed. It is customary in the analysis of WSS to treat the end-user demands $\mathbf{Q}$ as fixed boundary conditions (the system must be proportioned in order to satisfy them). The solution of the system with $\mathbf{Q}$ independent of $\mathbf{h}_N$ is called ‘demand-driven’.

The set of nonlinear equations holds in so-called stationary conditions, i.e. it assumes end-user demands constant over time. This is a simplification, and is valid as long as the boundary conditions vary smoothly with time (i.e. quasi-stationary conditions). In seismic conditions, this is not the case where abrupt variation of water demands due to pipe ruptures and leakages is soon replaced by a new stationary state.

Model of an EPN

In an EPN, edges are transmission or distribution lines, while nodes are either source buses, transformation buses or demand/load buses (a bus is the EPN-specific term denoting a node). The EPN function is to provide end-users with electric power having a voltage level within a narrow range for the functioning of electrical devices. Source buses, enclosed in power plants, can be of two different types, slack or generation. Each EPN has only one slack bus that has the function of compensating the unbalance between the power required by end-users and the power provided by generator buses.

The EPN lines compose a Transmission and Distribution (T&D) system, divided into two general tiers: a transmission system with a grid-like topology, which spans long distances at high voltages, usually between 60 and 750 kV, and a more local distribution system at intermediate voltages, characterised by a tree-like topology. The latter is further divided into a *medium* voltage distribution system at voltages in the low tens of kV, and a *low* voltage distribution system, which consists of the wires that directly connect most domestic and small commercial customers, at voltages that vary worldwide between 100 and 240 V. The distribution system can be both overhead and underground.

Lines at different voltages are terminated in substations. In general, an electric substation is a facility that serves as a source of energy supply for the local distribution area. Depending on their functions, substations can be of three different typologies, i.e. transformation, distribution and transformation/distribution. When the transformation function is required, two buses at two different voltages are present, while in distribution substations only one bus is present (load bus).

The class diagram for an EPN is shown in Fig. 18.5. Both *EPNedge* and *EPNnode* are abstract classes. Edges can be either underground or overhead lines. Nodes can be generator stations, slack buses, or load stations. The latter can be either transformation/distribution substations or distribution substations. The distinction has an influence on the function and the number of buses included in the model.

Examples of the main attributes and methods include, for instance, at the *EPN* level, the *capacityPerCapita* attribute that stores the required power per person, used by the *computeDemand* method jointly with the population in the node tributary cells to estimate the electric power demand. Two more examples are the *powerFlow* and the *defineStatCompState* methods. The former assembles and solves the flow equations in the functional model, while the latter, based on the IM at sites and the components' fragility curves, samples the damage state for all components (buses, switches, transformer, etc.) in the generic substation.

Within an EPN, overhead lines are generally not considered as vulnerable components (Vanzi, 1996), while underground lines may be modelled as vulnerable. Components in the substations have point-like fragility models. Power plants, if considered vulnerable, should be treated as critical facilities within this model (see Section 18.5.3); however, they can be modelled in a more simplistic way. In the current implementation, however, no vulnerability model is yet available for power plants.

As far as the functional model for the EPN is concerned, a very important and delicate aspect in the capacitive (flow) analysis of an EPN in a seismically active environment is the determination of the functional portion of the EPN in the post-earthquake conditions. This does not simply mean what components are damaged. Indeed, damage to components of a substation can lead to a short-circuit (SC) that may or may not propagate within the substation and/or further away from the substation to adjacent others, generating in extreme cases large-scale black-outs.

Hence, the functional model implemented consists of a sequence of two models. The analysis of SC propagation, where circuit breakers are active components playing a key role in arresting the SC spreading, precedes the power-flow analysis to determine voltage and power in all stations as well as current power and power loss in all transmission lines. The sub-model for SC propagation (Vanzi, 1996) is the reason why in the presented framework sub-stations are modelled with all their components and internal (electric) logic, so that SC propagation can be evaluated in each damaged state. In order to facilitate the modelling work for the analyst, the model automatically sets up all the components and links within a substation based on the substation type.

Finally, the set of nonlinear power-flow equations is solved for the functional portion of the EPN, which are given by:

$$\begin{cases} 0 = -P_i + \sum_{k=1}^N |V_i| |V_k| (G_{ik} \cos \theta_{ik} + B_{ik} \sin \theta_{ik}) \\ 0 = -Q_i + \sum_{k=1}^N |V_i| |V_k| (G_{ik} \sin \theta_{ik} - B_{ik} \cos \theta_{ik}) \end{cases} \quad [18.2]$$

where P_i and Q_i are the real and reactive power at node i , $|V_k|$ and $|V_i|$ are the voltage magnitude at nodes k and i , G_{ik} and B_{ik} are the real and imaginary parts of the ik term in the bus admittance matrix $\mathbf{Y}$, and θ_{ik} is the difference of voltage phases between nodes i and k . The solution can be found by using one of the classical derivative algorithms, e.g. Newton–Raphson. Once the station voltages are known, it is straightforward to compute the line currents and powers in both directions, the line power loss, and the powers (real and reactive) in all generation stations.

Figure 18.6 (bottom) shows a sample EPN, with three nodes and three edges, and the corresponding equations. As shown in the figure there are three equations for the three unknowns. Two equations express the real and reactive power balance for the load bus in node 2, while the third equation expresses real power balance for the generator in node 3. The known line impedances z_i do not appear explicitly in the equations, but are used to form the admittance matrix and hence are inside the G and B terms. The example is taken from Saadat (2002), which the interested reader should consult for an in-depth treatment of the subject. Similar to the WSS, the set of nonlinear equations holds in so-called stationary conditions, i.e. it assumes end-user demands constant over time.

18.5.3 The ‘CriticalFacility’ abstract class and sub-classes

This class is abstract and has a number of concrete classes that correspond to facilities. This consists of one or more buildings in an area that can be regarded as a single site at the scale of an urban/regional analysis. It also accommodates a function that is critical to the continued reliable working of the infrastructure, as well as to its response to disturbances. Examples include health-care facilities, (nuclear) power generation plants, (hazardous) industrial plants, harbours, etc. These systems are critical for the emergency response to a disaster, the availability of a critical energy supply, can trigger compounding hazards (toxic clouds, fires), or are critical to the transportation system.

The critical characteristics of the facilities warrants a special attention and modelling approach that goes beyond the simplistic approach adopted for commercial/residential buildings. The damage states of buildings and their content, as well as of the incoming flow of commodities to maintain functionality, needs to be described in detail in order to evaluate the

functional consequences and capture the full range on intermediate working conditions that can characterise these systems after a seismic event.

For instance, after a seismic event health-care facilities can have a variable number of beds available for hospitalisation, and of functional/staffed operating theatres for delivering surgical treatments. These quantities (beds and operating theatres) are a function of human, organisational and physical macro-components, and are determined based on the physical and functional damage to the structures and non-structural components. A methodology to model a health care facility in this detail is available in Lupoi *et al.* (2008), and is implemented in the model.

18.5.4 The 'Seismic' class

This class implements a distributed seismic hazard model, which, starting from an event on one of the sources/faults in the regional seismic environment (*Event* and *Source* classes in Fig. 18.2) predicts at each site the vector-valued IM (*LocalIntensity* class in Fig. 18.2) needed as an input to components' fragility models to evaluate their state of physical damage. The reason why the hazard model must be capable of predicting vector-valued IMs is firstly that fragility models can be function of vector IMs and, more generally, that several components may be located at the same site, each sensitive to different IMs. Moreover, vectors needed at site i and at site j may be different. Hence the output of the model implemented in the *Seismic* class is a vector $\mathbf{s} = \{\mathbf{s}_1 \dots \mathbf{s}_n\}$ of length $m = \sum m_i$, where $\mathbf{s}_i$ is the vector of length m_i of seismic IMs at the i th site.

This vector exhibits a variable degree of statistical dependence between its components, decaying with the distance between the sites, and usually larger between the components within each $\mathbf{s}_i$ (zero distance). There is sufficient statistical support for adopting a joint lognormal distribution for $\mathbf{s}$, under which assumption the statistical dependence is fully described in terms of the correlation among the components of $\mathbf{s}$.

Available data from past earthquakes have been exploited to estimate models for both the *within-site* correlation between different IMs (notably, spectral ordinates, e.g. Inoue and Cornell 1990; Baker and Cornell 2006), and to estimate models of *across-sites* or *spatial* correlation for the same IM (Jayaram and Baker, 2009). Ongoing work is also aimed at developing a cross-IM spatial correlation model (Weatherill *et al.*, 2012), which would allow sampling directly spatially distributed seismic intensities ('shake fields') from the probability density of $\mathbf{s}$, an approach already found, e.g. in Goda and Hong (2008).

The approach adopted in the implemented class does not rely on cross-IM spatial correlation. The hazard model is a sequential one where local

intensity at the surface given an event (generated in step 1) is predicted in two steps (2 and 3):

1. Events are generated in terms of magnitude and localisation (event generation model);
2. Spatial correlation is first dealt with:
 - An IM is chosen to predict a ‘shake field’ $s_{gr1}(\text{lat}, \text{long})$ on a *regular grid* that covers the entire study region. This IM, called ‘primary’, incorporates the spatial correlation (spatial correlation model), and is predicted on rock/stiff-soil conditions by using a ground motion prediction equation (GMPE) (class *GMPE* in Fig. 18.2). It represents the field of ‘bedrock’ motion.
 - The primary IM is interpolated at each site from the four closest grid points (see Fig. 18.9); the use of a regular grid, with grid size related to the field correlation length only, has two advantages. The first is that the number of points is independent of the number of *vulnerable* sites, and depends only on the extent of the study region. The second is that quasi-singular correlation matrices and the associated numerical problems in the simulation, produced by the fact that sites tend to be naturally lumped in clusters, are avoided.
3. Within-site cross-IM correlation is then introduced.
 - The remaining components of s_{ir} at each site (still ‘on rock’), called ‘secondary’ IMs, are sampled from the conditional distribution (secondary IM conditional model) $f(s_{ir2}|s_{ir1})$. These secondary IMs are still ground shaking quantities (e.g. spectral ordinates conditional on peak ground acceleration (PGA)).
 - Vector s_{ir} is multiplied component-wise times the vector $\mathbf{A}_i$ of the amplification functions to obtain surface intensities (amplification model, implemented in the *Amplification* class). Both simple deterministic code-based amplification, as well as probabilistic amplification are implemented. In the former case a single constant amplification factor is used for all frequencies. In the latter the ratio of surface to bedrock intensity for a number of frequencies is described by a vector $\mathbf{A}_i$ of correlated random variables.
 - Finally, where needed, geotechnical hazards are also predicted to complete the vector of the secondary IMs (class *geotechHazards*, with sub-classes, not indicated in Fig. 18.2, *landslide*, *liquefaction* and *coseismicRupture*).

The event generation model consists of a set of seismic sources (class *Source*, Fig. 18.2), described either in terms of a polygonal area or a fault plane, with:

- Magnitude recurrence model, either a characteristic or a truncated Gutenberg–Richter model:

$$\lambda_i(m) = \lambda_{0,i} [1 - F_M(m)] = \lambda_{0,i} \frac{e^{-\beta_i m} - e^{-\beta_i m_{u,i}}}{e^{-\beta_i m_{l,i}} - e^{-\beta_i m_{u,i}}} \quad [18.3]$$

where $\lambda_{0,i}$ is the mean annual rate of all events with magnitude between the lower bound magnitude $m_{l,i}$ and the upper bound magnitude $m_{u,i}$, and β_i is the rate of decay of the mean annual frequency with magnitude.

- Localisation model, either a uniform distribution for a point-source model within a polygonal seismic source, or a more refined model that predicts the rupture area and position over the fault plane, as a function of magnitude.

The model for the primary IM consists of a ground-motion prediction equation (the abstract class *GMPE* in Fig. 18.2), which in general has the form:

$$\ln s_i = \mu_{\ln s}(M, R_i, F, V_i) + \varepsilon_i \sigma_i + \eta_i \tau_i \quad [18.4]$$

where $\mu_{\ln s}(M, R_i, F, V_i)$ is the mean of the logarithm, or log-mean, of the intensity as a function of magnitude M , distance R , faulting style F and local site conditions, as expressed by the shear wave velocity V . Total variability of the intensity around the predicted log-mean is split in two terms, the intra-event error $\varepsilon_i \sigma_i$ which varies from site to site, for a given earthquake, and the inter-event error $\eta_i \tau_i$ which varies from event to event but is constant for all sites (with ε_i and η_i standard Gaussian variables).

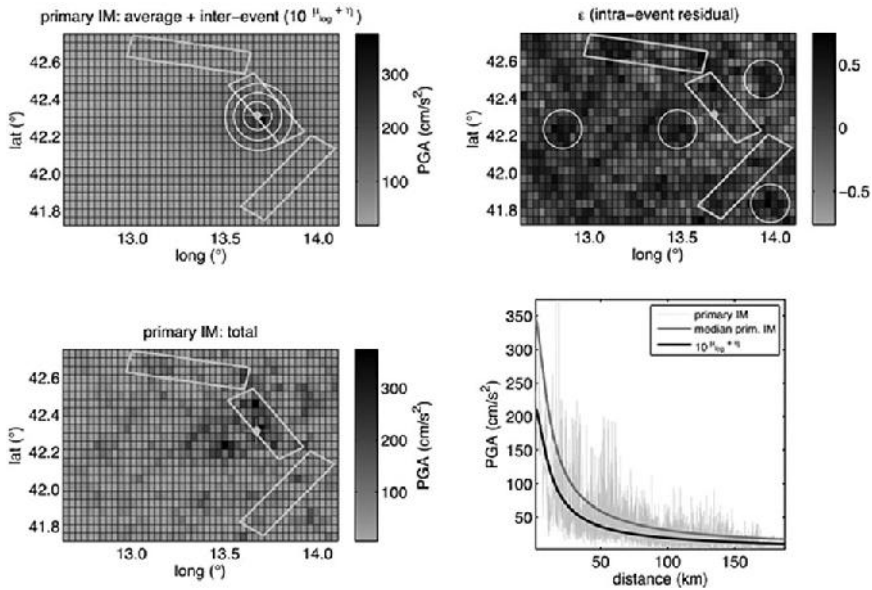
Simulation of (spatially) correlated intra-event residuals is carried out according to:

$$\mathbf{\varepsilon} = \mathbf{L}\mathbf{u} \quad [18.5]$$

where $\mathbf{u}$ is a vector of independent standard Gaussian variables and $\mathbf{L}\mathbf{L}^T = \mathbf{R}$ is the Cholesky decomposition of the correlation matrix $\mathbf{R}$ whose terms are obtained from an exponential correlation model of decay with distance of the form:

$$\rho_{\varepsilon_i \varepsilon_j}(h) = e^{\frac{-bh}{r}} \quad [18.6]$$

where b and r (the so-called ‘range’, or correlation distance) are the model parameters. Figure 18.7 shows one simulated *shake-field* of the primary IM, which has been obtained from the models employed for the application in Section 18.8. The figure shows four plots. Plots (a) to (c) are maps on the study region of the application. In each map the three rectangular areas



18.7 Sample simulation of the primary IM (PGA) for a magnitude 7 event in zone 2: (a) circular decay of the 'average' value ($10^{H_{log} + \eta}$); (b) field of intra-event residuals (the plotted quantity is 10^ϵ , i.e. the factor that multiplies the intensity in plot (a)), note the clusters of close values caused by spatial correlation; (c) total shake field; (d) plot of primary IM versus epicentral distance.

are the sources. The shake-field refers to an event on source 2, with the epicentre identified by a thick dot. Plot (a) shows the average intensity only, i.e. the intensity obtained by neglecting the intra-event terms ϵ , and illustrates the circular nature of the log-mean of the GMPE. Plot (b) shows the map of intra-event residuals, and small circles highlight the tendency of ϵ values to form cluster of size related to the correlation length. Plot (c) shows the resulting total shake-field. Plot (d) with the IMs values as a function of epicentral distance, shows the exponential decay of the GMPE mean, as well as the important intra-event deviations produced by the ϵ terms.

Once a value of the primary IM is obtained at each site through interpolation from the main shake-field, the assumption of joint lognormality of $\mathbf{s}_{ir}$ allows sampling $\mathbf{s}_{ir2}$ from the conditional lognormal distribution (subscripts i and r have been dropped for clarity) (Bazzurro and Cornell, 2002):

$$f(\mathbf{s}_2 | \mathbf{s}_1) = \frac{1}{(2\pi)^{-n/2} \sqrt{\det \mathbf{R}_{\ln \mathbf{s}_2 | \ln \mathbf{s}_1}} \prod_j s_{2j} \sigma_{\ln s_{2j} | \ln s_1}} \cdot \exp \left[-\frac{1}{2} (\ln \mathbf{s}_2 - \boldsymbol{\mu}_{\ln \mathbf{s}_2 | \ln \mathbf{s}_1})^T \mathbf{C}_{\ln \mathbf{s}_2 | \ln \mathbf{s}_1} (\ln \mathbf{s}_2 - \boldsymbol{\mu}_{\ln \mathbf{s}_2 | \ln \mathbf{s}_1}) \right] \quad [18.7]$$

where the conditional mean vector and the conditional covariance and correlation matrices are:

$$\boldsymbol{\mu}_{\ln s_2 | \ln s_1}(M, R, F, V) = \boldsymbol{\mu}_{\ln s_2}(M, R, F, V) + \mathbf{C}_{\ln s_2 \ln s_2 | \ln s_1} \frac{\ln s_1 - \mu_{\ln s_1}(M, R, F, V)}{\sigma_{s_1}^2} \quad [18.8]$$

$$\mathbf{C}_{\ln s_2 \ln s_2 | \ln s_1} = \mathbf{C}_{\ln s_2 \ln s_2} - \frac{\mathbf{C}_{\ln s_2 \ln s_1} \mathbf{C}_{\ln s_1 \ln s_2}}{\sigma_{s_1}^2} \quad [18.9]$$

$$\mathbf{R}_{\ln s_2 \ln s_2 | \ln s_1} = \mathbf{D}_{\ln s_2 \ln s_2 | \ln s_1}^{-1} \mathbf{C}_{\ln s_2 \ln s_2 | \ln s_1} \mathbf{D}_{\ln s_2 \ln s_2 | \ln s_1}^{-1} \quad [18.10]$$

with:

$$\mathbf{C}_{\ln s \ln s} = \begin{bmatrix} \sigma_{s_1}^2 & \mathbf{C}_{\ln s_1 \ln s_2} \\ \mathbf{C}_{\ln s_2 \ln s_1} & \mathbf{C}_{\ln s_2 \ln s_2} \end{bmatrix} = \mathbf{D}_{\ln s \ln s} \mathbf{R}_{\ln s \ln s} \mathbf{D}_{\ln s \ln s} \quad [18.11]$$

$$\mathbf{D}_{\ln s \ln s} = \begin{bmatrix} \sigma_{s_1} & \cdots & 0 \\ \vdots & \ddots & \vdots \\ 0 & \cdots & \sigma_{s_n} \end{bmatrix} \quad [18.12]$$

$$\mathbf{R}_{\ln s \ln s} = \begin{bmatrix} 1 & \cdots & \rho_{\ln s_1 \ln s_n} \\ \vdots & \ddots & \vdots \\ \rho_{\ln s_n \ln s_1} & \cdots & 1 \end{bmatrix} \quad [18.13]$$

The standard deviations in the above equations are *total* standard deviations $\sigma_T = \sqrt{\sigma^2 + \tau^2}$. Models for the within-site correlation coefficient are available in the literature, e.g. for spectral accelerations at different periods as in Inoue and Cornell (1990) with the simple form (valid for periods between 0.1 s and 4.0 s):

$$\rho_{\ln s_i \ln s_j} = 1 - 0.33 \ln(T_j / T_i) \quad [18.14]$$

18.6 Performance metrics

This section describes metrics to quantitatively measure the performance of an infrastructure and its components. These metrics often express either the comparison of a demand with a capacity quantity, as a difference or ratio, the consequence of a mitigation action, or the assembled consequences of all damages (the ‘impact’). In general, for an infrastructure (SOS), they can be categorised according to the three hierarchical levels into:

- component-level metrics;
- system-level metrics;
- infrastructure-level metrics.

This is the approach adopted in the current model, and computed values of each metric are stored in the corresponding object. Further, since the system is affected by significant uncertainty, probabilistic metrics are also employed. These are usually defined based on the deterministic counterparts, and take the form of their statistical moments and/or distributions. The next sections describe briefly some of the implemented performance metrics.

18.6.1 Component-level metrics

The infrastructure has been subdivided into three component classes, *InhabitedArea*, *Network* and *CriticalFacility*, and metrics can be grouped accordingly. For networks, at the component-level, they usually take the form of:

- a Boolean variable indicating the existence of a connection of the node with a node or set of nodes of interest;
- a ratio of post-earthquake to pre-earthquake value of a network-dependent quantity at a node, usually associated with a flow evaluation.

While the former type is basically network-independent, the second is obviously network specific. Examples include the voltage ratio $VR_i = V_{i,s}/V_{i,0}$ for an EPN, ratio of the voltage in the i th bus in the seismically damaged network to the reference value for non-seismic, normal conditions, and the head ratio $HR_i = H_{i,s}/H_{i,0}$ for a WSS, ratio of the water head in the i th node in the seismically damaged network to the reference value for non-seismic, normal operation conditions. It is well worth noting how, since voltage or water head computations require a flow analysis on the whole network, these metrics express functional consequences in the i th component of the physical damage to all system components (and, possibly, to other system components when interactions are accounted for).

For the *InhabitedArea* class, component-level metrics (i.e. per grid cell) include the percentage of damaged buildings (per category), and the number of casualties, fatalities, or displaced people. For the *CriticalFacility* class, metrics include the number of available beds and functional operating theatres in health care facilities. For the VR and HR measures, quantities, such as displaced people or operating theatres, are defined at component-level but reflect much wider system-level behaviour through their evaluation.

18.6.2 System-level metrics

For networks, system-level metrics can take the form of:

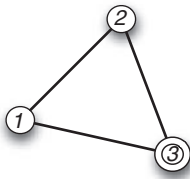
- Connectivity-related measures, such as the Simple or Weighted Connectivity Loss, which express the average reduction in connectivity

amongst sinks and sources. The latter, depending on the network type, can be interpreted as the reduction in the ability of sinks to receive flow from sources (e.g. for EPN or GAS), or the average increase of the path amongst sinks and sources (e.g. for RDN). The simple connectivity loss (SCL) is defined as $SCL = 1 - (\sum N_{i,s} / N_{i,0}) / n$ where $N_{i,0}$ and $N_{i,s} = \sum_{j=1}^{N_{i,0}} I_{ij,s}$ are the number of sources connected to the i th sink in undamaged and damaged conditions, respectively. The weighted connectivity loss (WCL) is defined as $WCL = 1 - [\sum N_{i,s} W_{i,s} / (N_{i,0} W_{i,0})] / n$, where the weights are defined as $W_{i,0} = \sum_{j=1}^{N_{i,0}} W_{j,0}$ and $W_{i,s} = \sum_{j=1}^{N_{i,0}} I_{ij,s} W_{j,s}$, as a function of other weights $W_{j,s}$ assigned to the sources connected to each node. The choice of the latter weights depends on the network type. For some networks (e.g. EPN or GAS) setting the weight $W_{j,s} = Q_{j,s}$ equal to the flow from the sources is meaningful, for others (e.g. RDN) it makes more sense to use as weight the inverse number of edges from the source to the node of interest $W_{j,s} = 1/N_{ij,s}$. Figure 18.8 shows an example of computation of the SCL and WCL measures for a simple network with three nodes (one source and two sinks) and three edges. Notice how the fact that, even if connectivity is maintained in the third run, the path from sink 2 to source 3 is longer and penalised by a value of $WCL > SCL$.

- Flow-related measures, such as the system serviceability index SSI , defined as the ratio of the sum of satisfied demands in the post-earthquake damaged conditions to the sum in the reference undamaged pre-earthquake conditions. Usually its definition assumes that the demand remains fixed before and after the earthquake. Note that the index looks only at a single system, without considering its interactions with other infrastructural systems. It can be enhanced (enhanced system serviceability index, $ESSI$) to account for demand variation. It can take the form of a difference of weighted flows, as for instance in the Driver's Delay metric (Shinozuka *et al.*, 2003).

Examples of type 1 metrics include, with reference to an EPN, the Power Loss $PL = 1 - (\sum P_{i,s} / P_{i,0}) / n$, which corrects the connectivity loss with the size of the power plants (real power P in MW) to which load buses are still connected to. Another metric is focused on the Impact on the Population $IP = 1 - (\sum POP_i P_{i,s} / P_{i,0}) / \sum POP_i$, where the summation extends to the number of load buses in the EPN, and POP_i is the population fed by the i th load bus (Poljanšek *et al.*, 2010).

Examples of type 2 metrics include the SSI , which can be defined, e.g. for an EPN, as the ratio of the real power delivered at load buses after an earthquake to that before the earthquake. The SSI is given by $SSI = 100 [\sum P_{i,0} (1 - R_i) w_i / \sum P_{i,0}]$, where $P_{i,0}$ is the delivered power at



Reference non-seismic conditions

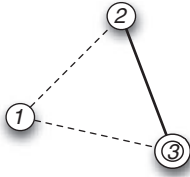
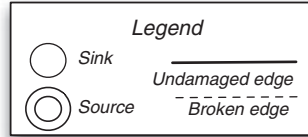
$$N_{1,0} = 1$$

$$N_{2,0} = 1$$

$$W_{1,0} = I_{1,0} W_{1,0} = I_{1,0} \frac{1}{N_{1,0}} = 1 \cdot \frac{1}{1} = 1$$

$$W_{2,0} = I_{2,0} W_{1,0} = I_{2,0} \frac{1}{N_{2,0}} = 1 \cdot \frac{1}{1} = 1$$

(a)

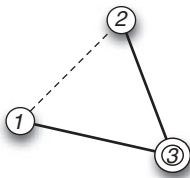


Seismic conditions, simulation run #1

$$\left. \begin{array}{l} N_{1,s} = 0 \\ N_{2,s} = 1 \end{array} \right\} \rightarrow SCL = 1 - \frac{1}{2} \left(\frac{0}{1} + \frac{1}{1} \right) = 0.5$$

$$\left. \begin{array}{l} W_{1,s} = I_{1,s} W_{1,s} = I_{1,s} \frac{1}{N_{1,s}} = 0 \cdot \frac{1}{\infty} = 0 \\ W_{2,s} = I_{2,s} W_{1,s} = I_{2,s} \frac{1}{N_{2,s}} = 1 \cdot \frac{1}{1} = 1 \end{array} \right\} \rightarrow WCL = 1 - \frac{1}{2} \left(\frac{0}{1} \cdot \frac{0}{1} + \frac{1}{1} \cdot \frac{1}{1} \right) = 0.5$$

(b)

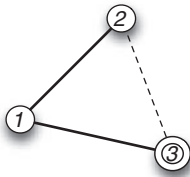


Seismic conditions, simulation run #2

$$\left. \begin{array}{l} N_{1,s} = 1 \\ N_{2,s} = 1 \end{array} \right\} \rightarrow SCL = 1 - \frac{1}{2} \left(\frac{1}{1} + \frac{1}{1} \right) = 0$$

$$\left. \begin{array}{l} W_{1,s} = I_{1,s} W_{1,s} = I_{1,s} \frac{1}{N_{1,s}} = 1 \cdot \frac{1}{1} = 0 \\ W_{2,s} = I_{2,s} W_{1,s} = I_{2,s} \frac{1}{N_{2,s}} = 1 \cdot \frac{1}{1} = 1 \end{array} \right\} \rightarrow WCL = 1 - \frac{1}{2} \left(\frac{1}{1} \cdot \frac{1}{1} + \frac{1}{1} \cdot \frac{1}{1} \right) = 0$$

(c)



Seismic conditions, simulation run #3

$$\left. \begin{array}{l} N_{1,s} = 1 \\ N_{2,s} = 1 \end{array} \right\} \rightarrow SCL = 1 - \frac{1}{2} \left(\frac{1}{1} + \frac{1}{1} \right) = 0$$

$$\left. \begin{array}{l} W_{1,s} = I_{1,s} W_{1,s} = I_{1,s} \frac{1}{N_{1,s}} = 1 \cdot \frac{1}{1} = 0 \\ W_{2,s} = I_{2,s} W_{1,s} = I_{2,s} \frac{1}{N_{2,s}} = 1 \cdot \frac{1}{2} = 0.5 \end{array} \right\} \rightarrow WCL = 1 - \frac{1}{2} \left(\frac{1}{1} \cdot \frac{1}{1} + \frac{1}{1} \cdot \frac{0.5}{1} \right) = 0.25$$

$$E[SCL] = \frac{0.5 + 0 + 0}{3} = 0.16$$

$$E[WCL] = \frac{0.5 + 0 + 0.25}{3} = 0.25$$

(d)

18.8 A basic triangular network and the evaluation of simple and weighted connectivity loss.

load bus i in pre-earthquake conditions, $R_i = |V_{i,s} - V_{i,0}|/V_{i,0}$ is the percent reduction of voltage magnitude, and $w_i = H(0.1 - R_i)$ is a function (H being the Heaviside function) accounting for the small tolerance on voltage reduction: its value is one for $R_i \leq 10\%$ and zero otherwise. The SSI index varies between zero, when there is no solution for the power-flow analysis, and 100 when the EPN remains undamaged after the earthquake. In an analogous manner, the SSI for a WSS can be defined as the ratio of the sum of the delivered water flows after an earthquake to that before the earthquake $SSI = 100 \sum_{i=1}^n Q_{i,s} / \sum_{i=1}^n Q_{i,0}$. This definition is modified from the metrics proposed by Wang *et al.* (2010): $SSI = 100 \sum_{i=1}^n Q_i / \sum_{i=1}^{n_0} Q_i$, where n_0 ($< n$) is the number of satisfied demand nodes after the earthquake and Q_i is unchanged before and after the event (demand-driven analysis, while in the presented model the analysis is head-driven).

18.6.3 Probabilistic metrics

Metrics that account for the uncertainty affecting the system are normally defined based on other metrics, such as those illustrated before. Typical examples are the damage consequence index (DCI) and the upgrade benefit index (UBI), both defined for a WSS (Wang *et al.*, 2010). The DCI measures the impact of damage (including breaks and leaks) on each pipe on the overall system serviceability, in order to identify *critical links* that significantly affect the system seismic performance. It is defined at the component level in terms of the expected value of the SSI, and hence reflects the functional consequence of damage to all systems' components: $DCI_i = (E[SSI] - E[SSI|L_i]) / (1 - E[SSI])$, where $E[SSI]$ is the (unconditional) expected value of SSI from a set of simulations in which the i th pipe might or might not be damaged; and $E[SSI|L_i]$ is the conditional expectation of SSI from another set of simulations under the same seismic hazard, but given that the i th pipe is damaged. As damage to the i th pipe is certain in the calculation of $E[SSI|L_i]$, theoretically, $E[SSI|L_i]$ is always smaller than $E[SSI]$ for which the pipe might or might not be damaged. Therefore, DCI_i is always positive, and is the percentage reduction of SSI given that the i th pipe is damaged.

The UBI measures the impact of an upgrade of an individual pipe on the overall system serviceability, and reflects the systemic functional consequence of damage to the whole system(s) at the component level. It is defined as: $UBI_i = (E_{\text{upgrade}_i}[SSI] - E[SSI]) / (1 - E[SSI])$ in which $E_{\text{upgrade}_i}[SSI]$ is the expected value of SSI given that the i th pipe is 'upgraded'. This means that the probability of pipe damage given an earthquake is significantly reduced with respect to its pre-upgrade value. UBI_i is the percent increase of SSI given that the i th pipe is upgraded, and can be used to identify critical links in seismic mitigation, as those with relatively large UBI values.

Both DCI and UBI can be effectively estimated by conditional sample analysis.

Further examples include the mean annual frequency (MAF) of exceedance of a threshold y based on a performance metric Y :

$$\lambda_Y(y) = \sum_i \lambda_{0,i} G_{Y|i}(y|i) = \lambda_0 \sum_i p_i G_{Y|i}(y|i) = \lambda_0 G_Y(y) \quad [18.15]$$

where $\lambda_0 = \sum_i \lambda_{0,i}$ is the MAF of all earthquakes in the region and $p_i = \lambda_{0,i}/\lambda_0$ is the probability that given the occurrence of an earthquake the active source is the i th one.

18.6.4 Infrastructure-level metrics

Metrics at this higher level usually take the form of a summation over systems or components. These summations require the assembled consequences to have the same units. Commonly employed quantities are money and people, who may be injured, dead or displaced. Currently and consistently with its emergency-phase focus, the model evaluates the total number of casualties, fatalities and the Shelter Seeking Population (SSP), as already mentioned in Section 18.5.1 and in more detail in Cavalieri *et al.* (2012).

Assessment of economic loss is actually a much more difficult proposition, since indirect loss is highly uncertain. A relatively large body of literature is available on the matter, with a variety of functions that try to map physical damage and its functional consequence (e.g. hourly cost times driver's delay) into monetary terms. Even these functions, however, are affected by significant uncertainty (e.g. Franchin *et al.*, 2006a), and the use of pre-earthquake reference values (e.g. repair cost per unit of damage) is questionable in post-earthquake market conditions, when many damaged sites will compete for a fixed number of construction sector operators (demand surge), or government's political decision may alter completely the scenario (e.g. Calvi and Spaziente, 2009). Besides, the full economic impact of an earthquake evolves over a period of time that is longer than the current reference time-frame of the model. For these reasons economic loss is not yet included.

18.7 Probabilistic assessment of the model

18.7.1 Uncertainties in the problem

The model introduced in the previous sections describes a complex system of interconnected systems, the infrastructure, immersed in an environment and subjected to hazards. Uncertainty affects all elements of the model. Some uncertainties have already been addressed. The following is a complete list of uncertain elements:

- Seismic activity of the seismogenic sources/faults (modelled through magnitude-recurrence laws, probability distributions for epicentre locations or rupture area, etc.).
- Local seismic intensities at the sites (modelled through ground-motion prediction equations, spatial correlation models, cross-IM correlation models, and site amplification models).
- Physical damageability of the components of the infrastructure (modelled by fragility models for structural and non-structural components).
- Functional consequences of the physical damage.
- Socio-economic consequences of physical damage (modelled by probabilistic cost models).

Further uncertainty enters the model because alternative models are available for each of the above items, and the corresponding parameters are themselves characterised by statistical uncertainty. Hence, for instance, in the seismic activity model the sources' boundaries, or the parameters of the associated recurrence law, like lower and upper magnitudes, can be defined with some variability. Denoting with $\mathbf{x}$, the vector collecting all random variables employed to model the uncertainties in the problem, probabilistic assessment of the model requires the joint probability density function $f(\mathbf{x})$. The way this joint density is employed to assess performance metrics such as those described in Section 18.6.3 depends upon the adopted reliability method.

18.7.2 Available methods

Available reliability methods are either simulation-based or non-simulation-based. Both have been applied to the analysis of infrastructural systems. The model is designed not to take a specific position on this matter; thus an *Analysis* class is composed of both *Simulation* and *nonSimulation* abstract classes. Pragmatically, the implemented portion of the model includes only simulation-based methods, and due to the reasons to follow, no non-simulation-based methods are implemented. In particular, some of the promising methods/approaches include the so-called matrix system reliability method (MSRM) (Song and Kang, 2009; Song and Ok, 2010) and the model of bayesian networks (BN) (Jensen and Nielsen, 2007; Straub *et al.*, 2008; Der Kiureghian, 2009; Straub and Der Kiureghian, 2009; Bensi *et al.*, 2009).

The first method appears to reach a limit when dealing with capacitive (flow) modelling of networks. Applications, to the knowledge of the authors, are limited to connectivity problems, as in Kang *et al.* (2008). The second method seems very promising in perspective, especially for dealing with the

problem of real-time or near-real-time decision-making (analyst positioned within the time-frame of interest, as in Section 18.2). This is due to its capability of updating random variables and probabilities associated with system states real-time under a continuous stream of incoming information on the states of an enlarging set of components in the aftermath of an earthquake. Two problems, however, may severely hinder its application to problems of realistic dimensions.

The first one is the computational effort, which can be enormous since the system state must be known in advance for all possible combinations of the components' states. As it is frankly put in Der Kiureghian (2009):

After an earthquake, the [system] assumes one of $N = m_1 m_2 \dots m_n$ distinct configurations, where n denotes the number of components [and m_i is the number of possible states of component i]. For example a [system] with two-state components, has $N = 2^n$ distinct configurations. [...] For each of these distinct configurations one can perform disciplinary analysis (e.g., water, power, traffic connectivity or flow analysis) to determine the state of the infrastructure and whether or not it performs the intended function. ... It should be obvious that a [system] with a large number of components can have an extremely large number of distinct configurations. Therefore, disciplinary analysis will have to be performed for a very large number of cases. One of the challenges in infrastructure risk assessment is to devise methods to handle the large amount of computations.

The second problem is that the BN model itself is a directed acyclic graph, where nodes represent uncertainties (random variables) and edges describe their statistical dependence. The size and topology of this graph are related to those of the corresponding physical system. Therefore, its very construction, before even thinking of carrying out inference upon it, can be a daunting task for a problem of realistic dimensions. Hence, a way to automatically set up a BN given the infrastructure and hazards is needed for practical application.

18.7.3 Implemented simulation framework

The *Simulation* class currently has two sub-classes only: *MonteCarlo* and *ImportanceSamplingKmeans*. The former is a straightforward implementation of plain Monte Carlo (MC) simulation, while the latter is an implementation of the importance sampling method (ISKm) described in Jayaram and Baker (2010). This importance sampling method is quite effective for

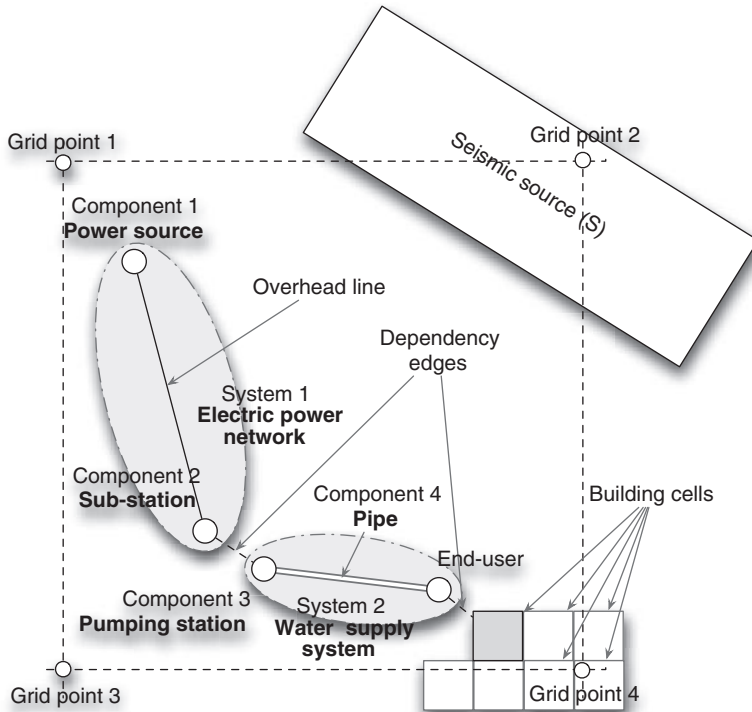
'developing a small but stochastically representative catalogue of earthquake ground-motion intensity maps that can be used for risk

assessment of spatially distributed systems. In this framework, Importance Sampling is used to preferentially sample “important” ground-motion intensity maps, and K-Means Clustering is used to identify and combine redundant maps in order to obtain a small catalogue. The effects of sampling and clustering are accounted for through a weighting on each remaining map, so that the resulting catalogue is still a probabilistically correct representation.’

The importance sampling density is the product of shifted Gaussian distributions for both the inter- and intra-event residuals η and ϵ , with shifts of the mean towards higher values that are given as a function of the system size and average separation distance amongst sites, and a stratified sampling density for the magnitude M . Together with the following clustering of the resulting maps, the overall effort is reduced from the tens of thousands simulation runs needed with MC to a few hundred runs.

In both cases the typical simulation run consists of sampling $\mathbf{x}$ from its distribution and evaluating the Infrastructure state and all the performance metrics of interest. Figure 18.10 shows a compact graphical representation of the variables in $\mathbf{x}$ and of their joint distribution, for the simple example infrastructure shown in Fig. 18.9. Random variables, or vectors, represented by circles, form the nodes of a graph where directed edges indicate statistical dependence. Models for flow-equations, etc. are pictorially represented as rectangles. Variables belonging to the same model are grouped with rounded rectangles. This representation is almost the same as that employed in BN models (Straub *et al.*, 2008), with the difference that here no Bayesian updating is performed and the figure simply illustrates the flow of the typical simulation run, from sampling of top (marginal) variables to that of the bottom (dependent) variables.

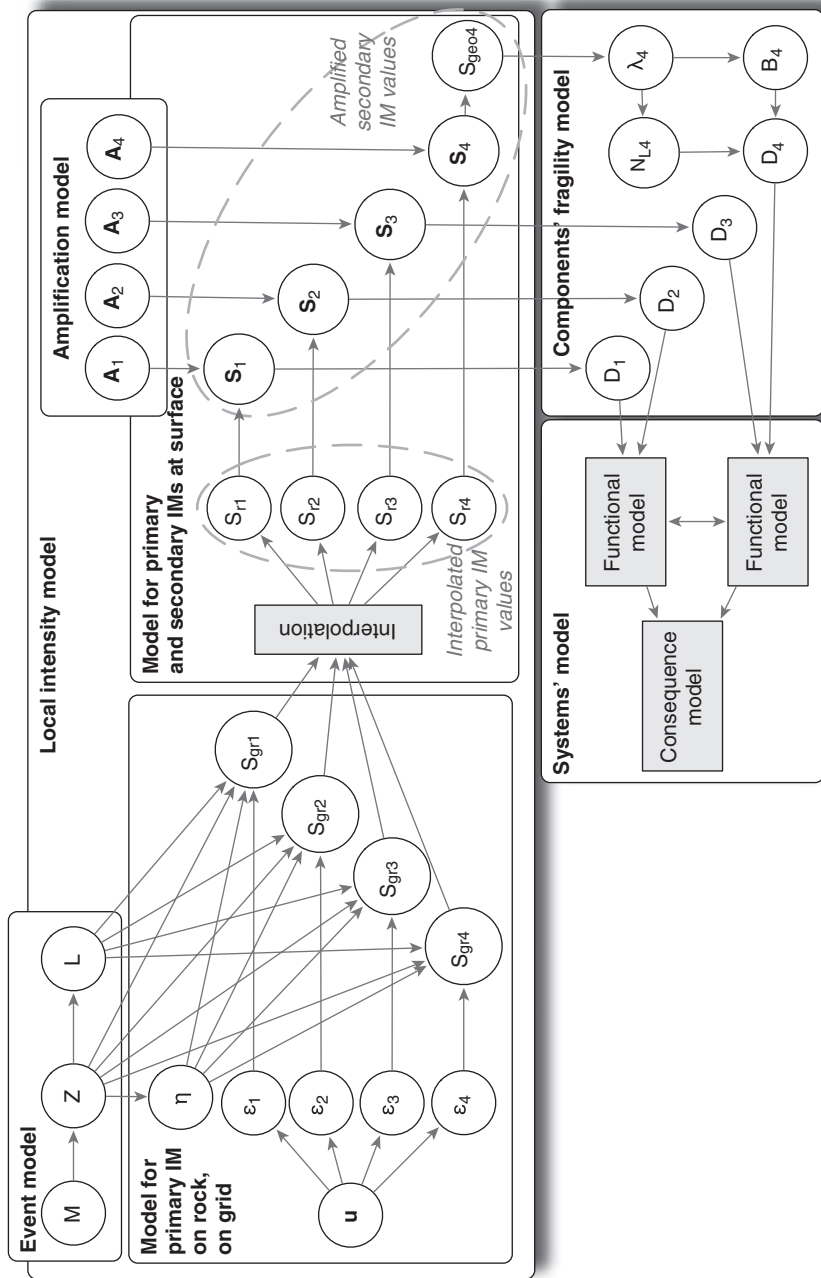
For the simple example shown in the figure, the run starts with the simulation of a magnitude value from the distribution of M . Conditional on the obtained magnitude value, the discrete random variable Z for the active seismogenic zone is sampled in the run (not all zones may be active for a given magnitude, as shown later in the application), and then the epicentre location within the zone. This completes the event generation step. The following steps (2 to 3 in Section 18.5.4) lead from the event macro-seismic characteristics to the local intensity at each site. First the variables η and ϵ (vector ϵ depends on the standard normal vector $\mathbf{u}$, see Section 18.5.4) are sampled to produce the primary IM values on the *grid* points for rock/stiff-soil conditions, denominated S_{grj} (j being the index spanning the grid points). Next these value are used to interpolate the S_{ri} values at each site (i being the index spanning the sites), still for rock/stiff-soil conditions. Finally, the vector $\mathbf{S}_i$ needed at each site is completed through conditional simulation of secondary IMs given the primary at the site S_{ri} , and component-wise



18.9 A simple infrastructure made up of two interconnected systems, each with two components. System 1 is an EPN with a generation node (point-like component 1) and a sub-station (point-like component 2) which provides power to a pump (point-like component 3) that inputs water into a pipe (line-like component 4) connected to a demand node that feeds water to the building in the dependent cells.

multiplication times the variables A_i sampled from their distribution at each site within the *amplification* model. As part of this process geotechnical IMs, such as, for example, permanent ground deformation (variable S_{geo4} in the figure), are sampled from a geotechnical hazard model for those components whose fragility model requires one, such as for instance the pipe in Fig. 18.9 (component 4).

The simulation run then proceeds with the assessment of the physical *damage* state of each component, represented by variables D_i , sampled from their respective fragility models. These can be of two types. Point-like components have fragility models consisting of a set of fragility curves associated with limit-states of increasing severity. The curves express the probability of exceedance of each limit-state conditional on the local seismic intensity. Determination of the damage state D_i is performed by sampling



18.10 Graphical representation of the typical simulation run: circles represent random variables, rectangles represent 'analytical/numerical models', arrows represent statistical dependence among random variables or input/output to/from a model. The graph represent the uncertainty in the simple infrastructure of Fig. 18.9.

a standard uniform number and checking between which curves it falls, at the intensity sampled for the current run. For line-like components, such as component 4 in Fig. 18.9, the fragility model usually consists of a Poisson distribution giving the number of damages per unit length, with rate (λ_4 in Fig. 18.10) function of the local intensity. Sampling the damage number and severity (breakage probability is also a function of λ_4) is performed from the Poisson distribution.

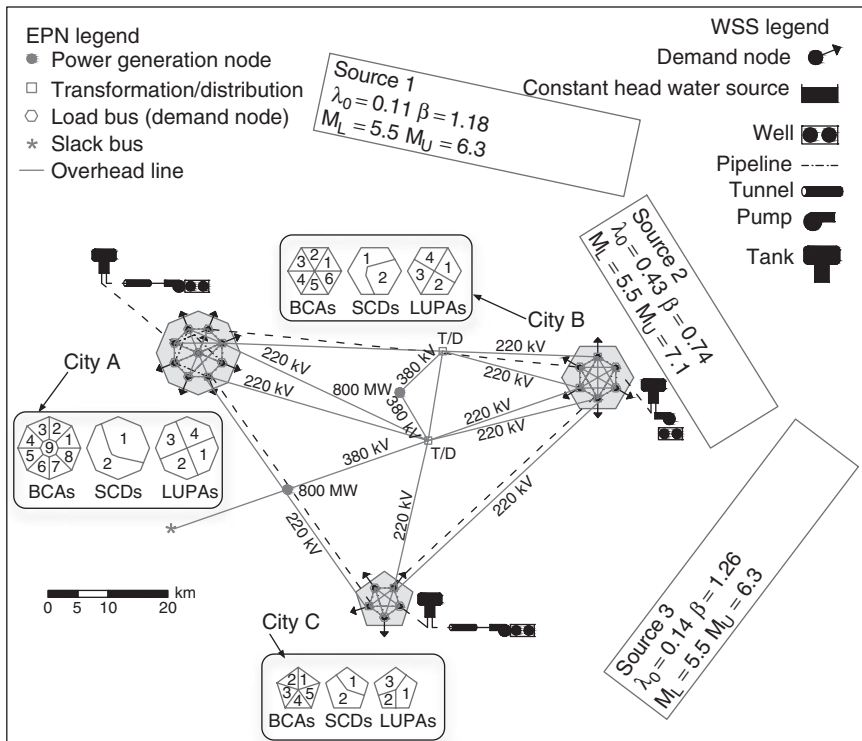
When the damage states of the four components are known, the functional models of the two systems are used to evaluate the functional consequences. At this stage, performance metrics (PI_c, PI_s, PI_{inf}) for components, systems and the whole Infrastructure can be evaluated.

18.8 Example of an application of seismic vulnerability analysis

18.8.1 Description of the study area

This section presents some results of a simple application of the described model to the hypothetical small region already shown in Fig. 18.3 and Plate I (between pages 452 and 453). Figure 18.11 shows the region, which has three urban areas, denominated cities A to C, of which the easternmost is closest to the three seismogenic zones. Each city is subdivided in a different number of building census areas, sub-city districts and land use plan areas, numbered according to the figure. These sets of polygons are associated with data, reported in Table 18.1, ranging from land use, to population, to building typology and number. For this simple example only two broad buildings categories are considered, roughly representative of Italian pre-WWII and post-WWII construction, denominated unreinforced masonry (URM) and reinforced concrete (RC) buildings.

The seismic fragility model for the two typologies is provided in the form of a set of two fragility curves (one for the *yield* and the other for the *collapse* limit state). These curves have been derived as part of the SYNER-G project by statistical post-processing of multiple fragility studies: different curves have been attributed to classes in the mentioned taxonomy, and harmonisation of limit-state definition and intensity measure has been performed (all curves are expressed in terms of PGA), coming up with lognormal shapes specified in terms of their two parameters, complemented with a characterisation of the epistemic uncertainty on the latter (for each typology a joint distribution of the log-mean and log-standard deviation of each limit-state is available). Details can be found in Crowley (2011). Table 18.2 reports the employed parameters. The final model of the buildings consists of a set of 181 rectangular cells, as already shown in Fig. 18.3 and Plate I



18.11 Study region employed to exemplify the presented model. Three sources affect the three cities, which are interconnected by lifeline networks such as the EPN and WSS shown in the figure.

(between pages 452 and 453), of which 86 are vulnerable, in the sense that contain buildings of either one or both of the above typologies.

Infrastructural systems

The three cities are served by two utility networks, an EPN and a WSS. The EPN consists of 25 nodes, including three non-vulnerable nodes, i.e. two generators and the slack bus. There are a total of 34 buses; this number is larger than the number of nodes because in transformation/distribution stations there are two buses, one on each side of the transformers at different voltages. This is because sub-stations are modelled in detail with all their internal components in order to capture SCs, damage propagation, and intermediate operational states. A total of 67 edges completes the network model (including the edges that are used to connect buses within stations). The two power generation nodes, linked to transformation/distribution sta-

Table 18.1 Number of buildings per Building Census Area (BCA), population per urban audit sub city district (SCD) and land use per Land Use Plan Area (LUPA), the latter with codes: 'I' industrial, 'C' commercial, 'R' residential, 'G' green

City	BCA	N_{URM}	N_{RC}	SCD	N_{inhab}	LUPA	Use
A	1	17 000	25 500	1	163 000	1	I
	2	19 215	23 485	2	144 000	2	C
	3	12 600	29 400			3	G
	4	34 400	8 600			4	R
	5	30 000	10 000				
	6	31 570	9 430				
	7	29 400	12 600				
	8	8 540	34 160				
	9	6 675	20 025				
B	1	22 500	22 500	1	124 000	1	C
	2	30 100	12 900	2	102 000	2	R
	3	33 000	11 000			3	I
	4	34 000	8 500			4	G
	5	19 620	23 980				
	6	13 410	31 290				
C	1	18 000	18 000	1	37 600	1	R
	2	14 600	21 900	2	56 400	2	G
	3	27 750	9 250			3	C
	4	24 920	10 680				
	5	15 480	18 920				

Table 18.2 Parameters of the lognormal fragility curves for the two limit states and the two considered building typologies

Parameter	URM		RC	
	Yield	Collapse	Yield	Collapse
$\mu_{\log}$	-2.041	-1.269	-1.832	-1.091
$\sigma_{\log}$	0.574	0.550	0.474	0.485

tions with high voltage (380 kV) lines, collectively supply 1600 MW of power. The slack bus, also called balance node, supplies the power imbalance between the generation and the demands. Smaller voltage (220 kV) lines then convey energy to the urban areas, where they are further reduced

to medium voltage (60 kV) lines. In this simple application low voltage lines are not modelled and power demands are aggregated to higher level sub-stations (the load buses in the figure).

The water supply system is made up of three local urban systems interconnected by three pipelines. This topology is quite unusual. In order to highlight interactions, the WSS is made artificially weak by providing only variable-head water sources in the form of tanks served by pumping stations. Each of these pumping stations can then be out of service as a result of an EPN failure to provide power at the reference sub-station of the pump. The well–pump–tank systems are dimensioned to cover the necessary water requirements under normal operational conditions. The network model comprises a total of 37 nodes, including three source nodes, and 53 edges.

In terms of fragility models, vulnerable components in the EPN are all the sub-station components, as described in Vanzi (1996). Overhead lines are not vulnerable. Notably, failure in one sub-station can propagate to neighbouring stations if circuit-breakers are also damaged. In the WSS, only pipelines are vulnerable and the fragility model is given in terms of a Poisson repair-rate per kilometre:

$$\lambda_{\text{repair}}(PGV) = K_1 0.0024 PGV \quad [\text{km}^{-1}] \quad [18.16]$$

where peak ground velocity (PGV) is in cm/s and K_1 is a function of the pipe material, soil and joint type, as well as diameter. The model, described in ALA (2004), includes a mean rate of repair due to permanent ground deformation, with the indication that the larger of the two rates should be used to predict damage. The second rate has not been used in this simple example where rock/stiff-soil conditions and no susceptibility to liquefaction/landslide phenomena are considered over the entire region.

Regional seismicity model

Seismic activity in the region is caused by the three sources shown in Fig. 18.11. The sources are of the area type, with epicentre location distributed uniformly within each source. Their activity is described through a truncated Gutenberg–Richter model, and the corresponding parameters are also reported in the figure: sources 1 and 2 have similar activities and the same lower and upper magnitudes. Source 2 is both more active, with a rate of events λ_0 three to four times larger than the other two sources, and characterised by a larger upper magnitude.

For the purpose of predicting local seismic intensity, PGA is taken as the primary IM, which coincides with the input IM for the buildings' fragility models at the location of grid cells centroids as well as for EPN sub-station components (transformers, buses, etc.). At the location of pipe segment

centroids, PGV is used as a secondary IM as an input to pipes' fragility model. Both simulation of PGA and the conditional simulation of PGV are carried out using the GMPE by Akkar and Bommer (2010), recently developed for the European context. Spatial correlation with correlation parameter (range) is estimated based on European records by Esposito *et al.* (2010). For within-site cross-IM correlation $\rho_{PGA,PGV} = 0.754$ is used. No site amplification, nor geotechnical hazard, is considered for this simplified example. The rectangular grid for the prediction of the primary IM consists of $71 \times 61 = 4331$ points.

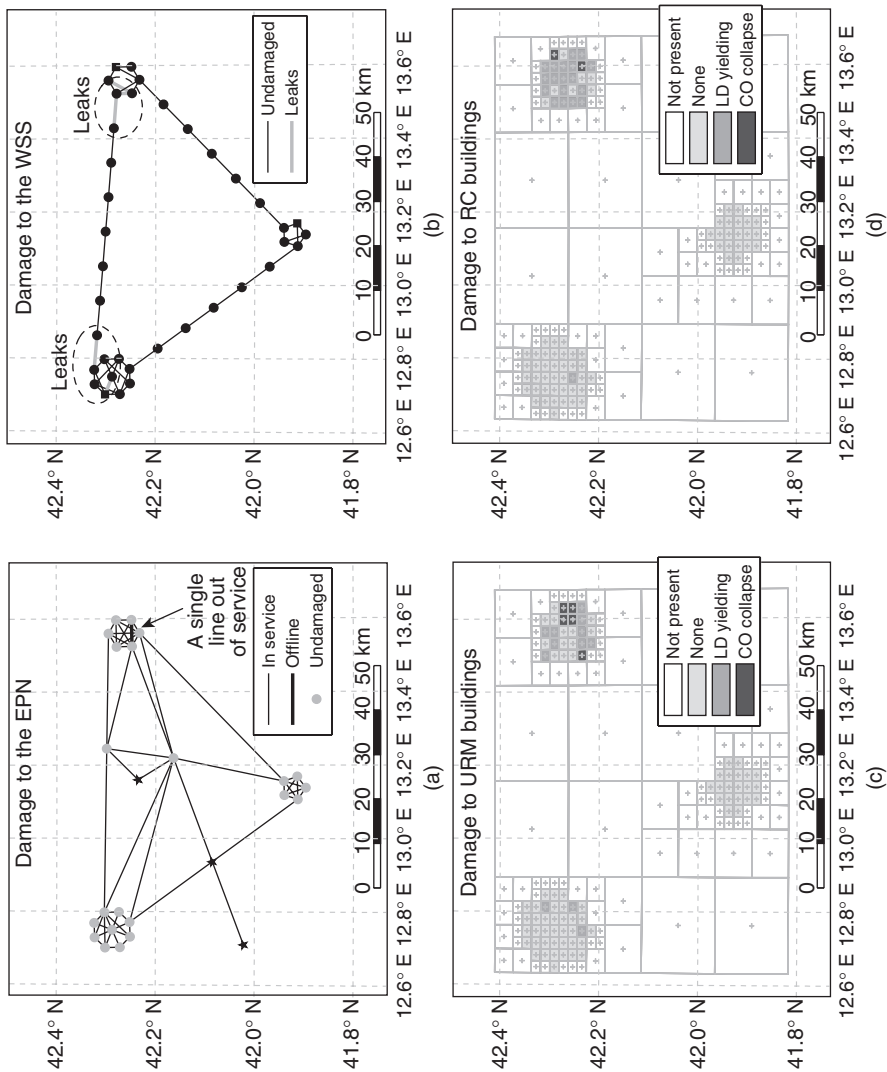
18.8.2 Results for a single scenario event

The impact of seismic events on the considered region is assessed through a simulation carried out with the importance sampling method by Jayaram and Baker (2010), without performing, however, the K-means clustering. A total number of 1000 runs has been carried out. This section describes results of one of these runs, i.e. the impact of a single 'scenario event', a magnitude 7 earthquake on source 2 (for this magnitude this is the only source that can generate the event, as shown in Fig. 18.14), quite close to city B. The corresponding shake-field has already been shown in Plate I (between pages 452 and 453) and Fig. 18.7.

Physical damage caused by the event is illustrated in Fig. 18.12. Plot (a) shows damage to the EPN. In general this system tends to be 'stiffer' than the WSS, with damage states mostly resulting in either a functional or not functional network. This is particularly true for this very small system lacking much redundancy. The shown scenario, however, is a relatively rare one in which damage occurred in a sub-station (in city B, the one closest to the epicentre) but circuit-breaker prevented its propagation within the station and to other stations, resulting only in a single line being off-line. Under these conditions the system of equations in [18.2] could still be solved for an equilibrium condition, though with lower voltages than needed in many stations. Plot (b) shows damage to the WSS. In this scenario there is no pipe rupture but leaks occur at more locations, in cities B and A.

Plots (c) and (d) show damage to URM and RC buildings, respectively. In the maps, a shade of increasingly darker grey, which is proportional to the damage state, is assigned to cells. White cells are outside urban areas and have no buildings. Damage is mostly limited to city B, with the larger number of collapsed buildings in the URM typology, as expected, though light damage (exceedance of the yield limit state) is widespread among RC buildings.

As observed in Cavalieri *et al.* (2012), where more details are given on the way large groups of buildings in urban areas are lumped into cells within



18.12 Physical damage to buildings and lifelines for the considered scenario: (a) damage to the EPN; (b) damage to the WSS; (c) damage to the URM buildings; (d) damage to the RC buildings.

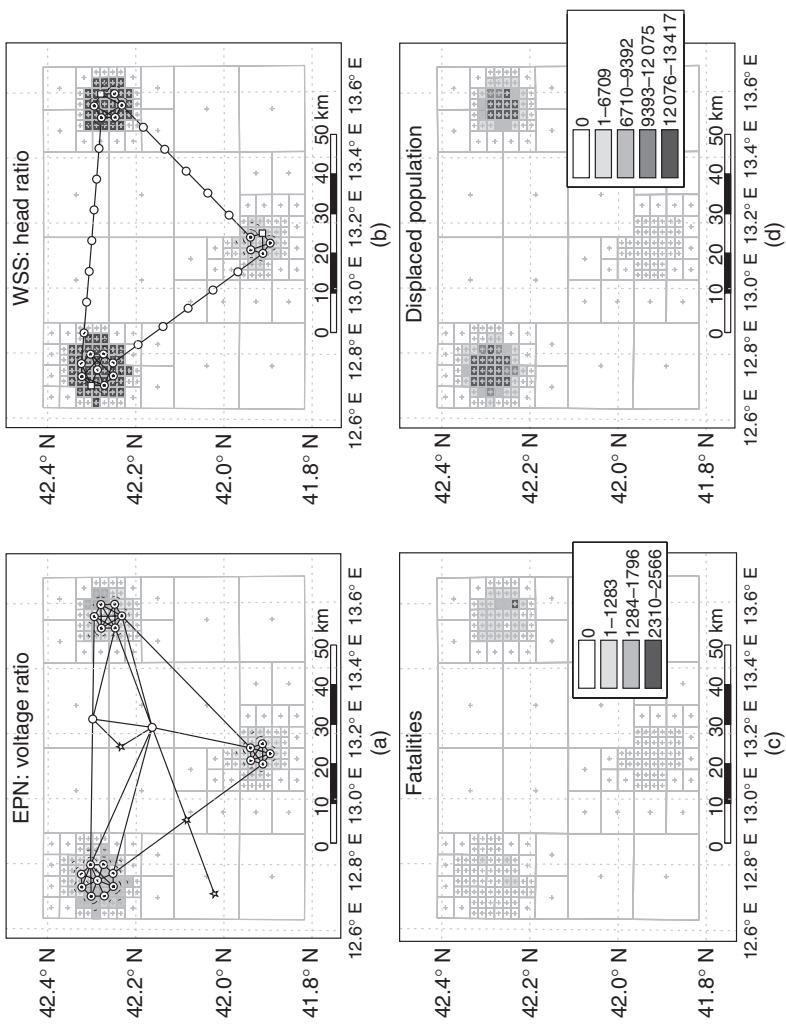
the presented model, the adopted meshing solution has several advantages and it is also convergent to the individual building modelling upon mesh refinement. However, attributing the sampled physical damage state to *all* the buildings of a given type within a cell can lead for coarse meshes, as in this simple example, to an overestimation of the likelihood of extreme damage states. Similarly, plot (b) shows how the long pipelines connecting the three urban areas are automatically discretised into smaller segments, in a relatively rough manner. Large values for maximum pipe segment length employed in the discretisation will result in biased estimates of extreme damage states. Caution should therefore be used in selecting these meshing parameters when looking for a good compromise between computational effort and accuracy of the results.

The functional and, ultimately, the social consequences of the above state of physical damage are illustrated in Fig. 18.13. The reduced service level in the EPN and WSS can be measured with different metrics. Plots (a) and (b) provide maps of the voltage and water head ratios VR and HR. These quantities, computed in the demand nodes of the two networks, are represented on the tributary cells, which are hatched with a shade of increasingly darker grey corresponding to lower values of either VR or HR. Given the relatively strict tolerance on the voltage of supplied electric power, the darker grey areas in both cities B and A, where $VR < 0.9$, can easily be associated with a likely failure in dependent systems. This indeed is what happens in the WSS, where cities B and A have no water at all (black cells), since in both cases the pumping stations had EPN reference nodes located in the $VR < 0.9$ areas. This loss of functionality affects the number of displaced people, shown in Fig. 18.13(d). The pattern of plot (b) is repeated, with the populations of cities B and A almost entirely displaced as a result of a severe utility loss. Fatalities instead, being just a function of direct physical damage to buildings, are mostly located in city B, where damage is non negligible, as already shown in Fig. 18.12(c). More details on these aspects can be found in Cavalieri *et al.* (2012).

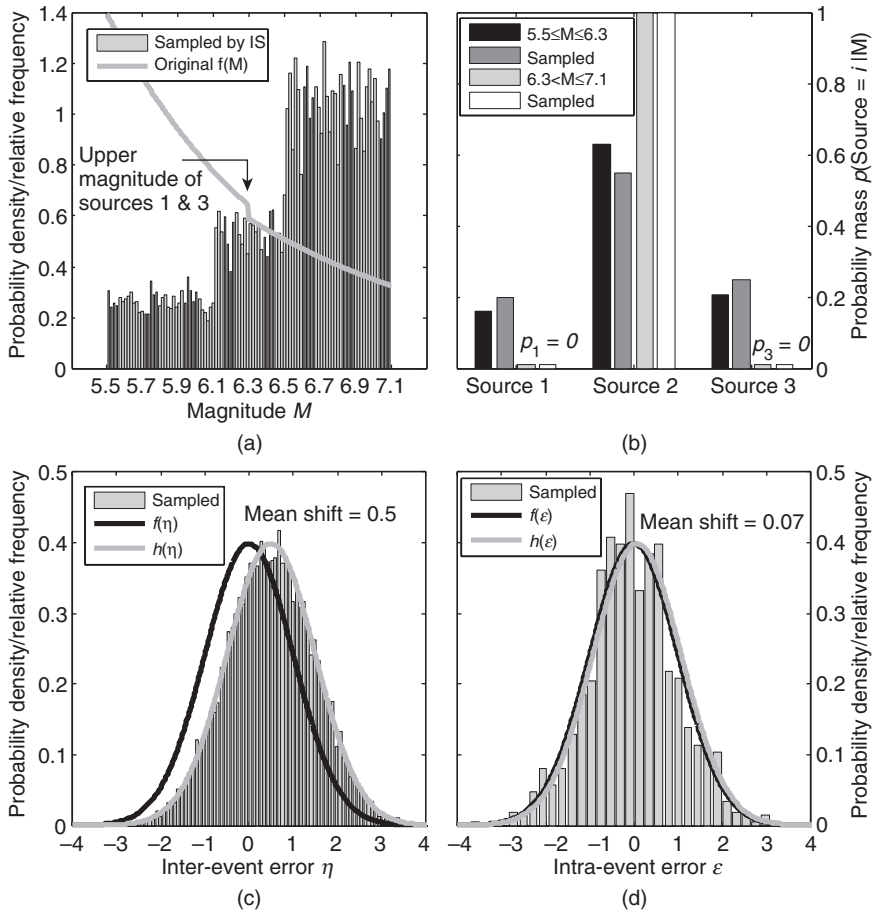
18.8.3 Results for the whole simulation

This section presents consolidated results obtained from the entire simulation. Figure 18.14 illustrates the seismic hazard. In particular, plot (a) compares the histogram of relative frequency of the sampled magnitude values, with the original probability density obtained as a weighted average of the truncated exponential distributions relative to each source:

$$f_M(m) = \frac{\sum_{i=1}^3 \lambda_{0i} f_i(M = m)}{\sum_{i=1}^3 \lambda_{0i}} \quad [18.17]$$



18.13 Functional consequences of the physical damage and social impacts for the considered scenario: (a) voltage ratio, function of direct physical damage to EPN stations; (b) head ratio (tributary cells of the same demand node share the same value), which is a function of direct physical damage to pipelines and indirect loss of function caused by EPN failures; (c) fatalities, function of direct physical damage to buildings; (d) displaced population, function of direct damage to buildings and of utility loss.



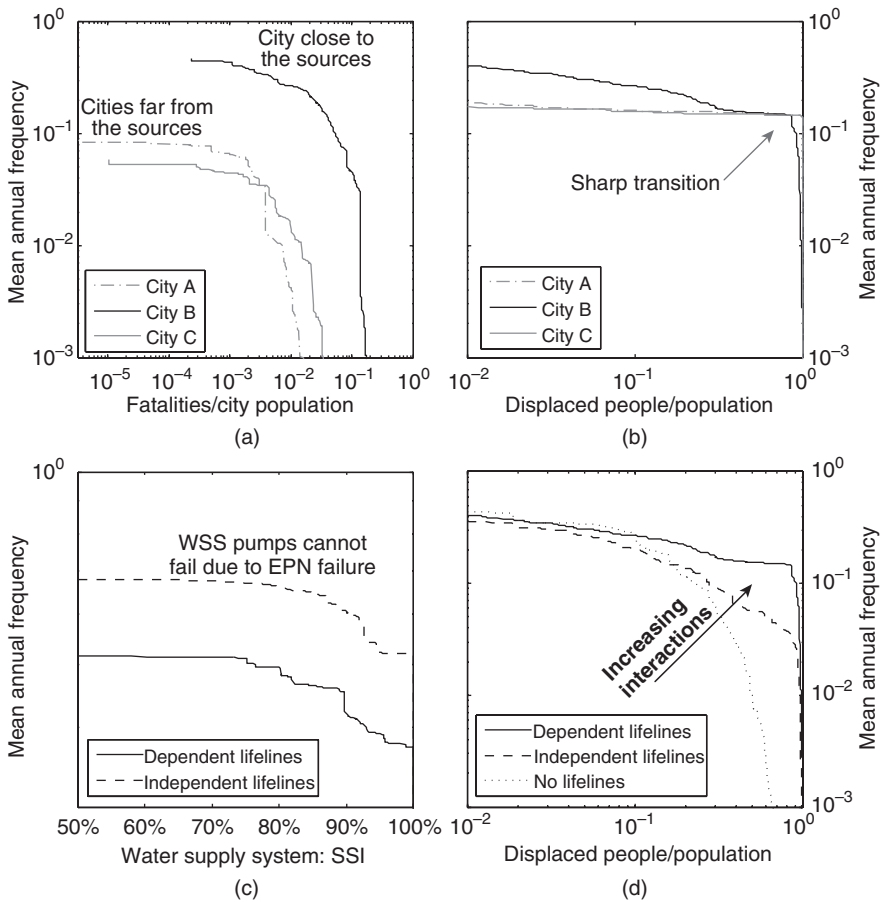
18.14 The regional seismicity: (a) original probability density for the magnitude versus the histogram of relative frequency of the magnitudes sampled by IS; (b) conditional probability of seismic zone given magnitude; (c) original probability density versus histogram of sampled values for the inter-event error η ; (d) original probability density versus histogram of sampled values for the intra-event error ϵ at a single site.

The plot shows how the density f_M has a step corresponding to the common upper magnitude $M_U = 6.3$ of sources 1 and 3. The sampled values on the other hand have an inverted trend with adequate coverage of the most important larger magnitude events. Plot (b) compares the target and sampled conditional distributions of the discrete variable Z (see Fig. 18.10, where an arrow from M to Z indicates statistical dependence), for values of M in the 5.5 to 6.3 and the 6.3 to 7.1 ranges. While in the first case any of the three sources can generate the event, and probabilities reflect the

relative activity rates, in the second case only source 2 can generate the event and $p_1 = p_3 = 0$.

Plots (c) and (d) compare the original (standard Gaussian) density for the inter- and intra-event residuals, with their shifted IS densities and the histogram of sampled values. The indicated mean shifts values are very consequential in obtaining good results and have been chosen following the indications in Jayaram and Baker (2010).

Finally, Fig. 18.15 shows mean annual rates of exceedance for some performance metrics of interest. Plot (a) refers to fatalities. Mean rates are



18.15 Complete simulation: mean annual rate of exceedance of (a) fatalities and (b) displaced population, as a ratio of total population, by city; different mean annual rates of exceedance obtained considering various levels of interaction of (c) system serviceability index and (d) normalised displaced population (curves refer to city B).

given separately for the three cities, and presented in terms of fatalities as a fraction of total population. City B, as expected, has the highest rate. All curves present a smooth behaviour. This is not the case for the MAFs of displaced population, shown in plot (b). Similarly to the previous plot, rates are given by city and in terms of the ratio of displaced to total population. All cities, but in particular cities A and C, present a sharp transition close to the 100% value. This is due to the fact that, as already seen in Fig. 18.13(d), displaced population is influenced by utility loss and for this simple example, the EPN is either fully functional or completely non-functional. In the latter case, independently of direct damage, buildings, even usable ones, become non habitable and occupants are forced to evacuate. This is more apparent in cities A and C where direct physical damage is less likely due to the larger distance from the seismogenic sources.

To gain further insight into this effect of interdependencies captured by the model, plot (d) shows the MAF of normalised displaced population for the most affected city B under different modelling assumptions. The solid curve is the same shown in plot (b), i.e. the case where utility loss is considered, and the lifelines are dependent (WSS on EPN). The dashed line shows what would happen if utility loss was still considered in the evaluation of building habitability, but the interaction between the two systems was disregarded. The MAF would consistently decrease, and, most importantly, the sharp kink at around 100% would be attenuated. This attenuation would be due to the reduced vulnerability of the WSS in case the possibility of EPN-induced failure of the pumps was removed. This is clearly shown in plot (c) where the MAF of the water supply system serviceability index is shown in the two cases. The kink in the displaced people MAF would completely disappear and the curve would more closely follow the trend of the fatalities MAF in plot (a) if displaced people were computed based only on direct physical damage (the ‘no lifelines’ case).

18.9 Future trends

The development of the presented model is still ongoing. Considerable research efforts are aimed at completing the implementation of the taxonomy of systems in Fig. 18.2, with particular attention to critical facilities. Future research will focus on methods to efficiently compute sensitivity measures within the simulation framework, on one hand, and on introducing Bayesian inference in the *Analysis* class on the other.

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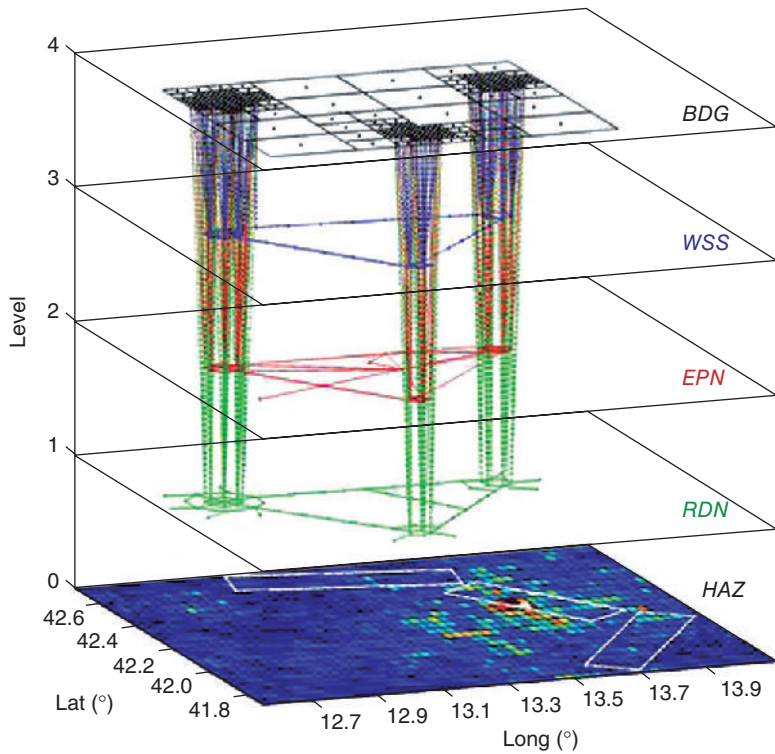


Plate I Graphical representation of the super-graph comprising all systems with their dependency edges. Components that are the same or close sites exhibit geographical interaction through common or closely dependent seismic intensity, shown in the bottom HAZ layer as a ground motion shake field. This shake field is the same also shown in Fig. 18.7, and results reported in the example application of 18.8 in terms of single scenario damage maps refer to this same event.

Seismic reliability of deteriorating reinforced concrete (RC) bridges

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Abstract: The chapter discusses different deterioration mechanisms that affect the performance and reliability of bridges. Focus is on reinforced concrete (RC) and post-tensioned (PT) bridges. First, a general discussion is provided on different forms of deterioration along with an overview of the underlying physical phenomena and their influencing factors, which include material properties and environmental conditions. Then, the chapter focuses specifically on corrosion mechanisms and provides a quantitative assessment of the effects of corrosion on the performance and reliability of RC bridges. Finally the chapter suggests future directions on the topics presented.

Key words: corrosion, deterioration, reliability, bridge.

19.1 Introduction

Structures are designed such that they meet some minimum level of safety. The American Association of State Highway and Transportation Officials (AASHTO) provides guidance on safety factors and reliability requirements for both newly designed and in-service bridge structures. Although significant research has been performed to assess the effects of deterioration of materials, less research has been performed to assess the influence of this deterioration on structure performance. Structures, and more specifically reinforced concrete (RC) and post-tensioned (PT) bridge structures, are exposed to various environmental conditions. These environmental conditions can lead to deterioration of the concrete and/or reinforcement. This chapter provides:

- a review of the mechanisms of deterioration that can occur in RC and PT structures;
- a review of types of structural damage resulting from the deterioration mechanisms, as the type of damage will provide pertinent information necessary to compute time-variant reliability;
- a discussion on the time-variant reliability of bridges; and

- A description of preventative measures and repair strategies, and future trends.

Significant research has been performed to better understand the mechanisms of deterioration. However, limited research has been performed to assess the structural performance of systems exhibiting damage from these deterioration mechanisms. This chapter will provide a brief overview of deterioration mechanisms followed by an overview of the time-variant performance of bridge structural systems. Although bridge structures were identified as the structure of choice in this chapter, the process can be applied to other systems.

19.2 Mechanisms of deterioration

Although the majority of bridges are durable and safe, when bridges are exposed to aggressive environments, performance and safety of the structures can be reduced. In addition to the environment, design parameters, material types, material quantities, and construction practices all impact the rate at which the concrete or reinforcement can deteriorate. The majority of deterioration in RC structures results from chemical reactions that occur between the material and environment or as a result of a phase change in the material. Corrosion (American Concrete Institute [ACI] 222R-01), alkali-aggregate (or silica) reactions (AAR or ASR), and sulfate attack (ACI 201.2R-08) are examples of chemical reactions that occur between a material and an aggressive compound in the immediate environment. Freeze-thaw damage (Cai and Liu 1998) and delayed ettringite formation (DEF) (Scherer 1999; Flatt and Scherer 2008) are examples of a phase change occurring within the material.

Although sulfate attack could be present in a variety of structures and environments, it is often associated with structures other than bridges. In addition, very limited work has been performed on assessing the impact of sulfate attack on structural performance. Because of this, this mechanism of deterioration will not be addressed herein. The literature does contain a vast amount of information on the mechanisms of corrosion of the steel reinforcement and how this process impacts the surrounding cementitious material (ACI 222R-01; Jaffer and Hansson 2009; Zhang *et al.* 2010). Less work has been performed on assessing the structural performance of corroding RC and PT structures. ASR and freeze-thaw damage have been identified in a large number of structures and DEF has been identified far less. As with corrosion, the impact of these deterioration mechanisms on structure performance has not been thoroughly investigated. The following sections will provide a general discussion on the mechanisms of deterioration commonly associated with RC and PT

structures. Following the review on the deterioration mechanisms and resulting damage, an assessment tool to quantify capacity and reliability loss associated with these deterioration modes is presented. In addition, an example is provided.

19.2.1 Corrosion of steel in concrete

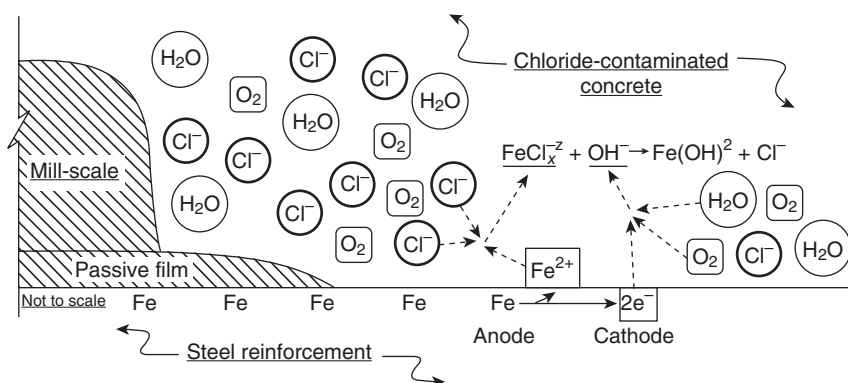
The mechanisms of corrosion have been widely reported throughout the literature and a comprehensive literature review is beyond the scope of this chapter. For more comprehensive information, the authors direct the reader to the American Concrete Institute's Committee 222 documents. Corrosion is the electrochemical reaction between a metal and its environment. For RC structures, the cementitious material (often concrete) provides a protective barrier against the aggressive elements in the surrounding environment. However, these aggressive elements can penetrate the protective concrete barrier and react with the reinforcing steel. When the reinforcing steel corrodes, a corrosion product is formed that is larger in volume than the original base steel. This corrosion product forms within the concrete voids and pores, placing the concrete in tension. Because cementitious materials are weak in tension, the formation of corrosion products commonly results in cracking and spalling of the cementitious cover. Once cracked or spalled, the protective capacity of the cementitious cover is further reduced, making the reinforcement even less protected and more susceptible to further deterioration.

Chlorides accelerate the corrosion process. Chlorides can be introduced to the concrete from exposure to saltwater, deicing or anti-icing chemicals, or can be introduced to the concrete as part of the constituent materials. Chloride-induced corrosion of steel reinforcement requires that sufficient chlorides be present at the steel–cementitious materials interface and that both oxygen and water are available (González *et al.* 1993; López and González 1993; Balabanić *et al.* 1996; Hussain *et al.* 2012). The availability of sufficient quantities of these elements or compounds will result in active corrosion of the steel reinforcement.

The mechanisms of chloride-induced corrosion have not been thoroughly defined. However, at locations where the mill-scale is discontinuous, a passive film likely exists. Passive films have been reported to be thin layers of oxides that occupy the reactive atom sites on the metal surface. By occupying these reactive sites, the film prevents metal atoms from reacting with the ions in the environment, thus preventing the corrosion process from initiating. Ghods *et al.* (2011a, b) showed that the passive film on steel in highly alkaline environments is approximately 5 nm (2×10^{-4} mils) thick and is composed of a thick, porous outer layer over a thin, dense inner layer. The inner protective layer was reported to be mostly composed of Fe^{2+}

oxides and contained relatively smaller amounts of Fe^{3+} oxides. The outer layer contained relatively larger amounts of Fe^{+3} oxides than the inner layer. However, this film is also likely heterogeneous and discontinuous. Jones (1982) reported that corrosion initiation is likely a result of the electrostatic attraction between the positively charged metal surface (without an oxide layer) and the negatively charged chlorides. Hime and Erlin (1987) reported that the chlorides react with the unprotected ferric and ferrous ions to first produce ferrous chloride. The ferrous chloride then reacts with hydroxyl ions to produce iron hydroxide ($\text{Fe}(\text{OH})_2$), freeing the chlorides to further react with more ferric and ferrous ions. Figure 19.1 shows a general schematic of the corrosion process. Note that water is converted to hydroxyl ions and the chlorides are released to further react with the base steel. This indicates that once chlorides are transported to the steel reinforcement surface, they are always available for further corrosion reactions. It also shows that the availability of water is necessary for the reaction to continue. Note that the rate at which the chlorides are transported to the steel surface is a significant parameter for determining the time to corrosion initiation and continued propagation.

In addition to chloride-induced corrosion, corrosion of the steel reinforcement can initiate as a result of carbonation of the concrete. Carbonation of the concrete occurs when the alkalis in concrete react with acidic components of the atmosphere, particularly carbon dioxide (CO_2). Carbonation results in progressively reduced alkalinity of the concrete. Because the stability of the passive film is dependent on high alkalinity, the protection from the passive layer is reduced with decreasing concrete alkalinity, resulting in increased corrosion activity of the reinforcement. Unlike chloride-induced corrosion where localized attack is common, carbonation-induced corrosion is more associated with general or uniform corrosion.



19.1 Simplified schematic of chloride-induced corrosion.

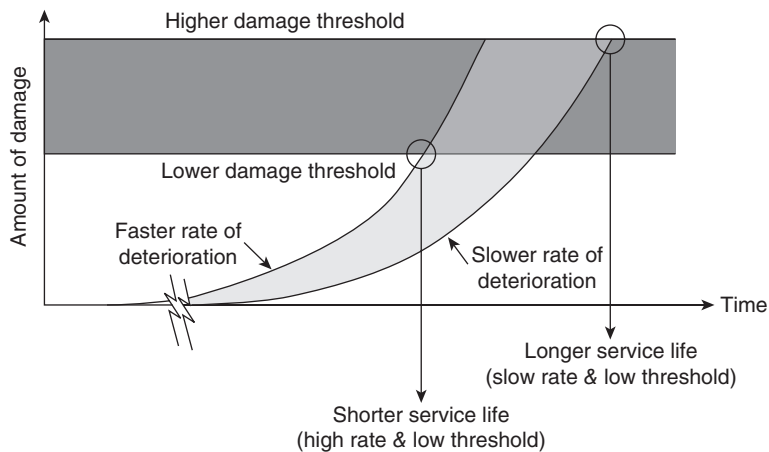
Uniform corrosion is more predictable and often easier to model and mitigate. Although carbonation-induced corrosion has been identified in many structures, chloride-induced corrosion is generally more prevalent (ACI 201.2R-08).

19.2.2 ASR

Another common mechanism of deterioration in concrete structures is ASR. A comprehensive assessment of the mechanisms of ASR is beyond the scope of this chapter and additional information on the mechanisms and damage can be found in Glasser and Kataoke (1981) and Folliard *et al.* (2006). ASR was first reported by Stanton (1940) and like corrosion, generates a product of larger volume that causes tension in the concrete. ASR requires three conditions to occur: available alkalis in the pore solution, reactive silica in the aggregate, and sufficient moisture in the concrete. When these conditions are present, a chemical reaction between the reactive silica in the aggregate and the alkalis in the pore solution occurs, producing an ASR gel in and around the aggregates. Because aggregates are uniformly distributed throughout the concrete, cracking typically propagates from aggregate surfaces resulting in cracking throughout the concrete – this cracking is often referred to as map-cracking. However, the type of major cracking depends on the type of structure, reinforcing details, and loading conditions. In addition, because the alkalis are often limited in the concrete, over time the ASR tends to slow down because the alkalis are consumed in the chemical reactions. Unlike chloride-induced corrosion where the propagation is continuous from one surface, ASR propagation decreases with time and occurs at many surfaces. Cracking due to ASR typically results not in spalling but in cracking and direct access of aggressive substances to the reinforcement surface. Although limited research has been performed on how ASR influences the capacity of a structure, it has been established that ASR can result in early corrosion of the reinforcement due to the premature cracking. Unfortunately, limited research has been performed on the synergistic effects of different deterioration mechanisms and research is sorely needed.

19.2.3 Deterioration and structure service life

Service life is defined as the time between when a structure is placed into service and removed from service. The service life of a structure may be reduced due to deterioration, variable environmental demands, increasing load demand, or an event that causes damage to the structure such that it can no longer be used for its intended purpose. What is important to note is that with the progress of structural deterioration, structural capacity is



19.2 Schematic showing structural damage and resulting service life for various conditions.

often reduced, thus increasing the potential for damage and/or failure. Figure 19.2 shows a typical plot of progressive structural damage over time. The figure shows that the rate of deterioration or change in damage threshold can influence the service life. In addition, as the deterioration and resulting damage increase, reliability of the structure decreases. The effects of deterioration on the reliability of structures; more specifically, issues related to bridges will be addressed in the following sections. Note that the environment in which the structure is placed, the materials used to construct the bridge, the proportions of these materials, design parameters (e.g., concrete cover), and construction practices are uncertain parameters that can all affect the deterioration rate and service life. As such, use of probabilistic methods for assessing service life is most appropriate.

19.3 Effects of deterioration on the reliability of bridges

The ability of a bridge to sustain seismic hazard can be quantified in terms of the conditional probability that a specified performance level is met for given measures of seismic loading. Such probability can be expressed mathematically as the probability that structural demands associated with the seismic event are less than the corresponding capacities of a bridge and of its components (Gardoni *et al.* 2002, 2003; Wen *et al.* 2003). Demands include forces, moments, and responses (like deformations, drifts, ductilities) in the bridge or its components associated with the seismic loading. Capacities are expressed in terms of the counterpart measures/quantities that the bridge or its components can sustain meeting a specified performance

level. Deciding which demands and capacities should be considered is based on an understanding of the possible modes of failure of a bridge and of its components. This section first describes the effects of deterioration on the structural capacity and the seismic demand on RC bridges. Then, it discusses the effects of deterioration on the seismic reliability of RC bridges.

19.3.1 Effects of deterioration on structural capacity of, and seismic demand on, RC bridges

Formulation of probabilistic models

Gardoni *et al.* (2002) proposed the following general form for a probabilistic model of the capacity at time t of a structural component i for a specified failure mode k :

$$C_{ik}(\mathbf{x}_0, \Theta_{Ck}, t) = C_{ik}(\mathbf{x}_0, \theta_{Ck}, t) + \sigma_{Ck} \varepsilon_{Ck} \quad [19.1]$$

where $\mathbf{x}_0$ is a vector of basic variables that includes material properties and geometry at time $t = 0$; $\Theta_{Ck} = (\theta_{Ck}, \sigma_{Ck})$ is a vector of the model unknown parameters, in which $\theta_{Ck} = (\theta_{Ck1}, \dots, \theta_{Ckn})$; $\sigma_{Ck} \varepsilon_{Ck}$ is the model error that captures the inaccuracy of the model in predicting the actual capacity due to an inexact model form or missing variables; ε_C is a standard random variable (i.e., with zero mean and unit variance); and σ_{Ck} is the standard deviation of the model error. For given values of $\mathbf{x}_0$, θ_{Ck} , and σ_{Ck} , the variance of the probabilistic model can be written as $\text{Var}[C_{ik}(\mathbf{x}_0, \Theta_{Ck}, t)] = \sigma_{Ck}^2$.

The term $C_{ik}(\mathbf{x}_0, \theta_{Ck}, t)$ can be made of a common deterministic model that captures the current understanding of the physics/mechanics of the deterioration, and an additional correction term that capture the systematic errors (bias) and inherent uncertainties. Developing $C_{ik}(\mathbf{x}_0, \theta_{Ck}, t)$ starting from a commonly used deterministic model facilitates the acceptance and practical use of the novel probabilistic model. The correction term is a function of $\mathbf{x}_0$, t and θ_{Ck} . A regression analysis can be used to assess the vector of unknown parameters $\Theta_{Ck} = (\theta_{Ck}, \sigma_{Ck})$ (a Bayesian approach to assess Θ_{Ck} on the basis of available information is described next). In addition, a selection process facilitates the development of a parsimonious correction term that has the minimum complexity still retaining accuracy of the model. Additional details can be found in Gardoni *et al.* (2002).

To estimate Θ_{Ck} , the following three assumptions have to be satisfied: (a) additivity of the model error term $\sigma_{Ck} \varepsilon_{Ck}$, (b) the model variance σ_{Ck}^2 is independent of $\mathbf{x}_0$ and t (homoskedasticity assumption), and (c) ε_{Ck} has the normal distribution (normality assumption). Transformations (e.g., a logarithmic transformation) of the quantity of interest can be used to approximately satisfy these assumptions and ameliorate potential non-

additivity, non-normality, and heteroskedasticity (Box and Cox 1964). Diagnostic plots (plots of the data or the residuals against model predictions or individual regressors) can be used to verify that these assumptions are approximately satisfied (Rao and Toutenburg 1997).

Similar to Eq. (19.1), a probabilistic demand model can be written in the following general form (Gardoni *et al.* 2003):

$$D_{ik}(\mathbf{x}_0, \boldsymbol{\Theta}_{Dk}, t, S_a) = D_{ik}(\mathbf{x}_0, \boldsymbol{\Theta}_{Dk}, t, S_a) + \sigma_{Dk} \epsilon_{Dk} \quad [19.2]$$

where D_{ik} is the demand variable that corresponds to the adopted capacity variable C_{ik} ; and S_a is a measure of seismic intensity, in this study the spectral acceleration at the natural period of the structure is considered. Probabilistic capacity and demand models developed following the above formulations are unbiased and capture the relevant uncertainties, including model errors, measurement errors, and statistical uncertainty.

Model assessment using a Bayesian statistical approach

A Bayesian statistical framework is ideally suited to calibrate probability models, especially when different sources of information are available. The body of information might include both objective information, such as empirical data and mathematical models, and subjective information, such as expert knowledge, engineering experience, and judgment. In the literature, a Bayesian approach has been used to construct probabilistic models that follow the above formulation to predict structural capacities (Choe *et al.* 2007, 2008; Huang *et al.* 2009; Gardoni *et al.* 2009; Pillai *et al.* 2010a) and demands due to different hazards including earthquakes (Zhong *et al.* 2008; Huang *et al.* 2010) and scour of foundation (Bolduc *et al.* 2008).

This section summarizes the Bayesian framework with reference to the probabilistic capacity models. The same formulation is also applicable to construct the probabilistic demand models. In a Bayesian approach, the distribution of the vector of unknown parameters $\boldsymbol{\Theta}_{Ck}$ is estimated by using the following updating rule (Box and Tiao 1992):

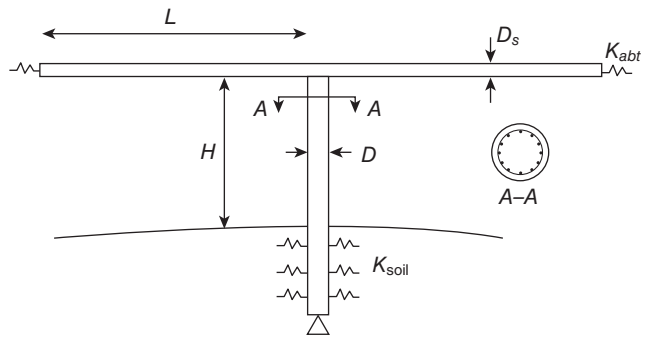
$$p(\boldsymbol{\Theta}_{Ck} | \mathbf{C}_{ik}) = \eta L(\boldsymbol{\Theta}_{Ck} | \mathbf{C}_{ik}) p(\boldsymbol{\Theta}_{Ck}) \quad [19.3]$$

where $p(\boldsymbol{\Theta}_{Ck})$ is the *prior distribution* of $\boldsymbol{\Theta}_{Ck}$ that captures what is known about $\boldsymbol{\Theta}_{Ck}$ before the vector of data $\mathbf{C}_{ik} = (C_{ik,1}, \dots, C_{ik,n})$ is observed; $L(\boldsymbol{\Theta}_{Ck} | \mathbf{C}_{ik})$ is the *likelihood function* that brings in the objective information about $\boldsymbol{\Theta}_{Ck}$ that comes from $\mathbf{C}_{ik}$; $\eta = [\int L(\boldsymbol{\Theta}_{Ck} | \mathbf{C}_{ik}) p(\boldsymbol{\Theta}_{Ck}) d\boldsymbol{\Theta}_{Ck}]^{-1}$ is a normalizing factor; and $p(\boldsymbol{\Theta}_{Ck} | \mathbf{C}_{ik})$ is the *posterior distribution* of $\boldsymbol{\Theta}_{Ck}$ that reflects the knowledge about $\boldsymbol{\Theta}_{Ck}$ that comes from both $p(\boldsymbol{\Theta}_{Ck})$ and $\mathbf{C}_{ik}$. The prior distribution can reflect any information about $\boldsymbol{\Theta}_{Ck}$ that is based on past experience, engineering judgment, or previous analyses (Choe *et al.* 2007, Zhong *et al.* 2009). When no past knowledge is available, a

noninformative prior can be used to reflect the fact that little or nothing is known before collecting $\mathbf{C}_{ik}$.

Structural capacity of, and seismic demand on, an example RC bridge subject to deterioration

This section gives an example of the effects of deterioration, specifically corrosion (although this could include other expansive deterioration mechanisms), on the structural capacity of and the seismic demand on a typical RC bridge. This example is taken from Gardoni *et al.* (2003) and Choe *et al.* (2009). Figure 19.3 shows an example RC bridge – a single-bent highway overpass. The values of the design variables listed in Table 19.1 are



19.3 Configuration of the example bridge (not to scale) (Gardoni *et al.* 2003). © 2003 Taylor & Francis.

Table 19.1 Variables for the pristine single-bent bridge

Description	Variable	Value/ distribution
Span length (right and left)	L [mm]	18300
Span-to-column height ratio	L/H	2.4
Column-to-superstructure dimension ratio	D/D_s	0.75
Concrete nominal strength	f'_c [MPa]	LN(27.6, 2.76)
Reinforcement nominal yield strength	f_y [MPa]	LN(448.2, 22.4)
Initial longitudinal reinforcement ratio of column	ρ_{l0}	2.0%
Initial transverse reinforcement ratio of column	ρ_{s0}	0.7%
Soil stiffness based on NEHRP groups (FEMA 273/274, 1997)	K_{soil}	B
Additional bridge dead load (as a ratio of the dead weight)	r	N(0.1, 0.025)

Note: LN() indicates the lognormal distribution, N() indicates the normal distribution, and the two parameters provided for these two distributions are the mean and standard deviation, respectively.
Source: Adapted from Choe *et al.* (2009).

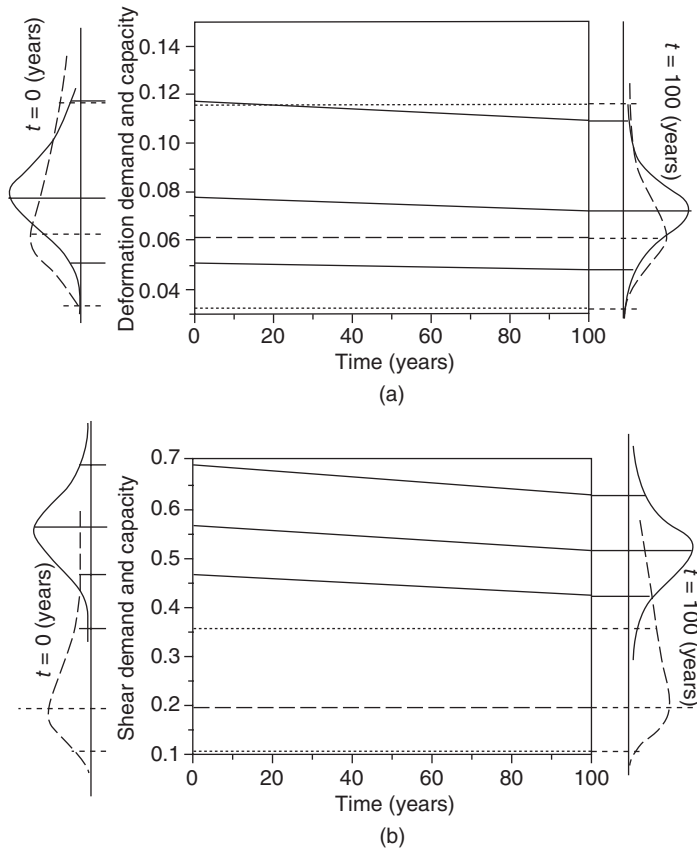
representative of current construction in California. Specifically, Mackie and Stojadinovic (2001) designed the bridge following the Caltrans Bridge Design Specification and Seismic Design Criteria (Caltrans 1999). For the parameters that affect the corrosion initiation and propagation, Choe *et al.* (2009) assumed that the concrete has a water-to-cement ratio (w/c) of 0.5 and 1 day curing time, and that the bridge is located in a tidal zone (thus affected by chlorides) with humid–dry cycles.

Three variables are assumed to be random: the compressive strength of concrete, f'_c and the yield stress of the longitudinal reinforcement, f_y , are modeled as a random variable to capture the uncertainty in the material properties, and the third random variable is the bridge dead load, r , is used to model the variability in the axial load. Specifically, it was assumed that f'_c follows a lognormal distribution with mean 27.6 MPa and 10% coefficient of variation; f_y follows a lognormal distribution with mean 448.2 MPa and 5% coefficient of variation; and r follows a normal distribution with mean equal to 10% of the dead weight and a 25% coefficient of variation. Additional variables needed to model the corrosion initiation and propagation (such as the reference diffusion coefficient, etc.) were taken to be random in accordance with DuraCrete (2000).

Figure 19.4 shows the deformation and shear capacity of the RC column and the corresponding seismic demands as a function of time, where the demands are computed for $S_a = 2.0$ g. The solid lines show the mean capacities along with the confidence bands computed as the $\text{mean} \pm 1$ standard deviation of the model error, σ_{Ck} , whereas the dashed lines show the mean demands along with the confidence bands computed as the $\text{mean} \pm \sigma_{Dk}$. The mean deformation and shear capacities decrease over time while the mean deformation and shear demands remain approximately constant. In addition, the mean shear capacity decays at a faster rate than the mean deformation capacity. The next section discusses the effects of the deterioration on the seismic reliability of RC bridges.

19.3.2 Effects of deterioration on seismic reliability of RC bridges

Time-dependent deterioration models can be used to assess the reliability of RC structures over time (Mori and Ellingwood 1993; Enright and Frangopol 1998a,b; Stewart and Rosowsky 1998; Li and Melchers 2005; Choe *et al.* 2009, 2010). Fragility estimates can be developed to quantify the reliability of a bridge structure or sub-structure at time t . The fragility is defined as the conditional probability that any demand measure, $D_{ik}(t)$, is larger than or equal to the corresponding capacity measure, $C_{ik}(t)$, for a given S_a . The fragility estimates can be computed using methods of structural reliability, such as first- and second-order reliability methods (FORM and



19.4 Variation over time of deformation and shear capacities (solid line) and demands for $S_a = 2.0/g$ (dashed line) for the example RC bridge due to corrosion: (a) deformation failure mode; (b) shear failure mode (Choe *et al.* 2009). Reprinted with permission from Elsevier © 2009.

SORM) and simulation techniques (Ditlevsen and Madsen, 1996). The estimates can account for the variability in materials, geometry, loading, and the extent of deterioration.

Fragility assessment

Following Gardoni *et al.* (2002, 2003), the fragilities for an undamaged component i in a failure mode k and for an entire undamaged structure in any mode of failure can be written respectively as

$$\begin{aligned}
 F_{ik}(\boldsymbol{\Theta}, 0, S_a) &= P[g_{ik}(\mathbf{x}_0, \boldsymbol{\Theta}, S_a) \leq 0 | S_a] \\
 F(\boldsymbol{\Theta}, 0, S_a) &= P\left\{\bigcup_{i,k} [g_{ik}(\mathbf{x}_0, \boldsymbol{\Theta}, S_a) \leq 0] | S_a\right\}
 \end{aligned}
 \quad [19.4]$$

where $\Theta = (\Theta_C, \Theta_D)$ and

$$g_{ik}(\mathbf{x}_0, \Theta, S_a) = C_{ik}(\mathbf{x}_0, \Theta_C) - D_{ik}(\mathbf{x}_0, \Theta_D, S_a) \quad [19.5]$$

Similarly, the fragilities at time t can be expressed as

$$\begin{aligned} F_{ik}(\Theta, t, S_a) &= P[g_{ik}(\mathbf{x}_0, \Theta, t, S_a) \leq 0 | t, S_a] \\ F(\Theta, t, S_a) &= P\left\{\bigcup_k [g_{ik}(\mathbf{x}_0, \Theta, t, S_a) \leq 0] | t, S_a\right\} \end{aligned} \quad [19.6]$$

where

$$g_{ik}(\mathbf{x}_0, \Theta_k, t, S_a) = C_{ik}(\mathbf{x}_0, \Theta_C, t) - D_{ik}(\mathbf{x}_0, \Theta_D, t, S_a) \quad [19.7]$$

In addition, predictive fragility estimates $\tilde{F}_{ik}(t, S_a)$ and $\tilde{F}(t, S_a)$ (Gardoni *et al.* 2002) can be constructed to incorporate the epistemic uncertainty in the models parameters Θ as

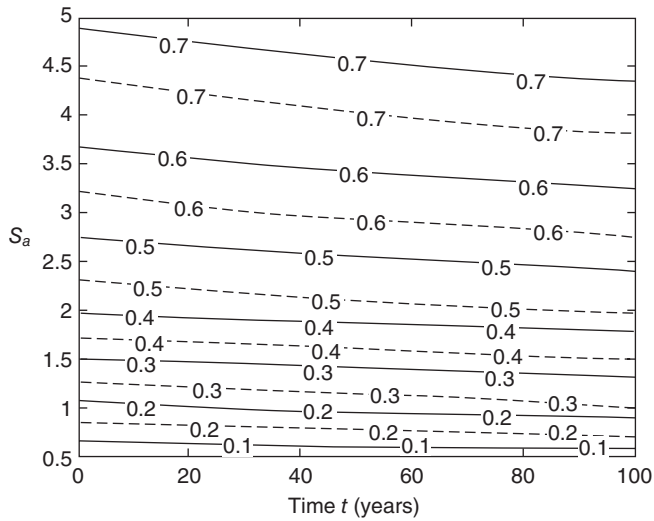
$$\begin{aligned} \tilde{F}_{ik}(t, S_a) &= \int F_{ik}(\Theta, t, S_a) f_{\Theta}(\Theta) d\Theta \\ \tilde{F}(t, S_a) &= \int F(\Theta, t, S_a) f_{\Theta}(\Theta) d\Theta \end{aligned} \quad [19.8]$$

The predicted fragilities are the expected value of $F_{ik}(\Theta, t, S_a)$ or $F(\Theta, t, S_a)$ over the distribution of Θ .

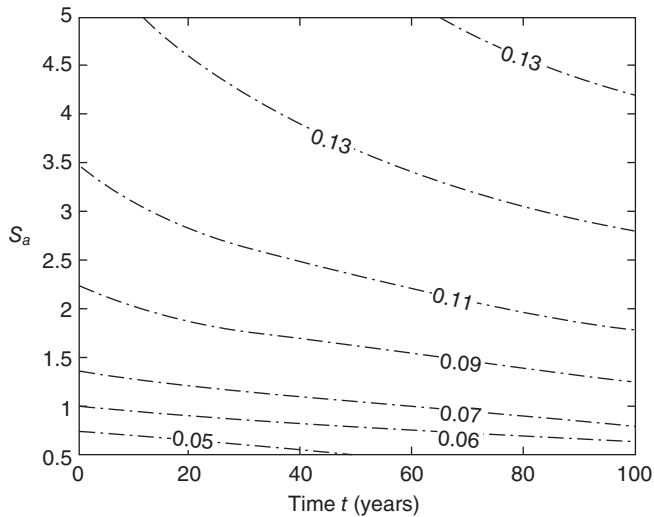
The analyses are carried out using OpenSees. This is a comprehensive, open-source, object-oriented finite element software that has reliability and response sensitivity capabilities (Haukaas and Der Kiureghian 2005). Choe *et al.* (2008, 2009) and Zhong *et al.* (2012a) extended OpenSees with the implementation of probabilistic models for corrosion initiation, corrosion rate, and loss of reinforcement area. In addition, Choe *et al.* (2007) and Zhong *et al.* (2012b) developed closed-form approximations to the fragility that do not require any specialized reliability software.

Reliability of an example RC bridge subject to deterioration

This section shows the results of the methodology described above as it is applied to the example RC bridge already introduced. The numerical results of the reliability analysis are taken from Choe *et al.* (2009). Figure 19.5 shows the iso-probability lines of $\tilde{F}_{ik}(t, S_a)$ and $\tilde{F}(t, S_a)$ as a function of t and S_a , where the iso-probability lines are contour lines along which $\tilde{F}_{ik}(t, S_a)$ and $\tilde{F}(t, S_a)$ have constant values. Figure 19.5(a) shows the iso-probability lines considering the deformation failure mode only (solid lines) and considering that either the deformation or the shear demands might exceed the corresponding capacities (dashed lines). Figure 19.5(b) shows the iso-probability lines considering the shear failure mode only. It can be observed



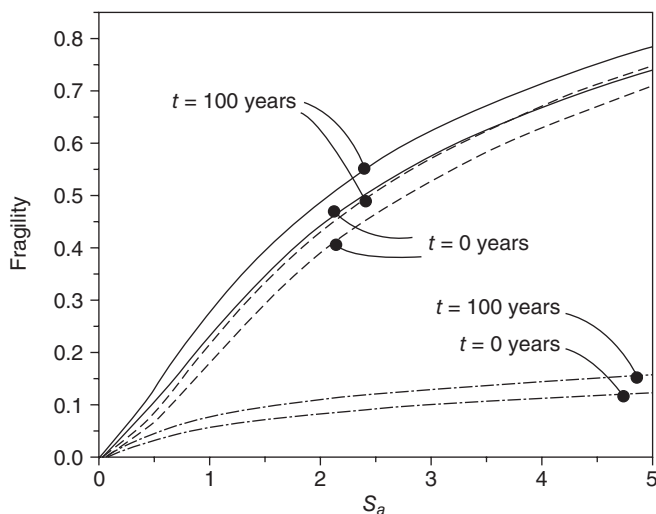
(a)



(b)

19.5 Iso-probability lines for $\tilde{F}(t, S_a)$ for the example RC bridge: (a) deformation (solid), and deformation and shear (dashed) failure mode, (b) shear failure mode (Choe *et al.* 2009). Reprinted with permission from Elsevier © 2009.

that as t increases due to the effects of corrosion the same values of $\tilde{F}(t, S_a)$ are reached at smaller values of S_a . In Fig. 19.4 it was observed that the mean shear capacity decays at a faster rate than the mean deformation capacity. As a result, Fig. 19.5 shows that the shear fragility increases at a



19.6 $\tilde{F}(t, S_a)$ for the example RC bridge considering deformation (dashed), shear (dash-dotted), and deformation and shear failure modes (Choe *et al.* 2009). Reprinted with permission from Elsevier © 2009.

faster rate than the deformation fragility. However, because the deformation failure mode controls (i.e., it is significantly more likely than the shear failure mode), then $\tilde{F}(t, S_a)$ varies at a rate close to the one of the deformation fragility.

Figure 19.6 compares the fragilities of the undeteriorated ($t = 0$ years) and the deteriorated bridge at $t = 100$ years considering the deformation failure mode only (dashed lines), the shear failure mode only (dash-dotted lines), and considering that either the deformation or the shear demands might exceed the corresponding capacities (solid lines). It can be observed that at $t = 0$ years the deformation failure mode is the most likely mode of failure, which is consistent with Caltrans specifications used to design the example bridge. At $t = 100$ years, consistently with what observed in Fig. 19.5, $\tilde{F}(t, S_a)$, increases for a given S_a , however, at least for this example bridge, the deformation failure mode remains the most likely one. Additional work and examples of the effects of corrosion on the reliability of RC bridges can be found in Zhong *et al.* (2012a), and Simon *et al.* (2010), while those on the reliability of post-tensioned (PT) bridges can be found in Trejo *et al.* (2009), Gardoni *et al.* (2009), and Pillai *et al.* (2009, 2010a,b).

Seismic fragility increment functions for deteriorating RC bridges

In the formulation described so far, a reliability analysis has to be performed at each time t using the corresponding capacity and demand

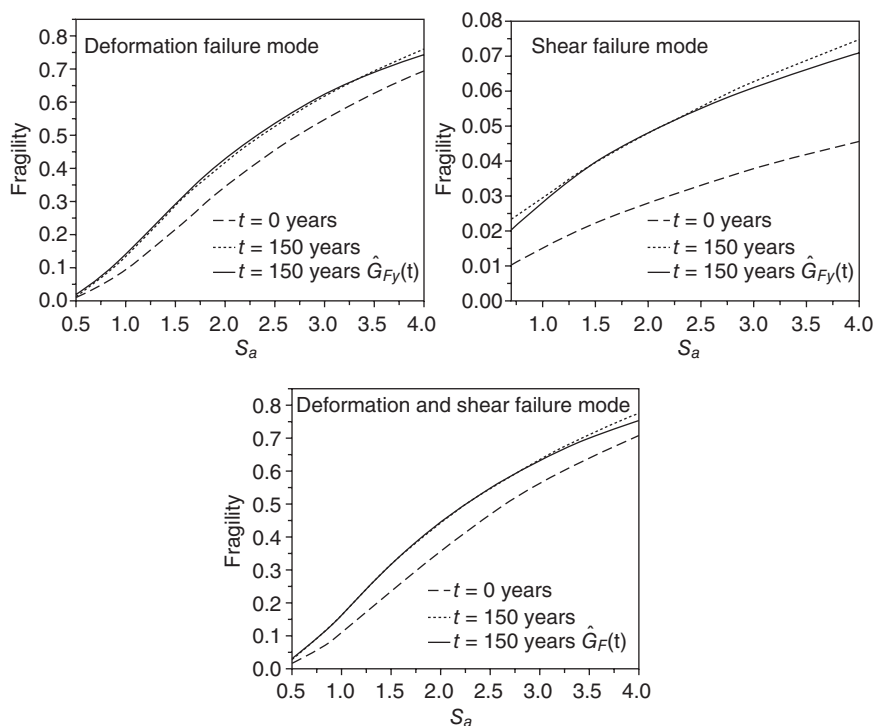
estimates $C_{ik}(\mathbf{x}_0, \Theta_C, t)$ and $D_{ik}(\mathbf{x}_0, \Theta_D, t, S_a)$. To avoid conducting a reliability analysis at each time t , Gardoni and Rosowsky (2011) developed seismic *fragility increment functions*, $\hat{G}_{F,k}(t)$ and $\hat{G}_F(t)$, for single column RC bridges. In this case, the subscript $i = 1$ and is therefore omitted for simplicity in the notation. The fragility increment functions provide estimates of the fragilities at time t , given the fragilities at $t = 0$, $\tilde{F}_k(t, S_a)$ and $\tilde{F}(0, S_a)$, without requiring any additional reliability analysis. The $\hat{G}_{F,k}(t)$ and $\hat{G}_F(t)$ are constructed such that estimates of the fragilities at time t can be obtained as

$$\begin{aligned}\hat{F}_k(t, S_a) &= \hat{G}_{F,k}(\mathbf{x}_0, \Lambda_k, t, S_a) \times \tilde{F}_k(0, S_a) \\ \hat{F}(t, S_a) &= \hat{G}_F(\mathbf{x}_0, \Lambda, t, S_a) \times \tilde{F}(0, S_a)\end{aligned}\quad [19.9]$$

The fragility increment functions are functions of the environmental conditions, the original material properties, a set of unknown model parameters, Λ_k or Λ , t , and S_a . The Akaike Information Criterion (AIC) (Akaike, 1978a,b) and the Bayesian Information Criterion (BIC) (Schwarz, 1978) were used to select accurate and parsimonious model forms for $\hat{G}_{F,k}(t)$ and $\hat{G}_F(t)$, and the Bayesian updating rule was used to estimate the unknown model parameters. The model selection and parameter estimation were carried out using fragility data obtained by conducting a predictive reliability analysis as described in Eq. 19.8 (Gardoni *et al.* 2002) using probabilistic capacity and demand models for corroding RC columns in the form shown in Eqs. 19.1 and 19.2 and calibrated by Choe *et al.* (2008, 2009).

The developed $\hat{G}_{F,k}(t)$ and $\hat{G}_F(t)$ are applicable within a wide range of environmental conditions and material properties. The fragility increment functions capture the effects of deterioration on the mean capacities and mean demands, and of the increasing uncertainty over time in the probabilistic models for the corrosion initiation and the time-dependent corrosion rate as it is reflected in the probabilistic capacity and demand models used to generate the fragility data.

As an example, Gardoni and Rosowsky (2011) used the developed $\hat{G}_{F,k}(t)$ and $\hat{G}_F(t)$ to estimate the fragilities for deformation ($k = \delta$), shear ($k = \nu$), and combined deformation–shear failure modes of the example bridge discussed in the previous section. Figure 19.7 shows the fragility estimates for the example bridge versus S_a . In each subplot, the dashed line indicates $\tilde{F}_k(0, S_a)$ or $\tilde{F}(0, S_a)$, obtained by a reliability analysis following Gardoni *et al.* (2003), the dotted line indicates the predictive fragility $\tilde{F}_k(t = 150 \text{ years}, S_a)$ or $\tilde{F}(t = 150 \text{ years}, S_a)$ obtained by a reliability analysis following Choe *et al.* (2009) as discussed in the previous section, and the solid line indicates $\hat{F}_k(t = 150 \text{ years}, S_a)$ or $\hat{F}(t = 150 \text{ years}, S_a)$ obtained using the fragility increment functions using Eq. 19.9. Overall the estimates obtained using the fragility increment functions are in close agreement with the predictive fragility estimates obtained from a traditional reliability analysis and only



19.7 Fragility estimates for the example RC bridge (Gardoni and Rosowsky 2011). Reprinted by permission of the publisher (Taylor & Francis Ltd, <http://www.tandf.co.uk/journals>).

marginal differences can be observed. In particular, is the range of interest for civil engineering applications, which corresponds to small values of demand, the fragility estimates are almost identical. These observations are made considering ‘worst case’ scenario with $t = 150$ years. It is expected that for smaller values of t the differences between the two approaches are even smaller.

Model updating based on data from field inspections

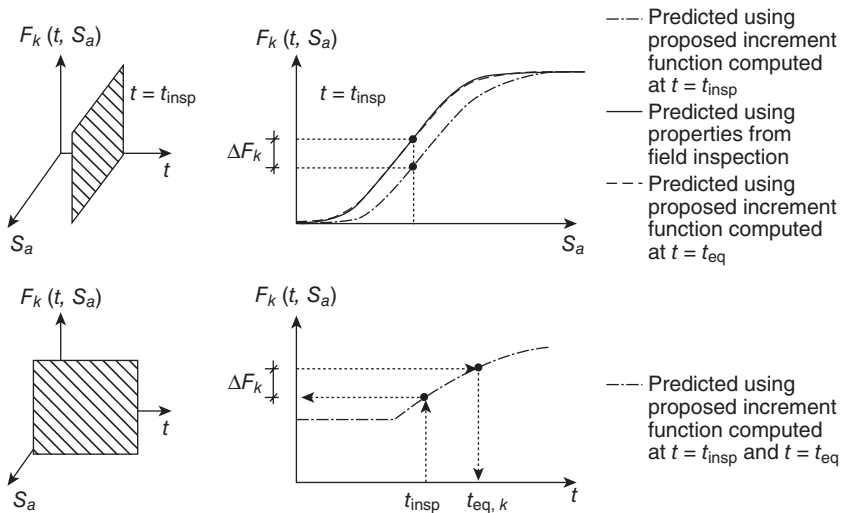
Fragility increment functions are valuable to capture the predicted deterioration of bridges over time for a reliability-based design, life-cycle cost analysis, or risk analysis conducted before the construction of a bridge. However, for existing bridges, additional information might be available from field inspections. This section briefly describes a methodology developed by Gardoni and Rosowsky (2011) to incorporate data from a field inspection at time t_{insp} to update the fragility estimates at $t > t_{\text{insp}}$. For brevity,

the updating process is presented for $\hat{F}_k(t, S_a)$; however the same process can be used to update $\hat{F}(t, S_a)$.

An estimate of $F_k(t_{\text{insp}}, S_a)$ can be obtained as $\hat{F}_k(t_{\text{insp}}, S_a) = \hat{G}_{F,k}(\mathbf{x}_0, t_{\text{insp}}, S_a) \times \tilde{F}_k(\mathbf{x}_0, S_a)$ using $\hat{G}_{F,k}(\mathbf{x}_0, t, S_a)$ to model the deterioration process but ignoring the information from the field observations. Alternatively, $F_k(t_{\text{insp}}, S_a)$ can be estimated by assessing the predictive fragility $\tilde{F}_k(\mathbf{x}_{\text{insp}}, S_a)$ by conducting a reliability analysis using directly the current values of the geometry and material properties obtained from the field inspection, and without using $\hat{G}_{F,k}(\mathbf{x}_0, t_{\text{insp}}, S_a)$. Figure 19.8 shows the difference, $\Delta F_k(S_a)$, between the two estimates. An equivalent time $t_{\text{eq},k}$ can be determined to update the deterioration process modeled by $\hat{G}_{F,k}(\mathbf{x}_0, t, S_a)$ so that the corresponding fragility estimate matches the one based on $\mathbf{x}_{\text{insp}}$, that is such that $\hat{F}_k(t_{\text{eq},k}) = \hat{G}_{F,k}(\mathbf{x}_0, t_{\text{eq},k}, S_a) \times \tilde{F}_k(\mathbf{x}_0) \approx \tilde{F}_k(\mathbf{x}_{\text{insp}})$. The deterioration process might affect the deformation and shear modes of failure differently, as a result, in general $t_{\text{eq},\delta} \neq t_{\text{eq},v}$.

If the updating of the deterioration process modeling is believed to be due to an inaccurate estimate of the time of initiation of the deterioration, then the fragility at a time t after the inspection (i.e., $t > t_{\text{insp}}$) can be estimated using $t_{\text{eq},k}$ as

$$\hat{F}_k(t, S_a) = \hat{G}_{F,k}[\mathbf{x}_0, (t_{\text{eq},k} + \Delta t), S_a] \times \tilde{F}_k(\mathbf{x}_0) \quad t > t_{\text{insp}} \quad [19.10]$$



19.8 Definition of $t_{\text{eq},k}$ (Gardoni and Rosowsky 2011). Reprinted by permission of the publisher (Taylor & Francis Ltd, <http://www.tandf.co.uk/journals>).

where $\Delta t = t - t_{\text{insp}}$. In this formulation, $t_{\text{eq},k}$ provides a shift along the time axis of $\hat{G}_{F,k}$ to adjust the initiation of the deterioration process; however, no correction is made to the deterioration rate. Alternatively, in case both the initiation time and the rate of the deterioration process need to be adjusted, then the fragility at time t after the inspection can be estimated as

$$\hat{F}_k(t, S_a) = \hat{G}_{F,k} \left[\mathbf{x}_0, \left(t_{\text{eq},k} + \frac{t_{\text{eq},k}}{t_{\text{insp}}} \Delta t \right), S_a \right] \times \tilde{F}_k(\mathbf{x}_0) \quad t > t_{\text{insp}} \quad [19.11]$$

In this case $t_{\text{eq},k}$ is used to shift $\hat{G}_{F,k}$ along the time axis and to contract or dilate the time axis to account for a difference in the deterioration rate.

19.4 Conclusions

Although little information is available on deterioration rates of structures exhibiting ASR and other mechanisms of deterioration, models are available for corrosion-induced deterioration. Preventive measures must consider the environment in which the structure is to be constructed and the materials used to construct the structure. In an environment that contains aggressive conditions, such as chlorides, sulfates, freeze–thaw cycles, carbonation, or other harmful conditions, their impact on deterioration must be considered during the design phase, and strategies to mitigate damage must be implemented. Concrete is a good material for civil engineering structures, enabling society to place structures in environments that were considered to be too harsh years ago. Using marginal materials and placing concrete in aggressive environments present challenges, especially if proper materials, design, and construction practices are not used, resulting in premature deterioration.

A deteriorating structure that continues to provide safe functionality still has value. In fact, mitigating the deterioration can add further value. This chapter provided a brief overview of typical deterioration mechanisms and developed a general damage model that is useful for assessing the reliability of bridges experiencing deterioration due to corrosion. This information can provide engineers and decision makers with quantitative information on the safety of the structure such that decisions can be made regarding repair, rehabilitation, or if demolition needs to occur. The limits for making these decisions are based on many factors (degree of damage, structure importance, use, etc.) and decision makers will have to identify which factors are more important in making an optimal decision. Although the procedure provided here focused on corroding bridges, the general procedure can be applied to a wide range of structure types and deterioration mechanisms. The literature on modeling the time-variant performance of RC structures is limited and significant research is needed to better understand the effects

on structure performance. Much of the on-going research is focusing on single mechanisms of deterioration. Research is needed to better understand the synergistic effects of multiple deterioration processes, often found in the field.

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Using a performance-based earthquake engineering (PBEE) approach to estimate structural performance targets for bridges

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Abstract: Performance-based seismic evaluation aims to provide information on expected seismic performance of structures in measurable and meaningful terms for decision makers (performance objectives). The Pacific Earthquake Engineering Research (PEER) Center utilized the performance-based earthquake engineering (PBEE) framework by linking the earthquake scenarios, design ground motions, structural demand, and performance variables. PBEE procedure may be simplified by decoupling the evaluation of the design ground motion levels (probabilistic seismic hazard assessment, PSHA) and that of the structural demand due to design ground motions (probabilistic seismic demand assessment, PSDA). The objective of this chapter is to present the vector-valued implementation of PEER PBEE approach. Key points of PSDA methodology such as selection of engineering demand parameters along with the intensity measures, effects of ground motion record scaling, and construction of probabilistic seismic demand models are discussed. Integration of vector-valued intensity measures into these models is explained in detail. Results of a recent study investigating the effect of vertical ground motions on ordinary highway bridges is used to demonstrate the application of PEER PBEE framework for a particular class of structures.

Key words: Performance-based earthquake engineering (PBEE), probabilistic seismic demand models, engineering demand parameters, probabilistic seismic hazard assessment (PSHA), vertical ground motions.

20.1 Introduction

According to modern building codes (FEMA-273, 1997; FEMA-350, 2000), structures should be designed to ensure specific performance objectives under design ground motion levels. These performance objectives can be expressed in terms of annual frequency of exceeding a given level of structural response (or structural demand parameter). Estimating the annual frequency of exceeding any structural demand parameter involves several steps, such as: computing the design ground motions according to the accepted hazard levels in the code, evaluating the structural response due

to the design ground motions, and comparing the calculated structural response parameters with the performance objectives. The Pacific Earthquake Engineering Research (PEER) Center utilized the performance-based earthquake engineering (PBEE) framework that covers the essential elements of this problem and encouraged the research efforts on each step towards the evaluation of structural performance objectives (Cornell and Krawinkler, 2000; Stewart *et al.*, 2002). The PBEE framework components and the links among the earthquake scenarios, design ground motions, structural response, and performance variables are explained in detail in the following section.

Studies in the past decade (e.g. Luco, 2002; Baker and Cornell, 2003; Tothong and Cornell, 2006) demonstrated that the PBEE procedure may be simplified by decoupling the evaluation of the design ground motion levels (probabilistic seismic hazard assessment, PSHA) and that of the structural response due to design ground motions (i.e. probabilistic seismic demand assessment, PSDA). The details of the PSDA methodology, especially discussions on controversial issues, such as selecting proper ground motions and intensity measures are presented in Section 20.3. Recent advances in the field call for the use of vector-valued intensity measures and vector-valued probabilistic seismic hazard assessment tools; therefore Section 20.4 is devoted to the vector-valued probabilistic seismic hazard/demand assessment procedure. An application of the PBEE framework for the evaluation of near-fault vertical ground motion effects on the ordinary highway bridges is provided in Section 20.5 to further explain the theoretical background given in the previous sections. Finally, the vital parts of the PBEE framework that requires further research and elaboration are discussed briefly in the last section.

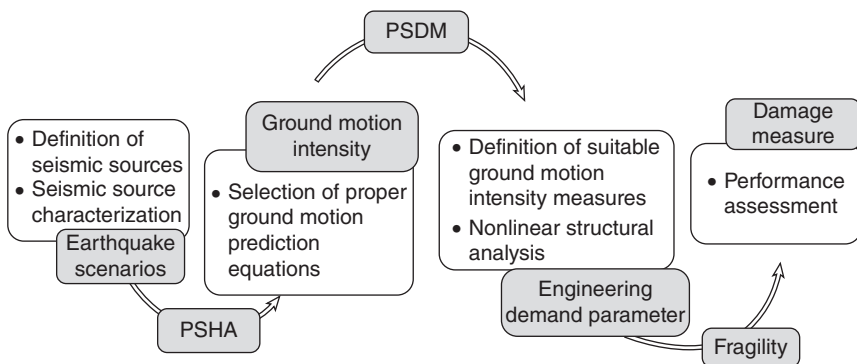
20.2 Performance-based seismic evaluation framework (PEER approach)

Performance-based seismic evaluation is a process that results in a realistic understanding of seismic risk due to future earthquakes. It aims to provide information on expected seismic performance of structures in measurable and meaningful terms for decision makers. To meet this objective, the PEER Center developed a PBEE framework that integrates a series of distinct and logically related parts of the problem. According to Stewart *et al.* (2002), PBEE includes the evaluation of the distributions of four fundamental variables conditioned on the previous variable in the chain: (1) evaluating the probability of exceeding ground motion intensity measures (IMs) at a site given earthquake scenarios, (2) distribution of engineering demand parameters (EDPs) given a particular set of IMs,

(3) distribution of damage measures (DMs; e.g. physical condition of a damaged element) given EDPs, and (4) evaluation of the probability of exceeding decision variables (DVs; e.g. human or collateral loss, post-earthquake repair time, and other parameters of interest to an owner) given appropriate DMs. These four steps in the performance-based design methodology are linked through the theorem of total probability. The relations between key variables in the PEER-PBEE framework are illustrated in Fig. 20.1 and the PBEE integral is presented in Equation 20.1:

$$v(DV) = \iiint G(DV|DM) \times dG(DM|EDP) \times dG(EDP|IM) \times v(IM) \quad [20.1]$$

where $v(DV)$ is the annual rate of exceeding the decision variable; $G(DV|DM)$ is the probability of exceeding the decision variable given the damage measure; $dG(DM|EDP)$ is the probability of exceeding the damage measure given the engineering demand parameter; $dG(EDP|IM)$ is the probability of exceeding the engineering demand parameter given the intensity measure; and $v(IM)$ is the annual rate of exceeding the ground motion intensity measure (Cornell and Krawinkler, 2000; Krawinkler, 2002). Integrals in Equation 20.1 represent the variability in each key item, conditioned on the previous variable in the chain. In order to simplify the problem, the distributions of the variables are assumed to be independent of each other. For example, DM is a function of EDP only and knowledge of IM provides no additional information (Baker and Cornell, 2003). This assumption permits the user to treat each part of the integral separately.



20.1 PEER performance-based earthquake engineering framework (PSDM = probabilistic seismic demand model, PSHA = probabilistic seismic hazard analysis).

20.2.1 Probabilistic seismic hazard analysis (PSHA) and ground motion intensity measures

The first part of the problem is the estimation of the annual rate of exceeding IM. In other words, seismic hazard analysis represents a link between earthquake scenarios and ground motion intensity in Fig. 20.1. Traditionally, probabilistic seismic hazard analysis (PSHA) due to a point source is carried out by evaluating the following equation (Cornell, 1968; McGuire, 2004):

$$v(IM > z) = N_{\min} \cdot \int_M \int_R f_M(M) f_R(M, R) P(IM > z | M, R) \times dM \times dR \quad [20.2]$$

where R is the distance from the source to site; M is the earthquake magnitude; $N_{\min}$ is the annual rate of earthquakes with magnitude greater than or equal to the minimum magnitude; $f_M(M)$ and $f_R(M, R)$ are the probability density functions for the magnitude and distance, and $P(IM > z | M, R)$ is the probability of observing an IM greater than z for a given earthquake magnitude and distance. The IM is the quantification of the characteristics of a ground motion that are important to the structural response. PSHA requires the definition of seismic sources close to the specific site and characterization of these seismic sources by appropriate probability density functions and recurrence models. After defining a suite of earthquake scenarios, the range of ground motions for each earthquake scenario is estimated and the annual rate of each combination of earthquake scenario and ground motion is computed. The probability that the IM will exceed z , $P(IM > z | M, R)$, is obtained from the ground motion prediction equation (GMPE) and includes an implicit integration over the ground motion variability. The probability that the IM will exceed z is given by:

$$P(IM > z | M, R) = \int_{\epsilon} f_{\epsilon}(\epsilon) \times P(IM > z | M, R, \epsilon) \times d\epsilon \quad [20.3]$$

where the epsilon (ϵ) is the number of standard deviations above or below the median, $f_{\epsilon}(\epsilon)$ is the probability density function for the epsilon (given by the standard normal distribution) and $P(IM > z | M, R, \epsilon)$ is either 0 or 1. In this formulation, $P(IM > z | M, R, \epsilon)$ identifies earthquake scenarios and ground motion combinations that lead to IM greater than z . The hazard (Equation 20.2) can then be written as:

$$\begin{aligned} v(IM > z) \\ = N_{\min} \cdot \int_M \int_R \int_{\epsilon} f_M(M) f_R(M, R) f_{\epsilon}(\epsilon) P(IM > z | M, R, \epsilon) \times dM \times dR \times d\epsilon \end{aligned} \quad [20.4]$$

While this form is more complicated than Equation 20.1, it has the advantage of clearly showing that the hazard integral accounts for aleatory variability in three main parameters of scenario earthquake: magnitude, distance, and epsilon (Gülerce and Abrahamson, 2010). Using Equation 20.4, the annual rate of exceeding any ground motion intensity measure may be calculated using the seismic source models and GMPEs suitable for the region.

20.2.2 Engineering demand parameter

The earthquake hazard is associated with direct consequences (damage to property or loss of function) and indirect consequences (such as loss of productivity or jobs) of earthquakes (McGuire, 2004). In order to predict the economic loss resulting from earthquake ground motions in a building or any other structure, it is necessary to predict the response of the structure when subjected to ground motions of different intensity levels (Baker and Cornell, 2003). To evaluate the seismic performance of structures, uncertainties in the ground motions and nonlinear structural responses need to be considered. The most direct way to make probabilistic estimates of earthquake damage is to express it as a function of M and R , as:

$$(EDP) = N_{\min} \cdot \int_R \int_M f_m(M) \times f_r(M, R) \times P(EDP > y|M, R) \times dM \times dR \quad [20.5]$$

where $P(EDP > y|M, R)$ is the probability of observing an EDP greater than y for a given earthquake magnitude and distance. PSHA procedures can be used to directly estimate earthquake damage using this form of the hazard equation. The concept is similar to the conventional PSHA, but the GMPE is replaced with a structure-specific response prediction equation. To develop a structure-specific response prediction model, the structure should be analyzed for a large number of ground motions. The results of nonlinear dynamic analysis can be used to model the distribution of structural demand with magnitude and distance by regression analysis. The implicit assumptions in this method are (Baker and Cornell, 2003): (1) the functional form of the regression equation, (2) the lack of dependence of EDP on the source characteristics not contained in the vector of independent variables (e.g., rupture duration), and (3) the lack of dependence of EDP on the geometry of the fault relative to the site. The need for hundreds of dynamic analysis to obtain a reliable estimate for the structure- and response-specific prediction model and the complications involved in the modeling process called for a simplified procedure that treats the ground motion hazard and structural response independently. Cornell and co-workers (Baker and Cornell, 2003; Tothong and Cornell, 2006) proposed

the probabilistic seismic demand model (PSDM) approach where the results of nonlinear dynamic analyses for a specific structure are used to evaluate behavior of important EDPs in terms of the IM levels. The main idea of these studies was to develop PSDMs for particular structures and to provide the annual frequency of exceeding a given structural engineering demand measure y conditioned on IM, as:

$$v(EDP > y | IM) = \int_{IM} f_{EDP}(EDP | IM) \times dv(IM) \quad [20.6]$$

where $f_{EDP}(EDP | IM)$ is the probabilistic seismic demand model for a particular EDP and IM. PSDMs represent the connection between IMs and EDPs as shown in Fig. 20.1 and provide information about the probability of exceeding the pre-determined critical levels of EDPs for a particular class of structures. These models may be used as risk-based design tools, since they present the variability in the structural demand parameters for specified ground motion intensities. Moreover, when coupled with PSHA, PSDMs can be used to compute structural demand hazard curves (Mackie and Stojadinovic, 2003). PSDMs may be incorporated into the hazard integral to directly estimate the annual probability of exceeding a certain EDP (Gülerce and Abrahamson, 2010). For an EDP that depends on only a scalar ground motion IM, the hazard integral for the EDP can be written as:

$$v(EDP > y) = \left\{ N_{\min} \cdot \int \int \int f_M(M) f_R(M, R) f_\epsilon(\epsilon) \right. \\ \left. P(EDP > y | \hat{EDP}[IM(M, R, \epsilon)], \sigma_{\ln EDP}) dM dR d\epsilon \right\} \quad [20.7]$$

where $\hat{EDP}(IM(M, R, \epsilon))$ is the median EDP and $\sigma_{\ln EDP}$ is the standard deviation of $\ln(EDP)$ for a given IM (EDP is modeled as lognormal variate). This approach combines the site-specific ground motion hazard with the structural responses of interest from nonlinear dynamic analyses of the given structure. The final result of the hazard integral given in Equation 20.7 is a structural demand hazard curve representing the annual probability of exceeding a specified value of EDP.

20.2.3 Damage measure

The last step of the PEER-PBEE approach is to evaluate the distribution of DMs with respect to the EDP levels. In current practice and building codes, damage measures are typically not continuous, rather a set of discrete damage states (FEMA-273, 1997; Miranda and Aslani, 2003). To assess the extent of incurred damage, fragility functions are assigned for discrete damage states, which provide the probability of exceeding a damage state for a given EDP level (represented by the third chain in Fig. 20.1). With

structural capacity information (fragility curves), the PSDA results can be used to compute the annual frequency of exceeding a specified damage state. Several methodologies for generating seismic fragility curves have been developed over the years, including empirical, elastic-spectral, nonlinear static, and nonlinear dynamic approaches (Nielson and Des-Roches, 2007). Development of these component-specific and region-specific fragility functions is a topic of current and future research.

20.3 Probabilistic seismic demand analysis (PSDA)

PSDA (Luco, 2002) involves a series of steps including: the selection of ground motion dataset for nonlinear analyses; modeling a class of structures by varying important structural parameters considered in design; selection of proper EDPs and IMs; and formulating the relationship between EDP and IM (i.e. PSDMs). Each one of these steps has been examined by various researchers in the past decade and substantial progress has been made on vital parts of the process, such as nonlinear dynamic analysis of structural models and selection of suitable IMs and EDPs. Some controversial issues like selection and scaling of ground motions for nonlinear dynamic analyses and functional shape of PSDMs are topics of ongoing discussions and require further research efforts.

The selection of the time histories in nonlinear dynamic analyses is critical, since the spectral shape of the ground motion has a substantial effect on the structural demand due to nonlinear response of the soil and structure. It is common practice to select the ground motion time histories based on seismological properties, such as magnitude and distance to the fault. Similar site conditions, style of faulting, and directivity effects may also be considered in the selection process. Additionally, the scale factor required to scale the time series to the design ground motion level may be considered. In general, scale factors closer to unity are preferred and many ground motion experts recommend a limit on the amount of scaling applied (Watson-Lamprey and Abrahamson, 2006). The issues on ground motion selection and scaling for PSDA are closely related with the IM selection process. Selection of the proper IM is also critical in terms of reducing the uncertainty in prediction of the structural response, decreasing the number of required nonlinear dynamic analyses, and decreasing the effect of ground motion selection and scaling on the PSDM. An IM containing information on the ground motion characteristics (e.g. spectral shape) and the structure (e.g. fundamental period) is preferable, since it will reflect the main characteristics of seismological and structural parameters on the structural response effectively.

Many researchers focused on the selection process and proposed quantitative measures for choosing the most appropriate IM for the structure

among the candidate IMs. According to Luco and Cornell (2007), the desired properties of an IM include *efficiency*, *sufficiency*, *scaling robustness*, and *feasibility*. An *efficient* IM is defined as one that results in a relatively small variability in the structural response given the level of IM; therefore, the variance of the PSDM is the quantitative measure of the efficiency. An IM is defined as efficient if the variance of the PSDM is small and using an efficient IM will reduce the number of nonlinear dynamic analyses. A *sufficient* IM is defined as one that brings negligible conditional dependency to the structural response from the previous chain of Equation 20.1. In other words, if an IM is sufficient, the EDP will be independent of the earthquake parameters (earthquake magnitude, distance, etc.). Sufficiency of an IM is desirable, because it reduces the complexity in the PSDA calculation as well as the record selection procedure (Luco and Cornell, 2007). If an insufficient IM is used and the selected records do not represent the hazard at the site, the performance-based seismic evaluation will be biased (Tothong and Cornell, 2006).

Scaling robustness is used to define the possibility of introducing bias to the structural demand by scaling the time histories used in nonlinear dynamic analyses. If an IM is robust, no statistically strong relationship exists between the structural response and the scale factors used for the scaling of records. Finally, *feasibility* of the ground motion measure should be considered for selecting an appropriate IM. Since the annual rate of exceeding the IM is determined by PSHA, the IM should be computable in terms of scenario earthquake parameters. Hazard maps and hazard curves are available or easily computable for some IMs, but other IMs require more effort, or even structure-specific information for their determination. An IM with higher efficiency may be less desirable on the basis of feasibility (Giovenale *et al.*, 2004). Peak ground acceleration (PGA) and spectral acceleration ($S_a(T_1)$) at the fundamental period (T_1) of the structure are the most commonly adopted IMs in the structural engineering applications (Shome and Cornell, 1999; Nielson and DesRoches, 2007; Padgett *et al.*, 2008).

Following the selection of ground motions and IMs, sample models of the particular class of structures are generated by varying key design parameters (e.g. model dimensions, damping ratio, material strengths, etc.) within the allowable range of code requirements. The selected and scaled time histories are employed in nonlinear dynamic analyses of these sample models and important response parameters are monitored throughout the analysis. Typically, peak values of responses are recorded and plotted versus the peak value of the IM for that ground motion for each analysis (Mackie and Stojadinovic, 2003). The EDPs are selected among the peak values of monitored structural response parameters; however the results of previous analysis or experiments on that class of structures may be used as a basis

for selection. These parameters may be related to the global (e.g. drift ratio), intermediate (e.g. moments, deformations at the bearings, and column curvature ductility), or local (e.g. stresses) elements of the structure.

During the SAC project performed on steel moment frame buildings (FEMA, 2000), PSDMs were defined as lognormally distributed (Cornell *et al.*, 2002). Therefore, the relation between EDPs and IMs can be presented with a linear dependence in log–log space. The recommended functional form of the PSDMs is:

$$\ln(EDP) = c_1 \times \ln(IM) + c_2 + \sigma_{\text{PSDM}} \quad [20.8]$$

where c_1 and c_2 are the regression coefficients and σ_{PSDM} is the standard deviation of the model. After EDPs and IMs are selected, regression analyses are performed for each EDP–IM pair to estimate the regression coefficients c_1 and c_2 . Better predicting models (more efficient models) are selected according to the dispersion measures for the fit (Mackie and Stojadinovic, 2003). Recent studies (Nielson and DesRoches, 2007; Padgett *et al.*, 2008) found that the PGA is the most appropriate IM for highway bridges and the PSDMs coupling several EDPs with the horizontal PGA were presented.

20.4 Vector-valued probabilistic seismic hazard assessment (VPSHA)

In addition to the common IMs (PGA and $S_a(T_1)$), more complex parameters like vector-based IMs have been examined by past researchers (Shome and Cornell, 1999; Bazurro and Cornell, 2002). According to Baker and Cornell (2005), using only S_a as the IM is not adequate to characterize the ground motion intensity since there is a large variability in the structural response due to different ground motions with the same S_a at a particular period. The reason for that variability is the spectral shape; using $S_a(T_1)$ as IM does not take S_a values at other periods into account. S_a at other periods has an impact on inelastic structures (due to period lengthening) and multi-degree-of-freedom systems with multiple modal periods. Baker and Cornell (2005) proposed ε as a proxy for spectral shape and recommended the use of vector-based IM, $\text{IM}(S_a, \varepsilon)$, coupling $S_a(T_1)$ at the fundamental period of the structure with ε . If an EDP depends on two different IMs (IM_1 and IM_2), the rate of exceeding a specific value of EDP can be computed using the conditional distribution of EDP given IM_1 and IM_2 . Therefore Equation 20.6 becomes:

$$\begin{aligned} v(EDP > y | \text{IM}_1, \text{IM}_2) &= \int_{\text{IM}_2} f_{EDP}(EDP > y | \text{IM}_1, \text{IM}_2) \\ &\quad \times f_{\text{IM}_1}(\text{IM}_1 | \text{IM}_2) d\text{v}(\text{IM}_2) \end{aligned} \quad [20.9]$$

where $f_{\text{EDP}}(\text{EDP} > y | IM_1, IM_2)$ is the PSDM for a particular EDP and two IMs, $f_{IM_1}(IM_1 | IM_2)$ is the probability density function for the first IM conditioned on the second and $dv(IM_2)$ is the annual rate of exceeding a specified value of IM_2 (within some small increment). For EDPs depending on a vector of ground motion parameters, a vector-valued PSHA is required (Bazzurro and Cornell, 2002). The vector-valued seismic hazard integral for calculating the structural demand is (Gülerce and Abrahamson, 2010):

$$v(\text{EDP} > y) = \left\{ \begin{aligned} & N_{\min} \cdot \int \int \int \int f_M(M) f_R(M, R) f_{\epsilon_{IM_1}}(\epsilon_{IM_1}) f_{\epsilon_{IM_2}}(\epsilon_{IM_2} | \epsilon_{IM_1}) \\ & P(\text{EDP} > y | \hat{E}DP[IM_1(M, R, \epsilon_{IM_1}), IM_2(M, R, \epsilon_{IM_2})], \\ & \sigma_{\ln \text{EDP}}) dM dR d\epsilon_{IM_1} d\epsilon_{IM_2} \end{aligned} \right\} \quad [20.10]$$

where ϵ_{IM_1} and ϵ_{IM_2} are the epsilons for IM_1 and IM_2 , $f_{\epsilon_{IM_1}}(\epsilon_{IM_1})$ is the probability density function for ϵ_{IM_1} , and $f_{\epsilon_{IM_2}}(\epsilon_{IM_2} | \epsilon_{IM_1})$ is the probability density function for ϵ_{IM_2} conditioned by ϵ_{IM_1} . The form in Equation 20.10 differs from the formulation given by Bazzurro and Cornell (2002) such that the double integral in Equation 20.10 is carried out over ϵ_{IM_1} and ϵ_{IM_2} , rather than IM_1 and IM_2 . The modified formulation clearly shows that the correlation of the variability of the intensity measures needs to be considered (Gülerce and Abrahamson, 2010). $f_{\epsilon_{IM_1}}(\epsilon_{IM_1})$ is given by the standard normal distribution, whereas $f_{\epsilon_{IM_2}}(\epsilon_{IM_2} | \epsilon_{IM_1})$ is conditioned on the value of ϵ_{IM_1} and depends on the correlation of ϵ_{IM_1} and ϵ_{IM_2} . The covariance of ϵ_{IM_2} with respect to ϵ_{IM_1} is computed from the correlation of the normalized residuals from the GMPE regression analysis. The probability density function for ϵ_{IM_2} can be defined as a function of ϵ_{IM_1} as:

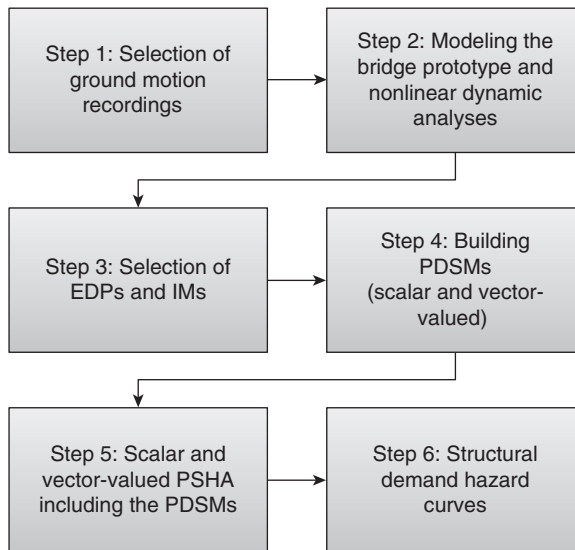
$$\epsilon_{IM_2}(T_2) = \rho \times \epsilon_{IM_1}(T_1) \pm \sigma_{\epsilon_{IM_2} | \epsilon_{IM_1}} \quad [20.11]$$

where ρ is the correlation coefficient and $\sigma_{\epsilon_{IM_2} | \epsilon_{IM_1}}$ is the standard deviation of the correlation. A vector-IM-based analysis facilitates the record selection process, assuming that no dependence upon any other parameters is remained (Baker and Cornell, 2005). Using a vector-valued IM also decreases the variability in the PSDM and thus the number of required nonlinear analyses to develop the PSDM model. However, to perform vector-valued PSHA, the correlation of residuals of the ground motion model (or models) across the periods need to be known. Only a few of the recent GMPEs provide these correlation coefficients to be used in vector-valued analysis (e.g., Abrahamson and Silva, 2008; Gülerce and Abrahamson, 2011). Another drawback of the vector-valued IMs is the inability of the standard PSHA software packages to run vector-valued calculations.

20.5 Performance-based seismic evaluation of ordinary highway bridges

During 2004–2008, a detailed analytical study on the seismic response of ordinary highway bridges was conducted at UC Davis, California. The primary objective of the study was to quantify the effects of near-fault vertical ground motions on the seismic response of single bent, two-span highway overpasses. Results from the study were published in a series of papers; Kunnath *et al.* (2008), Gülerce and Abrahamson (2010), and Gülerce *et al.* (2012), focusing mainly on the structural demand due to vertical ground motion component. The study followed the PEER-PBEE framework by developing PSDMs for the particular structures using scalar- and vector-valued IMs, and by performing many scalar- and vector-valued PSHA analyses to address the question of when the vertical component should be included in design. Results of that study will be used as an example to explain an application of the PBEE methodology. The flowchart of the major steps is provided in Fig. 20.2.

The initial step of the process was the selection of ground motion recordings to be used in the nonlinear dynamic analyses of the bridge prototype. A subset of 56 near-fault recordings (rupture distance smaller than 50 km) having the strongest horizontal components (horizontal PGA is greater than or equal to 0.5 g) was selected from the PEER-NGA database (Chiou



20.2 PBEE framework for evaluating the vertical ground motion effects on the structural response.

et al., 2008). Since the primary objective of the study was to address the effect of vertical ground motions, more near-fault recordings with either large or small vertical components compared to the horizontal component were added to the dataset. The resulting set of 114 ground motions (228 horizontal and 114 vertical components) were used in the nonlinear dynamic and regression analyses (Step 1 in Fig. 20.2).

Kunnath *et al.* (2008) used a portion of the widening project of the Camino Del Norte Bridge located in California as a typical ordinary standard highway bridge and representative of the class of structures. The prototype was a single bent bridge with two spans of 30.95 m and 30.52 m in length and many configurations were generated by changing the dimension of the bridge model without violating the code specifications on allowed dimensional and balanced stiffness requirements. The nonlinear dynamic analyses were carried out in two stages (Gülerce *et al.*, 2012): in Stage 1, only horizontal components of the motion were applied as a base case, while in Stage 2, both horizontal and vertical components were applied simultaneously to monitor the increase in the response due to the inclusion of the vertical component (Step 2 in Fig. 20.2). Analysis results of the bridge configuration with horizontal first-mode fundamental periods of 0.27 and 0.46 seconds (in longitudinal and transverse directions, respectively) and vertical first-mode fundamental period of 0.12 seconds are presented in the following sections.

Results of the nonlinear dynamic analyses showed that some structural response parameters were amplified significantly when the vertical accelerations were incorporated, such as the axial force demand in the column and moment demands in the girder both at the mid-span and at the face of the bent cap (Kunnath *et al.*, 2008). On the other hand, some commonly examined structural response parameters such as lateral displacements of the deck, were not influenced significantly by the inclusion of vertical motions (Gülerce *et al.*, 2012). The parameters that were affected significantly due to vertical accelerations were selected as the structural response measures to be used in the PBEE framework. Similar to the previous practice (Mackie and Stojadinovic, 2003), peak values of the response history were chosen as EDPs (after normalizing with the dead load values to offer a rational basis for comparison of different bridge configurations). Type 1-EDPs were defined using positive peaks of the axial load in the column, moments at the mid-span and at the support as:

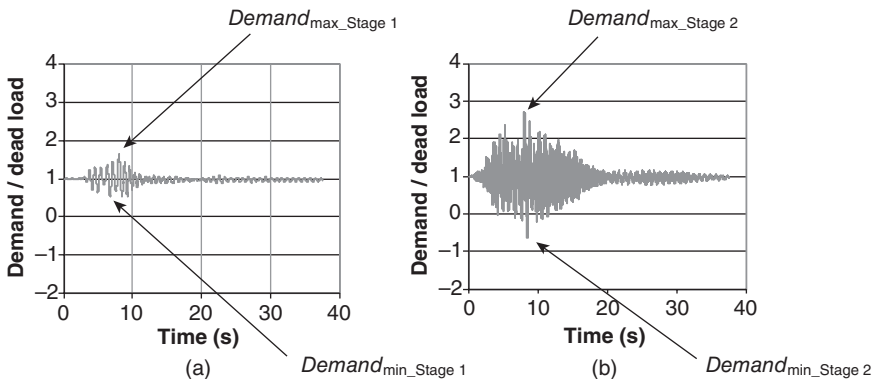
$$Type1-EDP = \frac{(Demand)_{\max, Stage 1}}{DL} \quad [20.12]$$

where $(Demand)_{\max, Stage 1}$ is the positive extreme value of the demand time history at the base case (without the vertical accelerations) and DL is the corresponding dead load value as shown in Fig. 20.3(a).

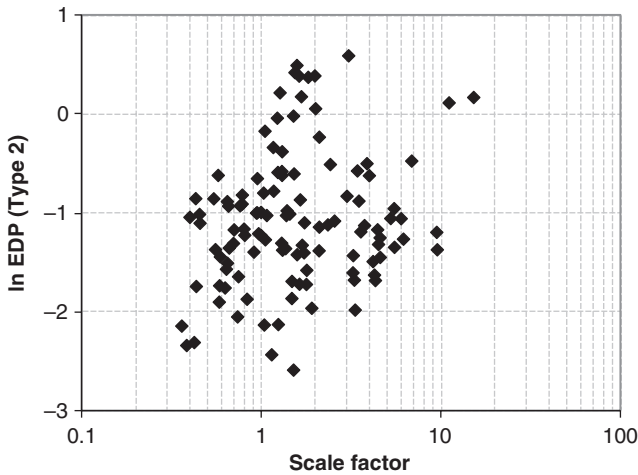
Type 2-EDPs were defined using the negative peaks of the moment demand time histories at the mid-span and at the support to account for the effects of negative bending due to vertical ground motions as:

$$\text{Type2-EDP} = \frac{(Demand)_{\min, \text{Stage1}} - DL}{DL} \quad [20.13]$$

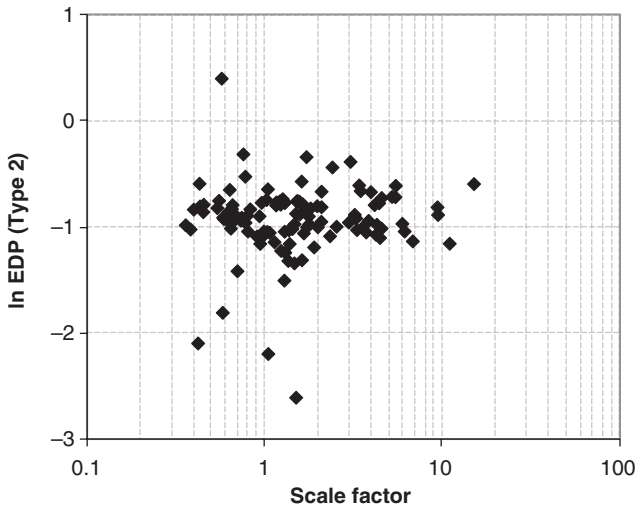
where $(Demand)_{\min, \text{Stage1}}$ is the negative extreme of the demand time history at the base case (Gülerce *et al.*, 2012; Fig. 20.3(b)). To define a possible relation between the ground motion characteristics and the EDPs, characteristics of the ground motions which produced the largest adverse effects were studied systematically. As mentioned in the previous sections, $S_a(T_1)$ at the fundamental period of the structure along with PGA is the most commonly preferred IM due to its efficiency, sufficiency, and feasibility. S_a at the vertical, transverse and longitudinal periods of the bridge were selected as IMs for this study. The effects of ground motion selection and scaling, defined by the scaling robustness property of an IM, can be checked by evaluating the dependency of EDP on the scale factor. The ground motions used in this study were scaled to an example target spectrum (median response spectrum from Abrahamson and Silva (1997) GMPE for M7, D5km) to examine this property using two different approaches; in the first approach, only one scale factor per set (a set is defined as two horizontal and vertical ground motion components) was used, while in the second approach, separate scale factors for different components of the ground motion record sets were used. The distribution of an example Type 2-EDP with respect to the scale factors is given in Fig. 20.4 for one-scale-factor case and in Fig. 20.5 for separate-scale-factor case, respectively.



20.3 Typical demand-time histories from (a) Stage 1 and (b) Stage 2 of nonlinear dynamic analyses.



20.4 Distribution of Type 2-EDP (in natural log units) vs. scale factor for the one-scale-factor case.



20.5 Distribution of Type 2-EDP (in natural log units) vs. scale factor for the separate-scale-factor case.

The scatter of the EDP in Fig. 20.4 indicates that there is large variability in the response for the records that have similar scale factors. Therefore, even if the scale factor for a record is near unity, that record may not give a good estimate of the average response. The variability of response is not sensitive to the scaling of the records, since the amount of scatter in the EDP does not change with the scale factor. By using separate scale factors

for different ground motion components, the scatter in EDP is reduced significantly (Fig. 20.5). Still, some of the records with large-scale factors produce responses close to average, and some of the records with scale factors close to unity generate response values greater than or less than the average. That leads to the conclusion that it is possible to obtain unbiased results even for large-scale factors if the records are selected appropriately and that factors other than the scale factor, such as proper selection of IMs, are critical. Figures 20.4 and 20.5 validate the scaling robustness property of the selected IMs and ground motion recordings in the UC Davis study.

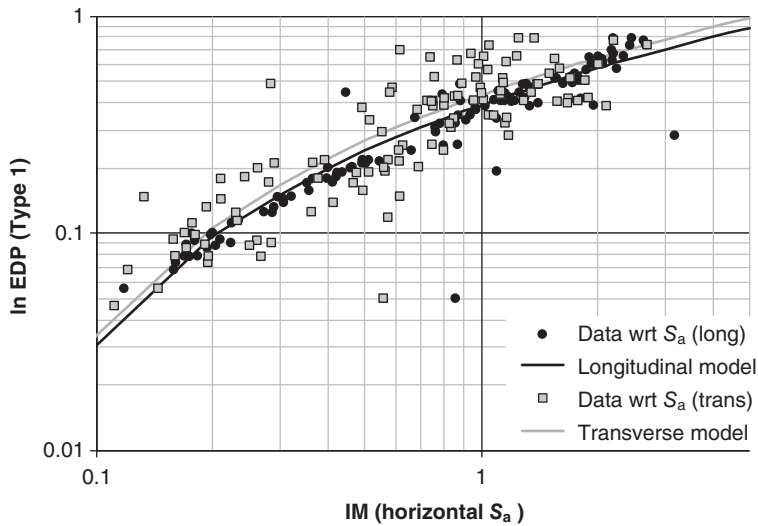
20.5.1 PSDMs for scalar IMs

Different PSDMs for the horizontal ground motion effects on the seismic response of ordinary highway bridges are available in the literature (Mackie and Stojadinovic, 2003; Nielson and DesRoches, 2007; Padgett *et al.*, 2008). The UC Davis study extends the previous studies in two important aspects: the analysis focused on the combined effects of vertical and horizontal ground motion components, and the functional form of the seismic demand models was not pre-determined as given in Equation 20.8 (Gülerce *et al.*, 2012). Nonlinear dynamic analyses results were used in regression analysis to develop scalar-valued PSDMs given only horizontal IMs for the first stage and to develop vector-valued PSDMs given both horizontal and vertical IMs for the second stage (Step 4 in Fig. 20.2). From preliminary analyses, it was found that a quadratic dependence on the IMs was applicable to (e.g. Fig. 20.6) Type 1-EDPs, whereas a linear dependence was adequate for Type 2-EDPs, as given in Equations 20.14 and 20.15.

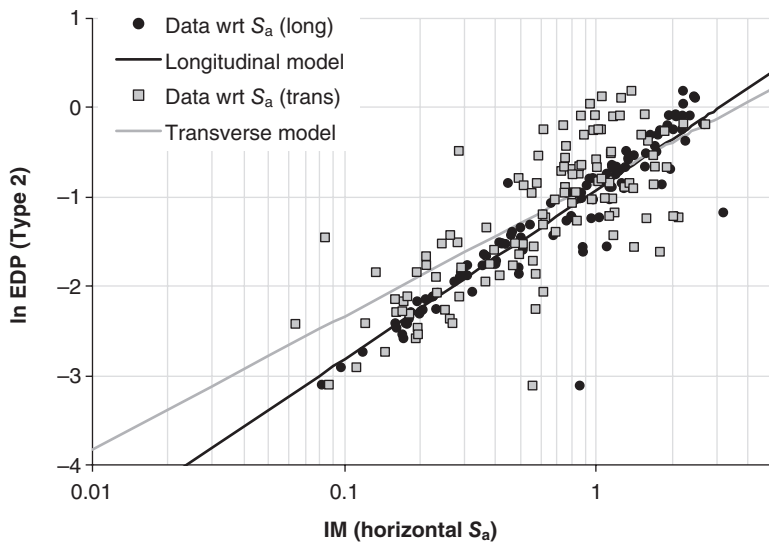
$$\ln[EDP(\text{Type 1})] = c_1 \times [\ln(IM) - c_2]^2 \pm \sigma_{\text{Stage1}} \quad [20.14]$$

$$\ln[EDP(\text{Type 2})] = c_3 + c_4 \ln(IM) \pm \sigma_{\text{Stage1}} \quad [20.15]$$

where c_1 , c_2 , c_3 and c_4 are the model coefficients estimated using nonlinear regression, and σ_{Stage1} is the standard deviation of the scalar PSDMs at the first stage. Note that the functional form of the PSDMs for Type 2-EDPs is identical to the form recommended by Cornell and Krawinkler (2000). Figure 20.6 presents the actual data points (in terms of the natural logarithms of Type 1-EDPs) and two PSDMs that model the same structural response data using different IMs; S_a at the longitudinal period of the bridge and S_a at the transverse period of the bridge. The actual data points in Fig. 20.6 clearly show the quadratic dependence of Type 1-EDPs to the selected IMs. The scatter in the actual structural response data increases when the horizontal S_a at the transverse period of the bridge model was used. Gülerce *et al.* (2012) analyzed all bridge configurations and concluded that the



20.6 Distribution of Type 1-EDP (in natural log units) vs. the transverse and longitudinal IMs.



20.7 Distribution of Type 2-EDP (in natural log units) vs. the transverse and longitudinal IMs.

horizontal S_a at the longitudinal period is a better predictor of the response in terms of efficiency. Figure 20.7 presents the actual data points and two PSDMs that model the same structural response data using different IMs; S_a at the longitudinal period of the bridge and S_a at the transverse period

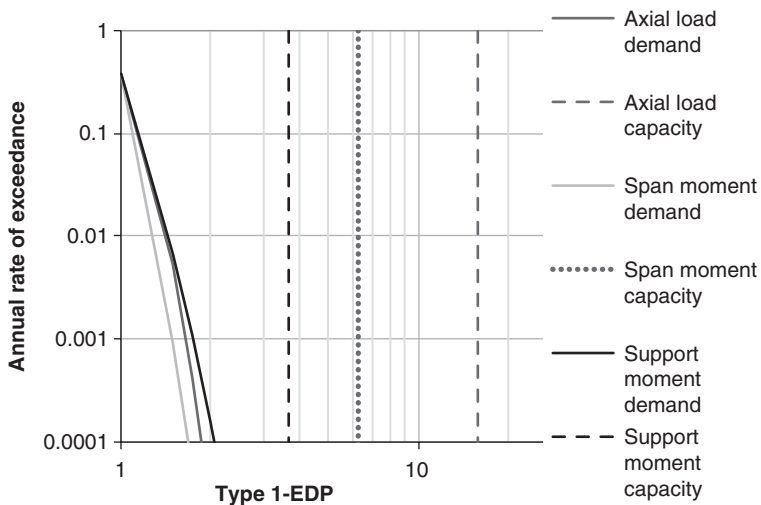
of the bridge for Type 2-EDPs. For this type of EDPs, no curvature of the actual data was observed and the linear dependence in log-log space was sufficient. Similar to Type 1-EDPs, S_a at the longitudinal period of the bridge model was found to be more efficient for this type of EDP. The variations in the bridge model, regression coefficients, and standard deviations of PSDMs for both longitudinal and transverse IMs can be found in Gülerce *et al.* (2012).

20.5.2 Scalar PSHA for structural demand

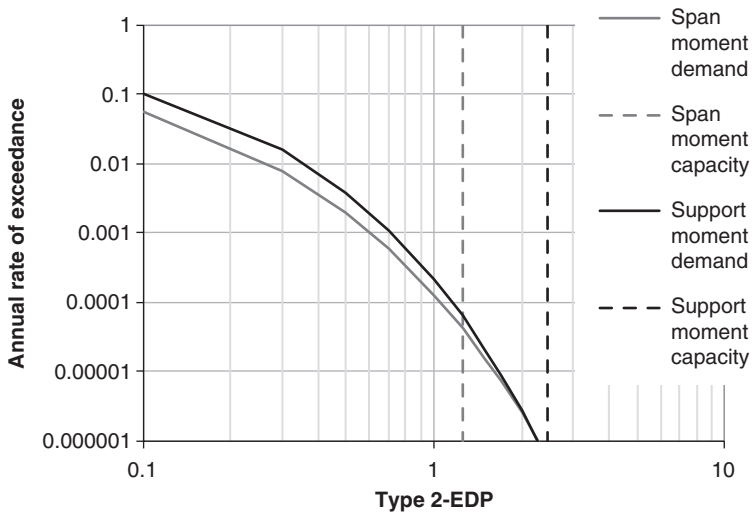
Using the Type 1- and Type 2-PSDMs developed at Stage 1 (no vertical accelerations), PSHA was conducted by directly incorporating the PSDMs into the PSHA integral. For Type 1- and Type 2-EDPs that depend on scalar IMs only, Equation 20.7 was modified as (Gülerce and Abrahamson, 2010):

$$v(EDP\_Stage1 > z) = \left\{ \begin{array}{l} N_{\min} \cdot \int \int \int f_M(M) f_R(M, R) f_\epsilon(\epsilon) \\ P(EDP\_Stage1 > z | \hat{EDP\_Stage1} \\ (S_a(M, R, \epsilon)), \sigma_{Stage1}) dM dR d\epsilon \end{array} \right\} \quad [20.16]$$

where $v(EDP\_Stage1 > z)$ represents the annual rate of exceeding the specified value of EDP without the effect of vertical accelerations. Figure



20.8 Structural demand hazard curves at Stage 1 for an example near fault site for Type 1-EDPs.



20.9 Structural demand hazard curves at Stage 1 for an example near fault site for Type 2-EDPs.

20.8 shows the structural demand hazard curves for Type 1-EDPs (axial load demand and positive moment demands at the mid-span and at the support) for a near-fault site. Similarly, Fig. 20.9 presents the structural demand hazard curves for Type 2-EDPs (negative moment demands at the mid-span and at the support) for the same site. The broken lines in both figures indicate the capacity defined as the onset of inelastic deformation/ yielding recalling that the girders in a highway bridge are designed to remain elastic under seismic action (Kunnath *et al.*, 2008) for each EDP. Details of the analysis, fault system, and source characteristics used for the assessment were explained in Gülerce and Abrahamson (2010). According to Figs 20.8 and 20.9, none of the Type 1-EDPs exceed the corresponding capacities at Stage 1. There is a chance of exceeding the capacity for Type 2-EDPs, but these probabilities are small (less than 0.0001 annual probabilities).

20.5.3 Vector-valued PSDMs and vector-valued PSHA

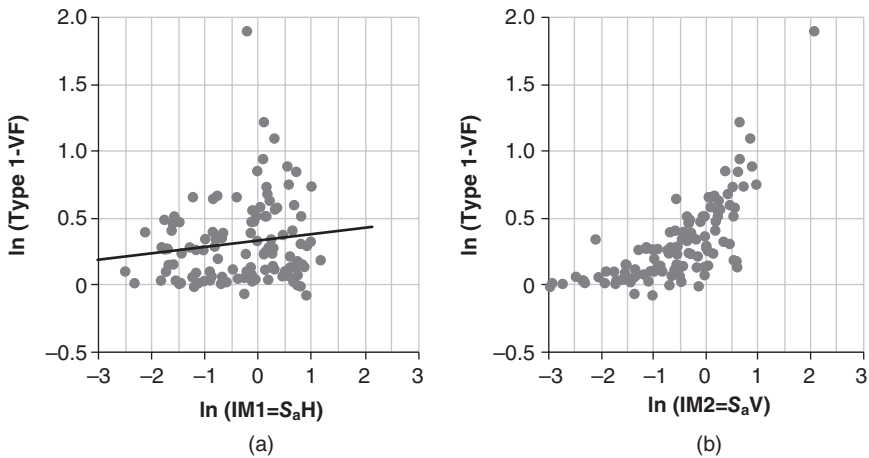
The increase in the EDPs (defined by Equations 20.12 and 20.13) when the vertical ground motions were included in the nonlinear dynamic analyses (Stage 2) can be described by vertical factor (VF) (Gülerce *et al.*, 2012). Similar to the EDPs at Stage 1, two types of vertical factors were defined and normalized as shown in Equations 20.17 and 20.18. Type 1-VF represents the amplification for Type 1-EDPs, while Type 2-VF is for Type 2-EDPs:

$$Type1 - VF = \frac{\frac{(Demand)_{\max, Stage2}}{DL}}{Type1 - EDP} \quad [20.17]$$

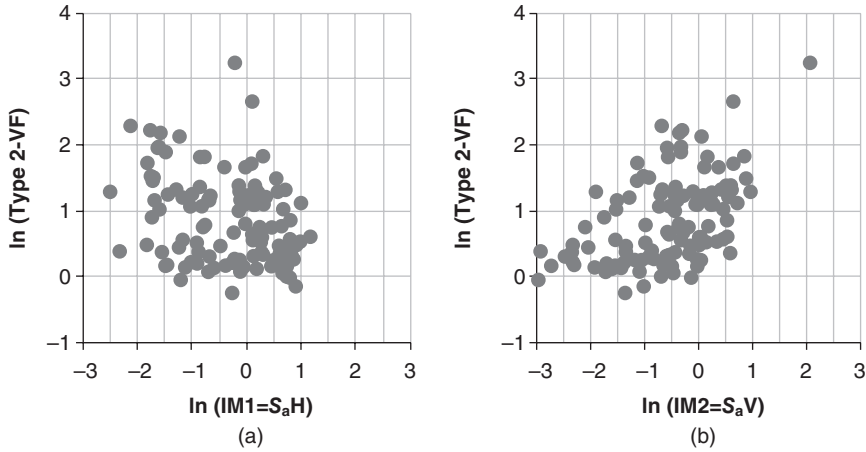
$$Type2 - VF = \frac{\frac{(Demand)_{\min, Stage2} - DL}{DL}}{Type2 - EDP} \quad [20.18]$$

where $(Demand)_{\max, Stage2}$ and $(Demand)_{\min, Stage2}$ are the positive and negative extremes of the demand versus time history at Stage 2. Since both horizontal and vertical ground motion components have impact on the structural response at this stage, distribution of the vertical factors with respect to the vertical and horizontal S_a was checked to investigate the trends of the data as shown in Figs 20.10 and 20.11 for Type 1-VFs and Type 2-VFs, respectively.

Figures 20.10 and 20.11 reveal that both types of vertical factors increase as the vertical S_a increases. The structural response is also dependent on the horizontal S_a , but this dependence is weaker than the dependence on the vertical S_a . The PSDMs were constructed using different functional forms for Type 1- and Type 2-VFs by including two IMs instead of one; the horizontal S_a at the horizontal (either transverse or longitudinal) period of the bridge (S_{aH}) and the vertical S_a at the vertical period of the bridge (S_{aV}). The PSDMs for the Type 1-VFs and Type 2-VFs are given in Equations 20.19 and 20.20, respectively.



20.10 Distribution of Type 1-VF with respect to (a) horizontal spectral accelerations and (b) vertical spectral acceleration.



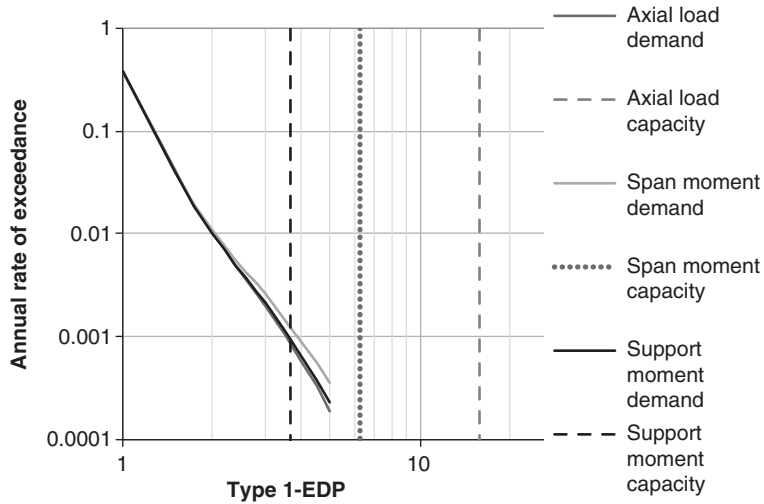
20.11 Distribution of Type 2-VF with respect to (a) horizontal spectral accelerations and (b) vertical spectral accelerations.

$$\ln(\text{Type1-VF}) = \left\{ \begin{array}{l} 0 \rightarrow \text{if } \frac{S_{av}}{S_{aH}} \leq \ln(c_1) \\ 0 \rightarrow \text{if } S_{aH} \leq 0.1 \\ \left[\ln\left(\frac{S_{av}}{S_{aH}}\right) - \ln(c_1) \right] \times [c_2 + c_3 \times \ln(S_{aH})] \pm \sigma_{VF} \rightarrow \text{otherwise} \end{array} \right\} \quad [20.19]$$

$$\ln(\text{Type 2-VF}) = \left\{ \begin{array}{l} 0 \rightarrow \text{if } \frac{S_{av}}{S_{aH}} \leq c_4 \\ c_5 + c_6 \times \ln\left(\frac{S_{av}}{S_{aH}}\right) \pm \sigma_{VF} \rightarrow \text{otherwise} \end{array} \right\} \quad [20.20]$$

where c_1 to c_6 are the model coefficients determined by nonlinear regression (Gülerce *et al.*, 2012). Since PSDMs depend on a vector of IMs, vector-valued PSHA was required at this stage. Gülerce and Abrahamson (2010) modified the vector-valued hazard integral given in Equation 20.10 for structural demand as:

$$v(\text{EDP\_Stage2} > z) = \left\{ \begin{array}{l} N_{\min} \cdot \int \int \int \int f_M(M) f_R(M, R) f_{\epsilon_H}(\epsilon_H) f_{\epsilon_V}(\epsilon_V | \epsilon_H) \\ P[\text{EDP\_Stage2} > z | \hat{EDP}(S_{aH}(M, R, \epsilon_H), \\ S_{aV}(M, R, \epsilon_V)], \sigma_{VF}] dM dR d\epsilon_H d\epsilon_V \end{array} \right\} \quad [20.21]$$



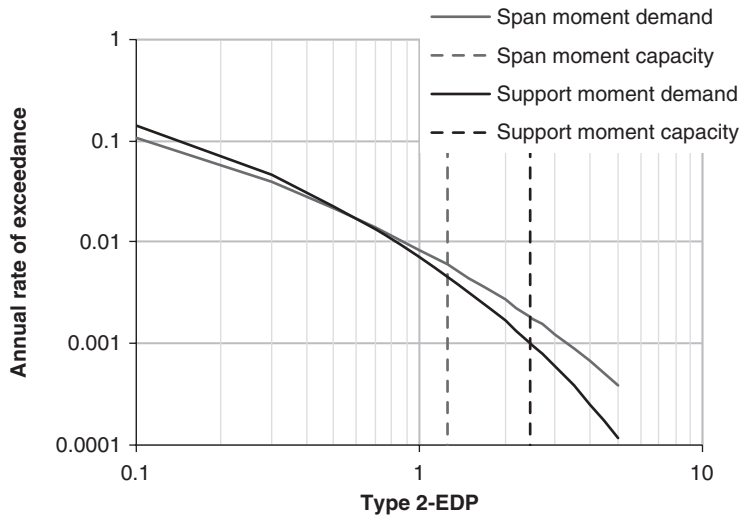
20.12 Structural demand hazard curves at Stage 2 for an example near fault site for Type 1-EDPs.

where $v(EDP_{Stage2} > z)$ represents the annual rate of exceeding the specified value of EDP including the vertical accelerations, S_{aH} is the horizontal S_a at the horizontal period of the structure, S_{aV} is the vertical S_a at the vertical period of the structure, ϵ_H is the epsilon for horizontal ground motion model, ϵ_V is the epsilon for the vertical ground motion model, $f_{\epsilon_H}(\epsilon_H)$ is the probability density function for ϵ_H , and $f_{\epsilon_V}(\epsilon_V|\epsilon_H)$ is the probability density function for ϵ_V conditioned on ϵ_H . The structural demand hazard curves for Type 1-EDPs and Type 2-EDPs for the same site at Stage 2 are presented in Figs 20.12 and 20.13, respectively.

Figure 20.12 shows that the probability of exceeding the axial load capacity at the column and the positive mid-span moment capacity increased significantly, but these EDPs still remained below the capacity. Annual probability of exceeding the support moment capacity (for both Type 1- and Type 2-EDPs) was computed as 0.001 at this stage (Figs 20.12 and 20.13). Negative mid-span moment demand was identified as the most critical parameter under the influence of vertical accelerations and the annual probability of exceeding the negative mid-span moment capacity was substantial (equal to 0.01) at Stage 2 (Fig. 20.13).

20.6 Future trends

Recent research results and ongoing discussions on the main components of the PBEE framework including the PSHA, development of PSDMs, use



20.13 Structural demand hazard curves at Stage 2 for an example near fault site for Type 2-EDPs.

of scalar or vector-valued PSDMs to build structural demand hazard curves, and performance objectives were discussed in the previous sections. Even though numerous studies focusing on the vital parts of the problem in the last decade brought in major improvement, there is still significant need for research efforts, especially on the PSHA procedure and its applications. Standardized procedures for estimating the annual rate of exceeding the specific levels of ground motion IMs are crucial in the implementation of the PBEE methodology. PSHA and its main components (seismic source characterization models and ground motion prediction models) are rapidly evolving with the increasing number of special projects (e.g. nuclear power plants, bridges and high-rise structures). However, for many faults around the world, critical seismic source parameters, such as fault geometry, mean characteristic earthquake, slip rate, reoccurrence intervals, time since the most recent large earthquake, etc., are either unknown or poorly constrained by existing seismological data. The current practice of using areal sources to overcome the obstacle of the unknown source characteristics introduces significant uncertainty to seismic hazard calculations in many regions; therefore, studies on developing improved seismic source models should be encouraged.

On the other hand, uncertainty introduced by ground motion prediction models is considerably higher than any other parameter models included in the hazard integral. New and improved GMPEs in terms of additional prediction parameters (depth of the source, basin effects, site-dependent

standard deviations, etc.), statistical approach, and a well-constrained global database were developed for California (Abrahamson *et al.*, 2008). In the near future, new models will be developed for other tectonic environments (e.g. subduction zones). The applicability of these new and improved global GMPEs in PSHA studies of other regions is an important issue and should be investigated by comparing the magnitude, distance, and site effects scaling of these global models by regional datasets.

Recent studies (Baker and Cornell, 2005; Gülerce and Abrahamson, 2010) suggested that use of scalar-IMs may not be adequate in characterizing the ground motion intensity and nonlinear structural response, thus there is a need to develop vector-valued IMs and PSDMs. Integrating the vector-valued PSDMs into the PSHA integral will require vector-valued hazard computations, therefore standard PSHA software packages should be modified to develop the capability of performing vector-valued calculations. Also, the correlation of residuals of the ground motion model (or models) across the periods should be provided by the developers of future predictive models to be able to perform vector-valued probabilistic seismic hazard and demand assessments. Finally, controversial issues such as selection and scaling of ground motions for nonlinear dynamic analysis and functional shape of PSDMs are topics of ongoing discussions and require further research efforts.

20.7 Acknowledgments

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Incremental dynamic analysis (IDA) applied to seismic risk assessment of bridges

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Abstract: This chapter focuses on the use of incremental dynamic analysis (IDA) for seismic performance and risk assessment of reinforced concrete bridges. Different record selection methods, including the epsilon-based, conditional mean spectrum (CMS)-based and uniform hazard spectrum (UHS)-based methods, are used for the prediction of the structural responses using IDA. In addition, the influences of including different earthquake types (i.e., crustal, subduction interface and inslab earthquakes) on the IDA results are studied. Guidelines for determining different damage states in bridge columns and developing fragility curves are provided. The important aspects of structural modelling of bridges for IDA are presented and the use of IDA for the seismic risk assessment of a 4-span bridge is demonstrated through an example.

Key words: incremental dynamic analysis (IDA), seismic risk, seismic performance, bridges, record selection, conditional mean spectrum (CMS), damage states, fragility curves.

21.1 Introduction

Seismic risk can be expressed as the potential economic, social and environmental consequences of seismic events that may occur in a specified period of time. In order to quantify the seismic risk associated with a certain structure, one needs to combine two important elements: seismic hazard and seismic fragility analyses. The determination of the seismic hazard is accomplished by using conventional probabilistic seismic hazard analysis (PSHA). For fragility analysis it is essential to predict the general behavioural aspects, consistent with experimental studies (e.g., Banerjee and Shinozuka, 2008a,b; Straub and Der Kiureghian, 2008). Fragility analysis for bridges is typically carried out using nonlinear dynamic or static analysis, such as incremental dynamic analysis (IDA) (Vamvatsikos and Cornell, 2002) and probabilistic seismic demand analysis (PSDA) (e.g., Gardoni *et al.*, 2003; Karim and Yamazaki, 2003; Lupoi *et al.*, 2003; Mackie and Stojadinovic, 2003; Nielson and DesRoches, 2007a,b; Padgett and DesRoches,

2008). The results are often presented in the form of fragility curves. The IDA procedure provides valuable information regarding the probabilistic distribution of damage at different seismic intensity levels, which can be used for the development of fragility curves and seismic risk assessment of structures. However, available research on the application of IDA for the seismic risk assessment of bridges is limited. In a study by Mander *et al.* (2007), IDA was used for the financial seismic risk assessment of bridges. However, studies on bridges have not considered the important effects of record selection and spectral shapes on the IDA predictions. In addition, peak ground acceleration (PGA) has been used in earlier studies as an intensity measure (IM). The use of PGA as an IM and scaling the ground motion records in IDA may result in biased predictions of the spectral shape effects and the structural responses (Vamvatsikos and Cornell, 2002). More research on the use of IDA for the risk assessment of bridges is needed.

This chapter focuses on IDA for seismic performance evaluation and risk assessment of bridges. IDA involves numerous inelastic time history analyses performed using a set of ground motion records, each scaled up or down (Vamvatsikos and Cornell, 2002). The IDA procedure has been adopted by some guidelines including the ATC-63 provisions (ATC-63, 2008) to determine the seismic performance, collapse capacity and fragility of buildings.

IDA provides a thorough evaluation of the seismic responses (seismic demands) for a wide range of seismic intensity levels corresponding to different damage states from yielding up to collapse.

In addition, IDA provides information regarding possible structural responses due to different ground motion records, including estimates of the median as well as the variability of the seismic responses, required for probabilistic seismic performance assessment of structures and seismic risk analysis (e.g., development of fragility curves and prediction of the annual rate of collapse). In this chapter the main concepts of IDA are briefly discussed and the general modelling requirements for IDA are explained. The record selection is an important issue which can significantly affect the IDA predictions. The effects of different record selection methods and including different earthquake types on the IDA results are presented. The concepts of conditional mean spectrum (CMS) are explained and its application to the record selection for IDA is demonstrated. Three different CMS curves are developed for a site such as Vancouver where the occurrence of three different earthquake types is possible. The concept of using fragility curves is discussed and the use of IDA to develop the fragility curves and to predict the probability of exceeding different damage states is explained. The application of IDA for the seismic risk assessment of a continuous 4-span bridge is presented as an example.

21.2 Incremental dynamic analysis (IDA)

21.2.1 Damage measures (DMs) and IMs

The IDA results are commonly presented using an IM versus a damage measure (DM) of interest. IM is a non-negative scalable quantity, which is obtained from accelerograms. In IDA, it is monotonically increased with a scale factor to attain various seismic excitation levels (Vamvatsikos and Cornell, 2002). Examples of scalable IMs which are commonly used in the IDA include the PGA, peak ground velocity (PGV) and the 5% damped spectral acceleration at the structure's first-mode period T_1 ($S_a(T_1)$). The use of $S_a(T_1)$ as IM is often recommended and used in different studies and guidelines (ATC-63, 2008; Vamvatsikos and Cornell, 2002).

DM is defined as a non-negative scalar quantity that characterizes the response of the structure to seismic excitations and can be evaluated from the output of the nonlinear dynamic analysis. Selecting an appropriate DM depends on the objective of the analyses and the characteristics of the structure. Some of the common choices include the maximum base shear, maximum column ductility demands, global cumulative hysteretic energy and maximum column drift ratio. For bridges, the maximum drift ratio of the critical columns is typically used for the DM parameter.

21.2.2 IDA curves

An IDA curve is the plot of the DM variable versus the IM parameter. These curves demonstrate the state of the DM parameter at different IM levels of the input records. This enables the study of the seismic demand parameters from low seismic intensity levels prior to yielding of the structure up to ultimate performance levels, such as dynamic instability of the structure (i.e., collapse), buckling or fracture of the steel bars, shear failure, etc. Each non-linear dynamic analysis performed at each seismic intensity level is presented by a point in the IDA curve. This point demonstrates the maximum seismic demand (in terms of the DM parameter) obtained at the corresponding IM level of the ground motion record. The full IDA curve then can be developed by fitting a curve through the points computed for different IM levels. Various types of IDA responses, including hardening behaviour, softening behaviour, waving behaviour and structural resurrection, may be observed (Vamvatsikos and Cornell, 2002).

21.2.3 Summarizing IDA results

Different ground motion records in IDA often result in different response predictions which are quite dissimilar, due to complex structural behaviour and large record-to-record variability. Therefore, it is essential to summarize

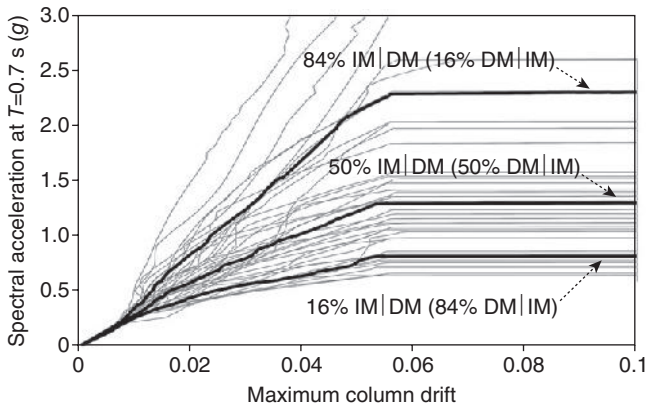
the large amount of data obtained through all IDA curves by quantifying the randomness introduced by the records. Appropriate techniques should be used to reduce the entire data to the probability distribution of DM given IM and to the probability of exceeding any specific limit-state given the IM level. The use of percentiles (fractiles) are the most appropriate approach to summarize the IDA, since at the point of dynamic instability, introduced by flatlines in the IDA curves, the DM values are infinite and therefore the use of mean values or similar parameters is not possible (Vamvatsikos and Cornell, 2002).

In order to evaluate the IDA result the median responses are determined along with the predicted dispersion of the results from different ground motion records. The IDA results thus can be summarized in percentiles, including median (50% percentile), 16% and 84% percentiles. With the assumption of a lognormal distribution of maximum drift ratio as a function of $S_a(T_1)$, the median (i.e., 50% percentile) is the natural 'central value' and the 84% and 16% percentiles correspond to the median times $e^{\pm \text{dispersion}}$, where 'dispersion' is the standard deviation of the logarithm of the values (Jalayer and Cornell, 2003). These percentile curves are much smoother than the individual IDA curves and can better represent the overall behaviour of a structure.

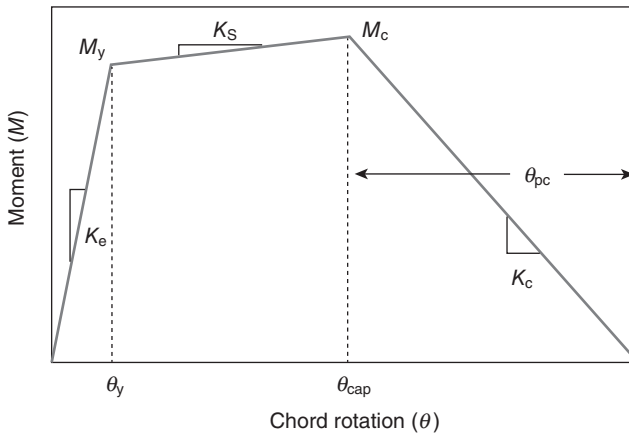
A summary of the IDA curves can be expressed either by the values of DM given IM or the values of IM given DM. The first method provides the distribution of DM in the structure at a given IM level, while the second approach provides the distribution of IM that causes a given level of DM in the structure. It has been shown that the line connecting the $x\%$ percentiles of DM given IM is similar as the one connecting the $(100 - x)\%$ percentiles of IM given DM (Vamvatsikos and Cornell, 2004). However for some cases with significant waiving and structural resurrection behaviour the two methods may result in different predictions (Tehrani and Mitchell, 2012d). An example of IDA curves summarized into 16%, 50% and 84% percentiles is shown in Fig. 21.1.

21.3 Structural modelling for incremental dynamic analysis (IDA)

In IDA, the intensity of the ground motion records is increased until they cause structural instability which is referred to as structural collapse. Therefore, the structural models in the IDA should be capable of simulating structural collapse and should directly or indirectly consider all significant deterioration modes that contribute to collapse including the stiffness, strength and inelastic deformations under reversed cyclic loading. The most important structural parameters that influence the IDA predictions include the plastic deformation capacity, θ_{cap} , and the post-capping rotation capacity,



21.1 An example of IDA curves developed using 44 crustal records for a 4-span bridge (fractile 16%, 50% and 84% IDA curves are shown by heavy lines) (Tehrani *et al.*, 2012).



21.2 Backbone curve parameters (adapted from ATC-63, 2008).

θ_{pc} (Ibarra *et al.*, 2005). These parameters are used to define a component backbone curve, as shown in Fig. 21.2.

The distinction between cyclic and in-cycle degradation is an important issue that should be considered in modelling (ATC-62, 2008). Dynamic response of systems with cyclic strength degradation is generally stable, while in-cycle strength degradation can lead to lateral dynamic instability (i.e., collapse). In reinforced concrete components, material nonlinearities that can lead to in-cycle strength degradation include concrete crushing, shear failure, buckling or fracture of longitudinal reinforcement, and splice failures (ATC-62,

2008). For IDA the in-cycle strength degradation is very important for prediction of the collapse capacity and the global dynamic instability of the structures, and is often considered by the negative post-capping stiffness defined in the component backbone curves (see Fig. 21.2).

Modern bridges are designed and detailed to meet the seismic code requirements for ductile response so that the flexural response governs the response and the collapse mechanism of the bridge. When structures with poor detailing are studied or when other collapse modes are possible, degrading collapse modes, such as shear failures or shear-flexure failures, can be considered directly in the structural models. Including code-conforming confinement reinforcement in the plastic hinge region of bridge columns will prevent such undesirable failure modes. Where such collapse modes cannot be incorporated directly in the models, often they are considered indirectly by imposing some drift limits corresponding to such collapse modes (i.e., non-simulated component limit state criteria). That is, after performing IDA, the collapse capacities can be modified for other collapse modes such that the corresponding IM values to the non-simulated collapse modes (in terms of DM) will be considered as the collapse capacity of the structure. However, this simplification in modelling through application of the non-simulated component limit state criteria should be accounted for by increasing the modelling uncertainty in the overall seismic performance assessment of the structure. The model proposed by Elwood (2004) is an example of the models that can predict the drift corresponding to shear failure and axial failure of columns with insufficient transverse reinforcement.

The influence of the abutments on the seismic response of the bridges can be included in structural modelling. However, the effects of abutments may be conservatively neglected if they do not significantly influence the seismic response. An alternative approach may be to analyse the bridges for two cases of restrained and unrestrained movements at the abutments to evaluate the seismic response of the structure and to identify the controlling case. For example, in the transverse response shear keys are often used to restrain the transverse movements of the superstructure at the abutments. The shear keys may either be designed based on the capacity design concepts so that they will survive under high ground motion intensities or they can be designed as fuses to fail at low ground motion intensity levels to prevent damage in the abutments and piles. Therefore, depending on the design philosophy for such elements, different modelling approaches may be accepted. The most direct approach would be to include the degrading behaviour and failure of shear keys in the structural models. The shear key models by Megally *et al.* (2003) can provide the required information for this purpose. The spring abutment model developed by Aviram *et al.* (2008a) can be used when the effects of abutments are directly considered in

modelling. In the abutment model, the effects of all important abutment parameters, such as strength and stiffness due to back-fill soil, abutment wing wall and back wall, piles, bearing pads, shear keys and gaps, are considered in the response.

The effects of expansion joints can be considered using gap elements in the structural models. For example, for the case of the continuous bridges, the expansion joints are typically situated at the end abutments and can be considered by means of gap elements in the abutment models. If cap beams are present, then the cap beams and beam-column joints are designed as capacity protected elements (design to transmit the maximum expected resistance of the columns) and therefore are modelled as elastic elements.

The drift capacity at bar buckling damage state can be used to define the drift at which strength degradation begins (i.e., θ_{cap} in Fig. 21.2). A study by Berry and Eberhard (2007) provides some empirical equations to estimate the drift capacity of bridge columns at different damage states, including cover spalling and bar buckling, which can be used in structural modelling and prediction of the capacity of columns. Very few test results are available to calibrate the post-capping stiffness of bridge columns. Haselton *et al.* (2007) developed an equation to estimate θ_{pc} and recommended a conservative upper limit of 0.1 for the post-capping chord rotation of columns which is controlling for most columns designed using modern seismic codes. Although this equation was developed for columns with rectangular cross-sections with rectangular ties, it may also be conservatively used for columns with circular cross-sections containing code conforming spiral reinforcement. This drift ratio can be used to define the post-capping stiffness, K_c , as shown in Fig. 21.2. A summary of different methods to estimate the drift capacity of bridge columns at different damage states is available in Tehrani (2012) and Tehrani and Mitchell (2012a).

21.4 Sources of uncertainty

Many sources of uncertainty contribute to total variability in predicted responses using IDA (ATC-63, 2008):

- Record-to-record (RTR) uncertainty. This uncertainty is due to variability in the seismic response of structures to different ground motion records. The variability in response is mainly due to different frequency content and characteristics of the records, variability in the hazard characterization of the ground motions records, and different duration of records.
- Design requirements-related (DR) uncertainty. Such uncertainty is associated with the quality of the design requirements.

- Test data-related (TD) uncertainty. This type of uncertainty is related to the quality of the test data used to define the structural system.
- Modelling (MDL) uncertainty. This uncertainty is associated with the quality and robustness of the nonlinear structural models and the ability to accurately simulate the nonlinear behaviour of the structural elements subjected to seismic excitations.

The total uncertainty can be predicted by combining RTR, DR, TD and MDL uncertainties in terms of lognormal standard deviation of β_{RTR} , β_{DR} , β_{TD} , and β_{MDL} using Eq. [21.1].

$$\beta_{\text{TOT}} = \sqrt{\beta_{\text{RTR}}^2 + \beta_{\text{DR}}^2 + \beta_{\text{TD}}^2 + \beta_{\text{MDL}}^2} \quad [21.1]$$

Record-to-record variability, β_{RTR} , can be obtained from the IDA results. Where no detailed information is available, the other sources of uncertainty may be estimated based on quality ratings. For example, in the ATC-63 provisions (ATC-63, 2008) the quality ratings of ‘Superior’, ‘Good’, ‘Fair’ and ‘Poor’ are translated into quantitative values of uncertainty of 0.20, 0.30, 0.45 and 0.65, respectively. More details concerning the criteria to determine the quality ratings for different sources of uncertainty are available in ATC-63 (2008). Other studies have also used similar values for uncertainty. For example in a study by Mander *et al.* (2007), the uncertainties due to modelling and predicting the capacity of the bridge columns were considered as 0.25 and 0.2, respectively.

The reliability-based methods can be used for the detailed predictions of the effects of uncertainty on different aspects such as structural modelling. Such methods often include Monte Carlo simulation, the first-order-second-moment method (FOSM), the first-order reliability method (FORM) and the second-order reliability method (SORM). A summary of the methods to estimate the modelling uncertainty is available by Liel *et al.* (2009). More details regarding the incorporation of the uncertainty in probabilistic seismic performance assessment and seismic risk analysis are available by Cornell *et al.* (2002), Jalayer and Cornell (2003), Dhakal and Mander (2006) and Ellingwood and Kinali (2009).

21.5 Record selection for incremental dynamic analysis (IDA)

21.5.1 Spectral shapes of the records and epsilon values

The spectral shape of the records is especially important for collapse assessment which can change the calculated collapse capacity of the structures by 70% (Baker and Cornell, 2006a; Haselton and Baker 2006; Zareian, 2006). Therefore, neglecting the influence of the spectral shapes in collapse

assessment of the structures using IDA can result in very conservative and biased predictions.

Epsilon, ε , which is computed by subtracting the mean predicted $\mu_{\ln S_a}(M, R, T)$ from the record's $\ln S_a(T)$, and dividing by the logarithmic standard deviation (as estimated by the ground motion prediction equation). is a proxy of spectral shape (Baker and Cornell, 2006a). As the epsilon value increases, the spectral shapes tend to be more peaked. Specifically, ε at a given period T is given by Eq. [21.2]:

$$\varepsilon(T) = \frac{\ln S_a(T) - \mu_{\ln S_a}(M, R, T)}{\sigma_{\ln S_a}(T)} \quad [21.2]$$

where $\mu_{\ln S_a}(M, \bar{R}, T)$ and $\sigma_{\ln S_a}(T)$ are the predicted mean and standard deviation, respectively, of $\ln S_a$ at a given period T and $\ln S_a(T)$ is the natural logarithm of the spectral acceleration of interest. The two parameters are obtained from ground motion prediction equations (GMPE). Rare strong ground motions have peaked spectral shapes that can be represented by positive epsilon values at the fundamental period, $\varepsilon(T_1)$. This peaked spectral shape is very different from a typical shape of uniform hazard spectra and accounting for this has been shown to increase the computed collapse capacity significantly. The values of spectral acceleration at the longer periods are important, because the effective period of the structure increases as the structure undergoes inelastic deformation. This is even more important for modern structures that have high ductility capacities and are designed for high inelastic demands. Similarly, the values of spectral acceleration at shorter periods can be important especially for structures with significant contributions of higher modes in the seismic response.

21.5.2 Record selection strategies

Record selection for inelastic time history analysis is a critical issue and many different methods have been developed and used for this purpose. This subject is even more important for the case of IDA and collapse capacity evaluation of structures (e.g., Tehrani *et al.*, 2012).

A realistic record selection strategy should take into account important characteristics of the ground motion records which influence the structural response. Such important aspects of records mainly include M , R , ε , and site conditions. Owing to the limited number of available natural records, selecting ground motions with similar M , R and ε values to match target values obtained through seismic deaggregation is not straightforward. Usually one or two of these parameters are chosen as key parameters for record selection. Accordingly, different record selection methodologies may be utilized.

Uniform hazard spectrum (UHS)-based approach

The uniform hazard spectrum (UHS) is defined as a set of spectral acceleration amplitudes over a range of vibration periods with equal probability of exceedance in a period of time and is widely used in current seismic design codes. The simplest method for record selection is to choose the records such that their response spectra match a target UHS over a range of periods. However, the UHS is a composite of predicted responses at different periods that correspond to a specific hazard level, and may not be representative of individual ground motion spectra. The seed records used in this method can be either artificial, natural or manipulated natural records. Artificial records can be generated using some available software to match a UHS. Alternatively, natural records can be manipulated by changing the frequency content, so that the response spectra of the records match a target UHS. Scaled natural records that are matched to a target UHS over a range of periods are often recommended by some codes. This range must include the important modes of the structure as well as the effects of period elongation due to inelastic deformation of the structure. A period range of $0.2T_1$ to $2T_1$ (where T_1 is the fundamental vibration period of a structure) is usually recommended and used for this purpose. This range is similar to the $0.2T_1$ to $1.5T_1$ range specified by ASCE 7-05 (ASCE, 2005), but statistical studies suggest that nonlinear structures are often sensitive to response spectra at periods longer than $1.5T_1$ (Baker and Cornell, 2008; Haselton and Baker, 2006; Vamvatsikos and Cornell, 2005).

Although conceptually simple, the UHS-based methods are deemed to be conservative, particularly when ultimate performance limits of a structure (such as bar buckling, fracture, collapse, etc.) are the primary objective of the analysis. This is because the response spectra of the records are forced to match a target UHS, such as that specified in the National Building Code of Canada (NRCC, 2010). Furthermore, the UHS tends to be dominated at any individual periods by motions above the median, whereas individual spectra are unlikely to be equally above-median at all periods. That is, no natural record can be found to match the UHS over a wide period range and hence the analysis results using UHS-matched records may be conservative and biased. To solve these problems, the use of the CMS in lieu of the UHS has been recommended in recent years (Baker, 2011).

MR-based record selection

In this method, records with the closest M and R to the mean magnitude and mean distance obtained from the seismic deaggregation are chosen. A difference of one unit in magnitude may be treated as equivalent to a difference of 40 km in distance (Baker and Cornell, 2006a). For the records

selected based on magnitude and distance, it has been shown that inclusion of ε in a vector IM results in reduced estimates of structural response relative to the estimates obtained with the scalar IM, $S_a(T_1)$ (Baker and Cornell, 2006a). This indicates that response predictions obtained using the MR-based record selection approach are biased when $S_a(T_1)$ is used as IM.

Epsilon-based record selection

$\varepsilon(T_1)$ is an important property to match when selecting ground motions for analysis, where the ground motion intensity is measured using $S_a(T_1)$. The value of $\varepsilon(T_1)$ is in fact a proxy for spectral shape, and it has been demonstrated that selecting records with similar epsilon values to mean epsilon values obtained from the seismic hazard deaggregation can result in a more realistic prediction of seismic responses, especially at ultimate performance levels including collapse where the spectral shape effects are more prominent.

CMS-based record selection

The CMS-based record selection strategy is the only method that can take into account the influence of all three ground motion parameters (i.e., M , R and ε) in record selection. The idea is to predict the expected spectral shape of the records with magnitude, distance and ε values equal to the target values obtained from the seismic deaggregation. After predicting the target spectral shape, records with similar spectral shape over a range of period of interest (usually $0.2T_1$ to $2T_1$) will be selected, while no strict criteria on M , R and ε are imposed. In other words, the records with spectral shapes matching the CMS for a given M , R and ε will be accurate predictors of structural response, regardless of their actual M , R and ε values (Baker and Cornell, 2006a). This method will be discussed more in the next section.

A study by Baker and Cornell (2005) suggests that the records selected based on ε or CMS can also be scaled without resulting in biased predictions, unlike other methods such as UHS-based and M , R -based record selection. These observations can be explained by the idea that spectral shape (i.e., spectral acceleration values at other periods, given S_a at T_1) is the record property that directly affects seismic response of structures, whereas magnitude, distance and ε are proxies for spectral shape (and ε is a more important proxy) (Baker and Cornell, 2006a).

21.5.3 CMS

The CMS provides the expected response spectrum, conditioned on occurrence of a target spectral acceleration value at the period of interest. It has

been found that the CMS can be used as an appropriate target response spectrum for selecting ground motions as input for dynamic analysis (Baker, 2011). In the development of a CMS, some important aspect of records including M , R and ε will be considered from the deaggregation of seismic hazard. Baker (2011) proposed a method for calculating the CMS, given by Eq. [21.3]:

$$\mu_{\ln S_a(T_i) | \ln S_a(T_1)} = \mu_{\ln S_a}(\bar{M}, \bar{R}, T_i) + \rho(T_i, T_1) \bar{\varepsilon}(T_1) \sigma_{\ln S_a}(T_i) \quad [21.3]$$

where an appropriate GMPE must be used to evaluate the mean and standard deviation of the natural logarithm of the spectral acceleration at the vibration period T_i denoted by $\mu_{\ln S_a}(\bar{M}, \bar{R}, T_i)$ and $\sigma_{\ln S_a}(T_i)$, respectively, in which $\bar{M}$, $\bar{R}$, and $\bar{\varepsilon}(T_i)$ are the mean magnitude, mean distance and mean epsilon at the considered period T_i , respectively. These mean values are calculated from the seismic hazard deaggregation and $\rho(T_i, T_1)$ is the inter-period correlation of spectral accelerations at vibration periods T_i and T_1 . The equation proposed by Baker and Cornell (2006b), Eq. [21.4], is used to calculate the inter-period correlation.

$$\rho(T_i, T_1) = 1 - \cos \left[\frac{\pi}{2} - \left(0.359 + 0.163 I_{T_{\min} < 0.189} \ln \frac{T_{\min}}{0.189} \right) \ln \frac{T_{\max}}{T_{\min}} \right] \quad [21.4]$$

where $T_{\max}$ and $T_{\min}$ are the larger and the smaller of T_i and T_1 , respectively, and $I_{T_{\min} < 0.189}$ is an indicator function that equals 1 if $T_{\min}$ is less than 0.189 s and equals 0 otherwise.

For measuring the match with the target spectrum, the sum of squared errors (SSE) between the logarithms of the ground motion's spectrum and the target spectrum is used as recommended by Baker (2011).

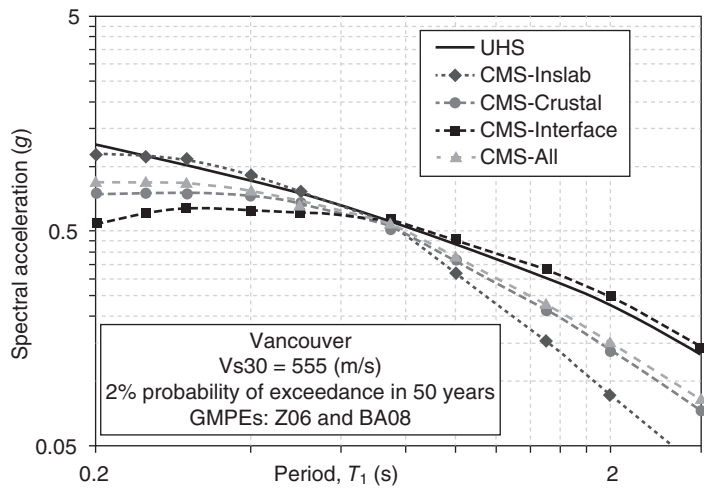
$$SSE = \sum_{j=1}^n [\ln S_a(T_j) - \ln S_{aCMS}(T_j)]^2 \quad [21.5]$$

where $\ln S_a(T_j)$ is the log spectral acceleration of the ground motion at period T_j and $\ln S_{aCMS}(T_j)$ is the log CMS value at period T_j . The values of SSE are computed for all records considered and the records with the smallest SSE values will be selected. This approach is more effective if ground motion scaling is allowed. To scale the ground motions, Baker (2011) recommends scaling each ground motion so that its spectral acceleration at the considered period, $S_a(T_1)$, matches the target spectral acceleration from the CMS, $S_{aCMS}(T_1)$. Although the concept of scaling the records is sometimes questioned, it has been observed that ground motions selected and scaled to match the CMS produce displacements that are comparable to those produced by unscaled ground motions, unlike scaling procedures using other methods (Baker and Cornell, 2005; Luco and Bazzurro, 2007). Thus

the scaling procedure outlined by Baker (2011) is not expected to adversely impact the resulting structural responses.

The procedure outlined by Baker (2011) is straight forward for sites where only crustal earthquakes occur (e.g., California). A complicated aspect in constructing a CMS for a site in south-western British Columbia, in comparison with a site in California, is that three earthquake types, having distinctly different characteristics, contribute to the overall seismic hazard. Therefore, three CMS must be constructed for record selection (Goda and Atkinson, 2011): ‘CMS-Crustal’, ‘CMS-Interface’ and ‘CMS-Inslab’. In order to account for these events, two sets of CMS can be considered as proposed by Goda and Atkinson (2011). The first approach is the CMS-Event-based procedure which is based on the weighted average of the CMS that are computed by using applicable GMPEs and the corresponding scenarios for the three earthquake types, in proportion to the relative influences of the scenarios (i.e., the number of records from each event type is proportional to the percentage of contribution of that event type in seismic hazard). The second approach is the ‘CMS-All-based’ which is the weighted average of ‘CMS-Crustal’, ‘CMS-Interface’ and ‘CMS-Inslab’ by considering relative influences of the individual earthquake types without considering different earthquake type records in the selection procedure. In fact ‘CMS-All-based’ is the simplified version of ‘CMS-Event-based’ in which different event types are considered in the construction of the CMS, but in selecting the records the relative influences of the scenarios are not considered. This results in variability of the seismic response based on the ‘CMS-All-based’ approach being smaller than that based on the ‘CMS-Event-based’ approach. However, it must be noted that for each earthquake type, the variability of the selected records in general tends to be underestimated for CMS-based or UHS-based approaches, since a close match to a target response spectrum is imposed (Baker, 2011). Nevertheless, the ‘CMS-Event-based’ approach can account for the variability of the CMS among different event types more reasonably than does the ‘CMS-All-based’ approach. It has been shown that the median IDA results computed using the ‘CMS-Event-based’ and ‘CMS-All-based’ are in good agreement (Tehrani *et al.*, 2012; Goda and Atkinson, 2011).

Figure 21.3 shows the CMS at 2% probability of exceedance in 50 years (i.e., return period of 2475 years), computed at $T = 0.7$ s for different event types separately using the deaggregation results presented in Table 21.1. The BA08 GMPE (Boore and Atkinson, 2008) and Z06 GMPE (Zhao *et al.*, 2006) were used for crustal events and subduction events, respectively. As expected, all CMS are approximately equal to the spectral acceleration values of the UHS at the target vibration period considered (i.e., $T = 0.7$ s). The CMS-Interface has rich spectral content in the long period range, while the CMS-Inslab has rich spectral content in the short period range. The



21.3 CMS using BA08 and Z06 GMPEs for $T = 0.7$ s (Tehrani *et al.*, 2012).

Table 21.1 Seismic deaggregation results for Vancouver at $T = 0.7$ s (epsilon values using the BA08 and Z06 GMPEs)

Probability of exceedance	Percentage of contribution: [Crustal, Interface, Inslab]	Crustal events	Interface events	Inslab events
		$[\bar{M}, \bar{R}, \bar{\epsilon}]$	$[\bar{M}, \bar{R}, \bar{\epsilon}]$	$[\bar{M}, \bar{R}, \bar{\epsilon}]$
10% in 50 yr	[48%, 17%, 35%]	[6.61, 37.2, 1.27]	[8.51, 142.4, 0.20]	[6.65, 76.1, 1.19]
2% in 50 yr	[40%, 27%, 33%]	[6.78, 18.3, 1.51]	[8.63, 142.8, 1.04]	[6.82, 64.6, 1.64]
0.5% in 50 yr	[38%, 30%, 32%]	[7.03, 10.3, 1.56]	[8.69, 141.4, 1.63]	[7.00, 55.9, 1.83]

Source: Tehrani: *et al.* (2012).

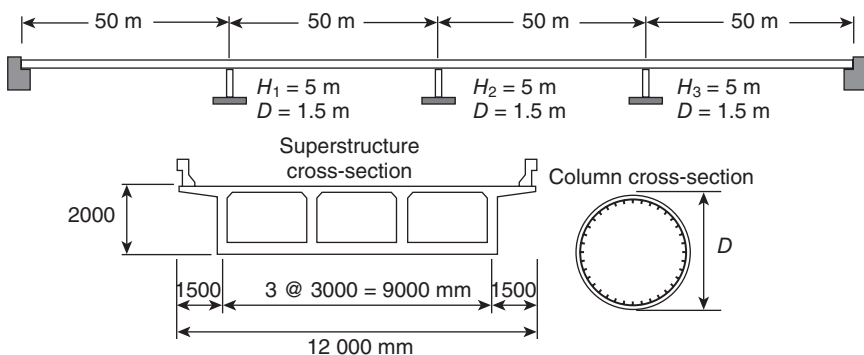
‘CMS-All’ is also computed which represents the weighted average of all three CMS according to the percentage of contribution of each event type. Figure 21.3 shows that the ‘CMS-All’ is very close to the ‘CMS-Crustal’ (Tehrani *et al.*, 2012).

21.5.4 Influence of record selection on IDA result

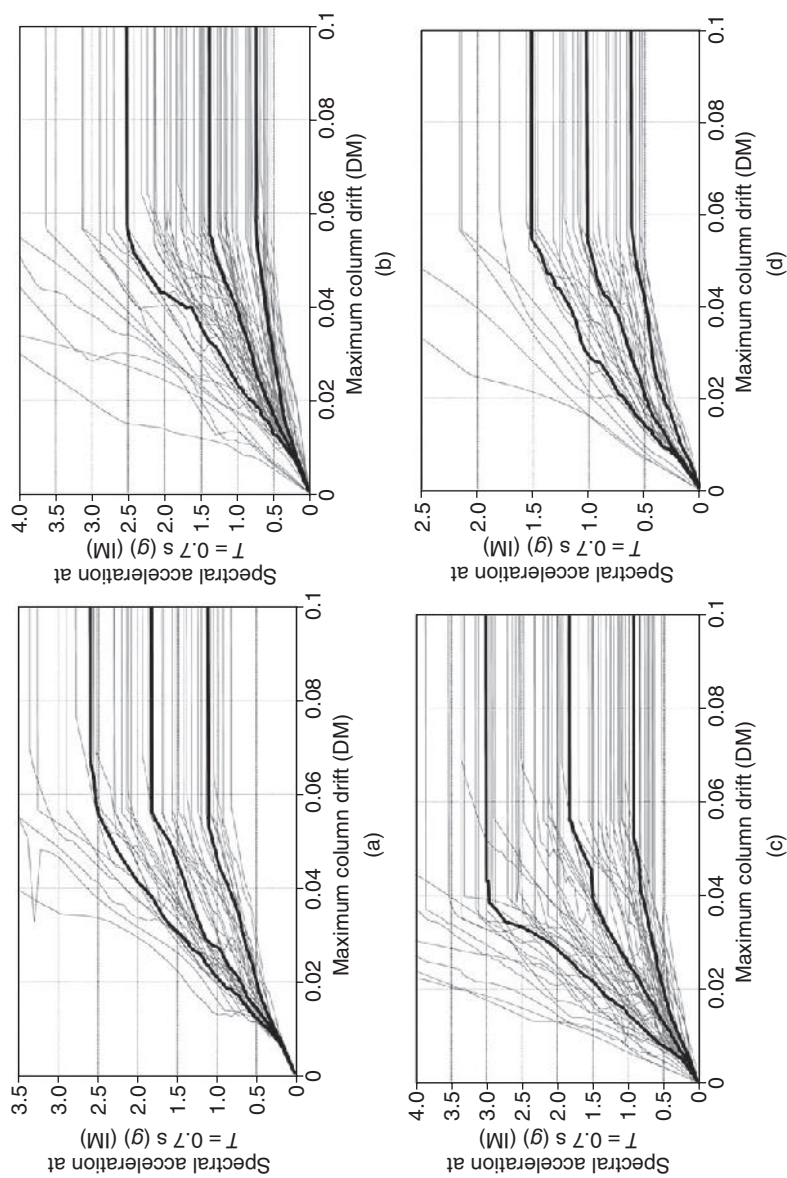
The influence of different record selection methodologies and the influence of including records from different earthquake types (i.e., crustal,

subduction interface and inslab earthquakes) on the IDA results were studied by Tehrani *et al.* (2012) for the case of a continuous 4-span bridge. The properties of the bridge studied are shown in Fig. 21.4. The bridge was designed using the 2010 CHBDC (CSA, 2010) for Vancouver, soil type C, and an importance factor of 1.5. The middle column had a longitudinal reinforcement ratio of 2%, while the other columns contained 1% of longitudinal reinforcement. The columns also had a spiral reinforcement ratio of 1.2% for confinement. For the case of the restrained transverse movements at the abutments the fundamental period of the bridge was 0.7 s. To investigate the effects of different record selection methods and earthquake types, only the transverse response of the bridge was studied. Ground motion records were selected from two extensive databases, the PEER-NGA database and the K-NET/KiK-NET database. Some limitations were imposed on the minimum magnitude (M), PGA and PGV of the records to consider strong ground motions available in the database. The characteristics of the records from the PEER-NGA database (179 records from 28 earthquakes) and the KNET/KIK-NET database (189 records from 23 earthquakes) are described by Goda and Atkinson (2009).

The IDA results including the IDA curves, 16%, 50% and 84% percentiles, and the standard deviation of the results (i.e., due to record-to-record variability) are presented in Fig. 21.5 and Table 21.2 for different record selection methods. The results are also summarized for different damage states including yielding, cover spalling and collapse. The use of the epsilon-based record selection resulted in highest median collapse capacity prediction. When different earthquake types are considered, it is necessary to increase the number of records to improve the predictions of the median



21.4 Bridge properties.



21.5 IDA results (at $T_1 = 0.7$ s) using: (a) epsilon-based method including only crustal events; (b) CMS-event-based method including all event types; (c) epsilon-based method including all event types; (d) UHS-based method including all event types (adapted from Tehrani *et al.* 2012).

Table 21.2 Summary of IDA results for different damage states (at $T_1 = 0.7$ s) using: (a) epsilon-based method including only crustal events; (b) CMS -event-based method including all event types; (c) epsilon-based method including all event types; (d) UHS-based method including all event types

(a) Epsilon-based (only crustal events)	(b) CMS-event- based (all event types)				
	IDA percentiles (g) ($S_a @ T_1 = 0.7$)			IDA percentiles (g) ($S_a @ T_1$)	
Damage states	Median	16%	84%	St Dev	Damage states
Yielding	0.14	0.13	0.14	0.09	Yielding
Cover spalling (exp.)	0.66	0.47	0.81	0.28	Cover spalling (exp.)
Collapse	1.84	1.12	2.62	0.42	Collapse
(c) Epsilon-based (all event types)	(d) UHS-based (all event types)				
	IDA percentiles (g) ($S_a @ T_1$)			IDA percentiles (g) ($S_a @ T_1$)	
Damage states	Median	16%	84%	St Dev	Damage states
Yielding	0.18	0.14	0.23	0.43	Yielding
Cover spalling (exp.)	0.71	0.46	1.25	0.52	Cover spalling (exp.)
Collapse	1.82	0.93	3.09	0.62	Collapse

Source: Adapted from Tehrani *et al.* (2012).

collapse capacity. This is because including three different earthquake types increases the record-to-record variability of the results. It is interesting that the median collapse capacity predictions considering three different earthquake types were similar to those predicted using crustal records only, as shown in Fig. 21.5a and 21.5c and presented in Table 21.2a and 21.2c. In a study by Tehrani and Mitchell (2012b) the median collapse capacities of different bridge configurations with different period ranges were evaluated using different earthquake types in IDA. The results indicated that the median collapse capacities obtained using crustal events only were typically close to those obtained considering three different earthquake types in IDA, whereas the record-to-record variability was much larger when three different earthquake types were considered. This may suggest that seismic evaluations using IDA for Vancouver could be carried out using crustal events only and to include the effects of different earthquake types the record-to-record variability could be increased. The record to record variability in such cases could be approximately predicted using the elastic response spectra of all records from different event types. More research is needed on this subject before a conclusion can be possibly made. The use of the CMS-based record selection resulted in somewhat lower predictions of the collapse capacity as shown in Fig. 21.5b compared to the predictions using the epsilon-based method (Fig. 21.5c). However, the CMS-based method was less sensitive to the number of records considered in the IDA (Tehrani *et al.*, 2012). The IDA results obtained using the UHS-based record selection are demonstrated in Fig. 21.5d and Table 21.2d. A comparison of the IDA results obtained using the UHS-based method with those obtained using the epsilon-based method resulted in an 80% difference in the median collapse capacity. This demonstrates the importance of the record selection on the IDA results. Therefore, neglecting the spectral shapes and epsilon values in the record selection can result in highly conservative predictions of the median collapse capacity of the structure.

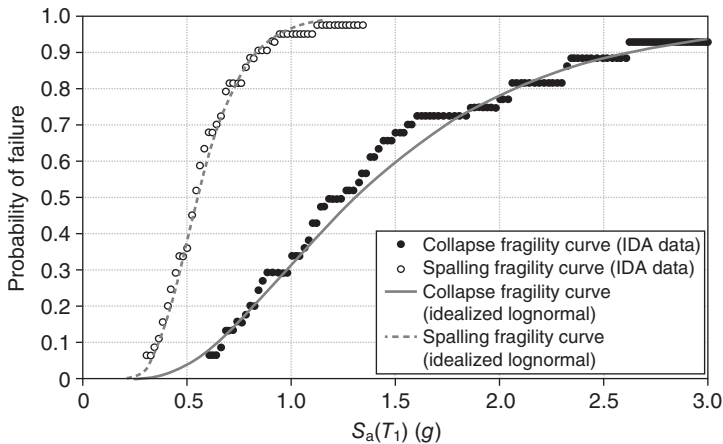
The direct use of the epsilon-based method to account for the spectral shape of the ground motion records is often time-consuming and may be complicated in practice. A simplified method has been developed by Haselton *et al.* (2011) to modify the collapse capacity predictions for the effects of epsilon and spectral shapes in IDA predictions. For this purpose, a fixed set of records can be used to perform IDA. The predicted capacities obtained using IDA are then modified for the effects of spectral shapes using a spectral shape factor (SSF). To compute the SSF, a regression analysis needs to be carried out to derive a relationship between the natural logarithm of the collapse capacities versus $\varepsilon(T_1)$ values for each record. The results from the simplified method are in good agreement with those obtained using the epsilon-based record selection method.

21.6 Development of fragility curves using incremental dynamic analysis (IDA) results

The fragility curve gives the conditional probability that a certain limit-state be exceeded (i.e., probability of failure) at a given IM value. Percentile IDA curves can be used to derive fragility curves. In Fig. 21.6 the probabilities of failure at different IM levels are derived from the IDA results shown in Fig. 21.1. The fragility curves were developed for collapse and cover-spalling damage states. The cover-spalling state was defined as exceeding the drift ratio of 1.85% determined using the equations by Berry and Eberhard (2007). The fragility curves developed using IDA data are shown by dots, in which each dot represents a ratio of the number of records that caused failure to the total number of records at an IM level. In Fig. 21.6, the cumulative lognormal distribution curves developed using the median and standard deviation of the IDA results at collapse and cover-spalling limit states are also shown. The fragility curves using IDA data can be estimated using such curves.

Assuming that the data are lognormally distributed, it is possible to develop the fragility curves at collapse (or any other limit-state) by computing only the median collapse capacity and logarithmic standard deviation of the IDA results at collapse. The fragility curves then can be analytically computed using Eq. [21.6]:

$$P(\text{failure}|S_a = x) = \Phi \left[\frac{\ln(x) - \ln(S_{a50\%}^C)}{\beta_{\text{RTR}}} \right] \quad [21.6]$$



21.6 Development of fragility curves for different limit states using IDA results.

where $\Phi(\cdot)$ is the cumulative normal distribution function, $S_{a50\%}^C$ is the median capacity determined from IDA, and β_{RTR} is the record to record variability in the IDA results.

In the IDA results, only the uncertainty due to record-to-record variability is considered explicitly. Therefore it is necessary to modify the fragility curves developed using the IDA results to include the uncertainty due to other sources of uncertainty as discussed before (i.e. β_{DR} , β_{TD} and β_{MDL}). The modified fragility curve then can be computed using the following equation:

$$P(\text{failure}|S_a = x) = \Phi \left[\frac{\ln(x) - \ln(S_{a50\%}^C)}{\beta_{TOT}} \right] \quad [21.7]$$

where β_{TOT} is the total uncertainty as defined by Eq. [21.1]. While the median collapse intensity is unchanged, additional uncertainty causes a large increase in the probability of collapse at the lower IM levels such as that corresponding to the maximum considered earthquake (MCE).

21.7 Case study for a continuous 4-span bridge

In this section, an application of IDA for the probabilistic seismic performance and risk assessment will be provided in more detail for a 4-span continuous reinforced concrete bridge located in Montreal.

21.7.1 Bridge properties and modelling assumptions

The structure studied includes a regular 4-span bridge supported by single columns similar to that shown in Fig. 21.4, except that the column heights and diameters were 7 m and 2 m, respectively. The span length is 50 m and the superstructure consists of a box girder with a uniform dead load of 200 kN/m. The bridge is assumed to be unrestrained against transverse movement at the abutments (i.e., it is assumed that the bridge is either designed for free transverse movements at the ends or the bridge shear keys at the abutments are expected to fail at high seismic intensity levels). The bridge was designed based on the 2010 CHBDC provisions (CSA, 2010) except that the design spectrum given in the 2010 NBCC (NRCC, 2010) was adopted. The design spectra in the 2010 NBCC correspond to 2% probability of exceedance in 50 years. The bridges were designed for Montreal and soil class C (i.e., $V_{S30} = 555$ m/s) with an importance factor I of 1.0. The concrete compressive strength and the yield stress of the reinforcing bars were taken as 40 and 400 MPa, respectively. The minimum reinforcement ratio was controlling in all cases, when either a force modification factor of $R = 3$ or $R = 5$ was used in design, and therefore all bridge columns in this

study contain the minimum longitudinal reinforcement ratio of 0.8% based on the 2010 CHBDC. For the transverse reinforcement a confinement reinforcement ratio of 1.2% was required.

The first mode of the bridge is a torsional mode with $T = 1.43$ s (with effective modal mass ratio close to zero). The fundamental period in the transverse direction is 1.30 s (with 60% contribution) and the period in the longitudinal direction is 0.92 s. In the transverse direction a third mode with $T = 0.75$ s (with about 40% contribution) is also important. For the three-dimensional analyses the geometric mean period of 1.09 s was used to compute the spectral acceleration of the records. It should be noted that after performing the IDA, using a fixed record set, the IM values can be computed at any desired period using the elastic response spectra of the records and therefore there is no need to perform the analyses again for different periods. In this study only the results at the fundamental period of the structure (which was found the most critical) are presented. Similarly the evaluations can be carried out at the other periods. However, it should be noted that for the record selection methods which are dependent on the period of the structure, the evaluations need to be carried out using different record sets at different periods and therefore the IDA should be performed for each period independently. The periods are computed using the effective stiffness of the columns (i.e., $K_e = M_y/\theta_y$ where M_y and θ_y are the moment and rotation at yield, respectively).

The modified Takeda hysteresis model (Otani, 1981) was used in this study to model the behavior of the reinforced concrete (RC) columns using Ruaumoko software (Carr, 2009). However, the hysteresis model developed by Ibarra *et al.* (2005) is recommended to be used when this model becomes available in the computer program. The modified Takeda model has two main parameters, alpha and beta, which control the unloading and the reloading stiffness, respectively. The values of alpha = 0.3 and beta = 0.3 were adopted to comply with both the recommended values in practice (Priestley *et al.*, 2007) and the median values from the tests (more details are available in Tehrani *et al.*, 2012). The structural modelling considered in this study is similar to that used by Priestley *et al.* (2007), except that the backbone curve shown in Fig. 21.2, including the post-peak response was used. More details concerning the modelling of the bridge are available in Tehrani *et al.* (2012) and Tehrani and Mitchell (2012a).

For the 3D analyses the yield interaction surface developed by Tseng and Penzien (1973) was used in the Ruaumoko program (Carr, 2009). The interaction factors for the flexural term (i.e., a and b factors in the original model by Tseng and Penzien, 1973) were considered as 2 for the circular bridge columns. The use of the beam elements in modelling (i.e., interaction of moments only) also resulted in similar predictions with about 15% differences on average. The small differences are due to the fact that the effects

of axial forces were not significant in the bridges considered in this study. The effects of axial forces on the seismic response are expected to be more important for the case of bridges with multi-column bents or bridges with shorter span lengths. In the 3D analyses as the moment strength or yield strength of column degrades, the stiffness also degrades accordingly such that the yield displacements remain constant and the definition of ductility remains consistent (Carr, 2009).

21.7.2 Damage states and developing the backbone curves

Several damage states were considered in the seismic evaluation of the bridge under study including yielding, serviceability, bar-buckling and collapse. The yielding and serviceability damage states for the bridge columns were predicted using the method described by Priestley *et al.* (2007) that involved the development and idealization of the moment-curvature diagram. The serviceability limit state is usually exceeded when the concrete cover outside the confined core starts spalling (i.e., the maximum compression strain in the unconfined concrete cover exceed 0.003 to 0.004) or crack widths are larger than allowable limits (i.e., the maximum tensile strain is larger than 0.015 for column elements; Priestley *et al.*, 2007). However, it should be noted that the median compressive strain corresponding to spalling based on available tests is reported as 0.008 with a coefficient of variation of 0.45 by Berry and Eberhard (2007).

The ultimate strain in the steel bars corresponding to bar buckling from the experimental equations by Berry and Eberhard (2007) was used to compute the ultimate curvature of the columns and the corresponding drift and ductility is defined as the point at which strength degradation begins (i.e., θ_{cap} in Fig. 21.2). The drift ratio at the bar-buckling damage state is similar to that obtained for the 'Life Safety' limit state based on the recommendations by Priestley *et al.* (2007) (more details available in Tehrani and Mitchell, 2012a). The conservative limit of 0.1 on post-capping capacity (Haselton *et al.*, 2007), θ_{pc} , was controlling for the bridge columns and was adopted in this study due to lack of research. This limit is deemed to be conservative for the bridge columns with spiral transverse reinforcement.

The bridge under study was designed and detailed to meet the code requirements for ductile response, including capacity design concepts and adequate support lengths at the abutments. The ductile columns contain code-compliant spiral reinforcement to confine the concrete, to avoid shear failure and to control buckling of the vertical reinforcing bars. For this continuous bridge, with all other failure modes avoided, the flexural response governs the response of the bridge (i.e., sidesway collapse mechanism). The collapse prediction is based on dynamic instability of the structure (Vamvatsikos and Cornell, 2002). While the columns will undergo a ductile

Table 21.3 Deformations at damage states for the bridge columns with $D = 2.0$ m and $H = 7$ m

Damage state	Drift	Curvature ductility	Displacement ductility
Yielding	0.54%	1	1
Serviceability	1.05%	4.1	2.0
Bar buckling	4.33%	23.1	8.1

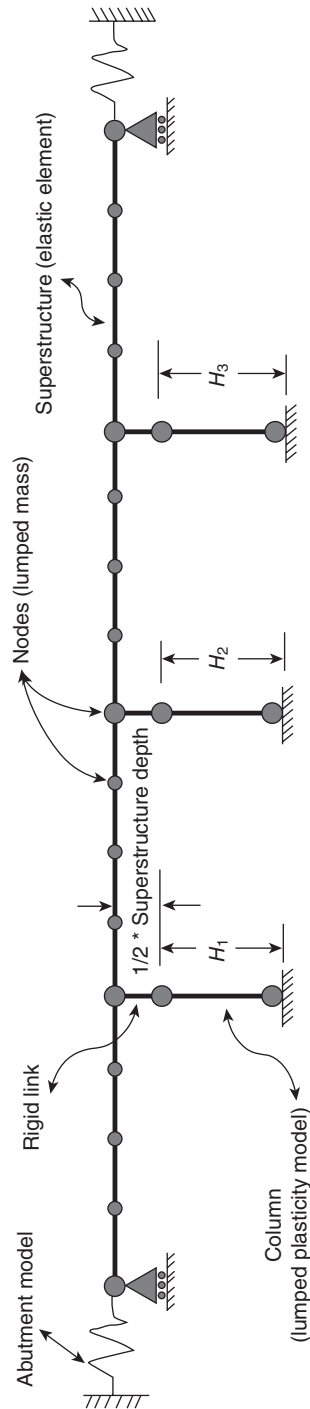
inelastic response during a major event, the bridge superstructure is designed to remain elastic and is typically modelled using elastic beam elements. Rigid elements were defined to model the superstructure depth, as shown in Fig. 21.7. The heights of these elements were considered as half of the superstructure depth. The drift and ductility capacities at different damage states are presented in Table 21.3 for the bridge columns studied. Therefore to define the backbone curve shown in Fig. 21.2, the drift ratios at yielding, beginning of strength degradation and collapse (i.e., zero strength) were taken as 0.56%, 4.33% and 14.33%, respectively.

21.7.3 Abutment modelling

The simplified abutment model developed by Aviram *et al.* (2008a) was used to model the influence of the abutments on the seismic response of the bridges in the longitudinal direction. The longitudinal response in the simplified model is a function of the system response including the gap, the abutment backwall and the soil backfill material. In the simplified model the effects of the bearing pads on the responses are ignored. After gap closure, the superstructure bears directly on the abutment backwall and mobilizes the full passive backfill pressure. Two rigid elements were connected to the abutment ends to model the superstructure width as recommended by Aviram *et al.* (2008a). More details of modelling the abutments and the influence of the abutments on the longitudinal response are available in Tehrani and Mitchell (2012c).

21.7.4 Record selection criteria and spectral shape effects

For crustal earthquakes, the ‘basic far-field’ records selected by Haselton and Deierlein (2007) were used in this study. This fixed record set includes 39 crustal records from the PEER-NGA database. Some minimum limits on event magnitude, PGA and PGV were imposed in the record selection to make sure that the selected records are strong enough to cause structural



21.7 Structural modelling of the bridge.

collapse (when scaled). It should be noted that IDA can be performed using fewer records (e.g., 22 records based on ATC-63).

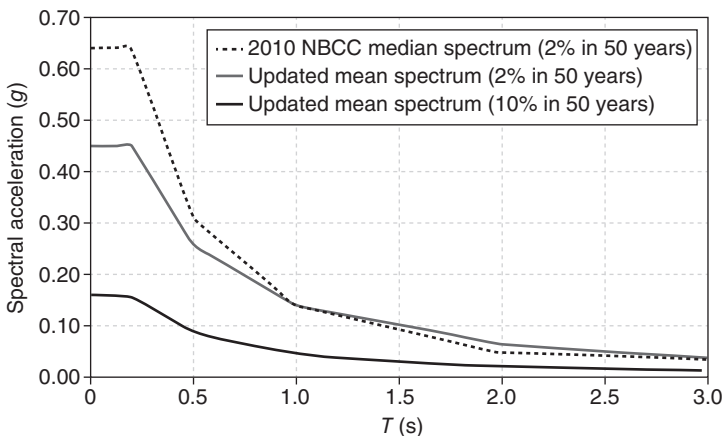
The prediction of the epsilon values of the records using the AB06 GMPE (Atkinson and Boore, 2006) for eastern North America and the deaggregation of the seismic hazard for Montreal indicated that the mean epsilon values of the records considered (in the range of periods of the bridge studied) are similar to the mean epsilon values obtained from the seismic hazard deaggregation. Therefore no modification of the results for the spectral shape effects was necessary.

21.7.5 Seismic hazard in Montreal

An updated seismic hazard model developed by Atkinson and Goda (2011) was used. The uniform hazard spectra at different hazard levels (i.e., 10% and 2% probability of exceedance in 50 years) obtained using the updated model along with the 2010 NBCC design spectra are shown in Fig. 21.8 for Montreal.

21.7.6 Seismic performance and risk assessment using IDA

To develop the IDA curves the spectral acceleration at the fundamental period of the bridges is used as the IM and the maximum drift ratio of the columns is used as the DM. The preferred choice of an IM in the case where the two axes of the structure have different fundamental periods has not

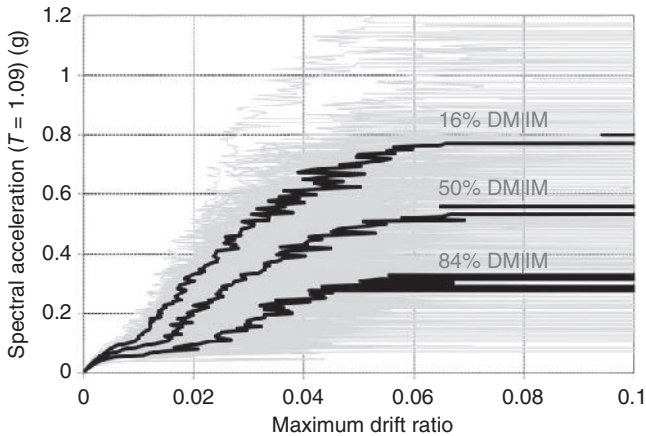


21.8 Uniform hazard spectra (UHS) at different hazard levels for Montreal and site class C.

been extensively examined. In the absence of further research, the geometric mean of the pair of ground motions, $S_{a_{g.m.}}$, at the geometric mean of the two periods is used as the IM parameter as recommended by Baker and Cornell (2006c).

The 39 pairs of records were applied twice to the model, once with the ground motion records oriented along one principal direction, and then again with the records rotated 90 degrees. This resulted in 78 IDA curves. The use of pairs of ground motions in IDA using 3D models for the multi-degree of freedom bridges with single column bents studied resulted in 'waving' behaviour with some points of structural resurrection (i.e., the structure resists seismic forces at higher IM values after a collapse at a lower IM value; see Vamvatsikos and Cornell, 2002, for the more details). This is because the timing, details and frequency contents of the ground motions have a significant effect on the seismic response and when the records are applied in pairs the complexity of the seismic response increases even more. Therefore the IDA analyses were performed using very small increments of IM values to determine the maximum seismic demands at each intensity level. The increments were considered as 5% of the spectral acceleration at the MCE level (e.g., 0.0065 (g) at $T = 1.09$ s). The IDA analyses were continued up to twice the intensity level at which the first global dynamic instability (collapse) was observed, due to possible structural resurrection.

The percentiles of the IDA results were determined using the IM|DM and DM|IM approaches. For the IM|DM method the lowest spectral acceleration at which the global dynamic instability occurs is determined as the collapse capacity for each record and subsequently the percentiles are determined. For complex IDA curves (i.e., due to waving and resurrection behaviour) the use of the IM|DM percentiles can be conservative. The DM|IM percentiles are determined based on the percentiles of the drift ratios at each IM level. The DM|IM percentiles for such cases can provide more reasonable predictions of the IDA percentiles. It should be noted that where the IDA curves do not exhibit waving and structural resurrection behaviours the line connecting the $x\%$ percentiles of DM|IM is typically the same as the one connecting the $(100 - x)\%$ percentiles of IM|DM (Vamvatsikos and Cornell, 2004). The DM|IM percentile curves are shown in Fig. 21.9. In Table 21.4 more details of the IDA results determined for different damage states are presented. Some researchers have expressed their concerns about the scaling of the pairs of ground motion and the choice of a scale factor for the orthogonal component of a ground motion (e.g., Baker and Cornell, 2006c). In the ATC-63 provisions (ATC-63, 2008) concern is expressed about the results from the 3D analysis being conservative. Because ground motions records are applied in pairs in three-dimensional nonlinear dynamic analyses, the resulting behaviour from each ground motion component is coupled. On the other hand the use of 2D



21.9 IDA results developed for the case of a regular bridge with $H_1 = H_2 = H_3 = 7$ m and $D = 2$ m (adapted from Tehrani and Mitchell, 2012d).

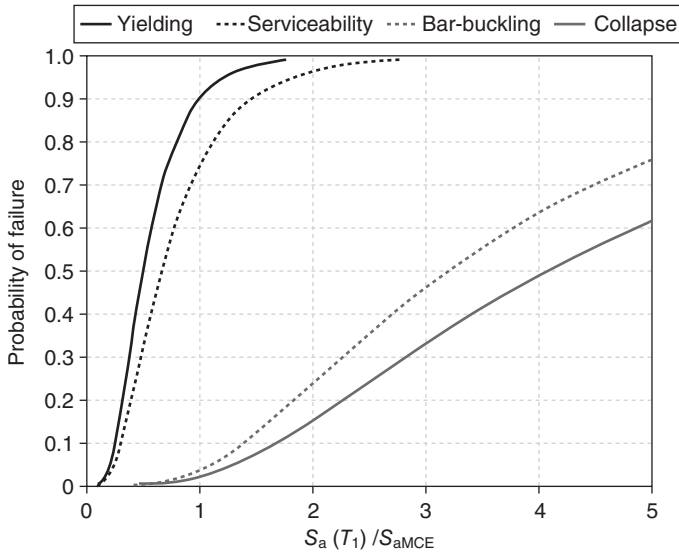
models may overestimate the overall capacity of the structure. More research is required to investigate the differences between the 2D and the 3D analyses for performing the IDA especially for the case of bridges. Fragility curves can be developed using the IDA results as discussed in Section 21.6. The record-to-record variability can be obtained from the IDA results at different damage states as reported in Table 21.4. Although there is little research on this subject, the quality of test data and modelling were assumed as good and the quality of the design requirements was considered to be superior in this study to estimate the total uncertainty, using the judgemental method described in Section 21.4.

The IDA results presented in Table 21.4 can be used along with Eq. [21.7] to develop the fragility curves at different damage states, as shown in Fig. 21.10. In the horizontal axis of the fragility curves in Fig. 21.10 the spectral acceleration at the fundamental period, $S_a(T_1)$, is normalized by the spectral acceleration at the MCE level, S_{aMCE} . The fragility curves can be better compared for different structures with different periods, when $S_a(T_1)$ is normalized. For example when $S_a(T_1)$ is twice the S_{aMCE} , the probability of collapse is around 15%. To predict the probability of collapse or the probability of exceeding different damage states at different hazard levels, Eq. [21.7] is used with $S_a(x)$ set as the value of the UHS at the corresponding hazard level (i.e., UHS for 2% or 10% probability of exceedance in 50 years given in Fig. 21.8). The probability of collapse as well as exceeding different damage states at the MCE level (i.e., 2% in 50 years), given in Table 21.4, is reasonably low and the performance of the bridge is satisfactory.

Table 21.4 IDA results and estimated probability of exceeding different damage states for the regular bridge

Damage state	IM/IDM percentiles (g)				DMI/IM percentiles (g)				Probability of failure at different hazard levels		
	50%	16%	84%	St Dev	50%	84%	16%	St Dev	β_{TOT}	10% in 50 years	2% in 50 years
Yielding	0.065	0.050	0.083	0.24	0.065	0.050	0.085	0.26	0.54	18.99%	90.19%
Serviceability	0.080	0.058	0.099	0.27	0.087	0.057	0.126	0.39	0.61	10.65%	74.47%
Bar-buckling	0.188	0.121	0.347	0.50	0.417	0.250	0.618	0.45	0.65	0.02%	3.75%
Collapse	0.347	0.201	0.513	0.50	0.534	0.277	0.772	0.51	0.70	0.01%	2.14%

Source: Adapted from Tehrani and Mitchell (2012d).



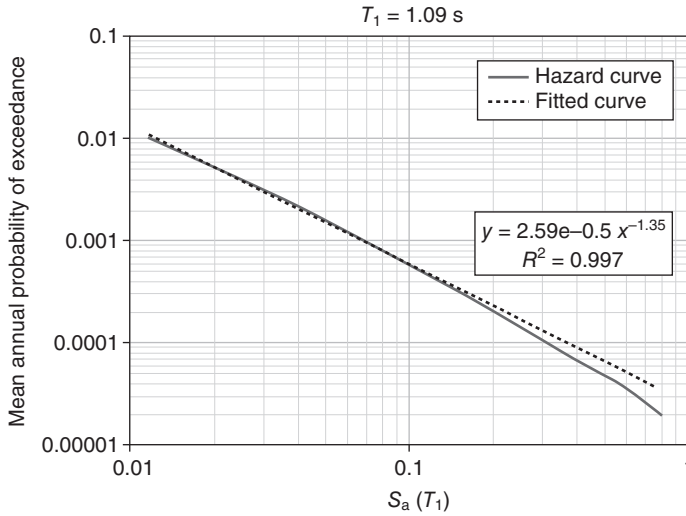
21.10 Fragility curves for different damage states (Adapted from Tehrani and Mitchell 2012d).

The probability of collapse at the MCE level is typically limited to 10% (and to 20% for rare cases) (ATC-63, 2008). These limits are currently based on judgement and accordingly different limits on the maximum probability of collapse may be accepted for different types of structures. For bridges different criteria might be accepted for seismic evaluations according to the functionality and importance of the bridge.

The computed probability of exceeding different damage states reported in Table 21.4 and Fig. 21.10 are conditional probabilities predicted at a given intensity level. To predict the seismic risk associated with exceeding different damage states (i.e., the total probability of failure in a period of time), the results obtained from the fragility curves should be combined with the results from the seismic hazard analysis (i.e., hazard curves) using the ‘total probability theorem’ (Benjamin and Cornell, 1970). The hazard curves provide the mean annual frequency that a certain level of seismic intensity is exceeded. The hazard curve obtained at $T = 1.09$ s for Montreal using the updated seismic hazard data provided by Atkinson and Goda (2011) is shown in Fig. 21.11.

Following the determination of the hazard curve, the mean annual probability of exceeding any particular damage state, λ_{DS} , can be estimated using Eq. [21.8] (e.g., Jalayer and Cornell 2003):

$$\lambda_{DS} = \int_{IM=0}^{IM=+\infty} f_R(IM) H(IM) dIM \quad [21.8]$$



21.11 Seismic hazard curve for Montreal, site class C and $T = 1.09$ s.

where $f_R(IM)$ is the probability density function of capacity in terms of IM and $H(IM)$ is the conventional hazard curve. This can be numerically computed by Eq. [21.9] using the fragility and hazard curves. F_R is the fragility function developed for a damage state (e.g., cover-spalling or collapse) and spectral acceleration, S_a , is used as intensity measure, IM.

$$\lambda_{DS} = \sum_{\text{All } S_{a_i}} [F_R(S_{a_i}) - F_R(S_{a_{i-1}})] H(S_{a_i}) \quad [21.9]$$

On the other hand, if some reasonable approximations are made, Eq. [21.8] can be analytically integrated assuming that the IM values of capacity are lognormally distributed and that the IM hazard curve can be approximated by fitting a straight line in the log-log space, $H(IM) = k_0 IM^{-k}$, (Shome and Cornell, 1999; Cornell *et al.* 2002). Based on these assumptions the integration in Eq. [21.8] will result in Eq. [21.10]:

$$\lambda_{DS} = \lambda(S_{a50\%}^C) \cdot \exp\left[\frac{1}{2}(k\beta_{TOT})^2\right] \quad [21.10]$$

Therefore, the mean annual rate of exceeding a damage state, λ_{DS} , can be estimated using Eq. [21.10] given the median capacity of the structure at the damage state, $S_{a50\%}^C$ and the total uncertainty, β_{TOT} , which are determined through IDA. The values of k_0 and k in Fig. 21.11 are estimated as $2.59e-5$ and 1.35 , respectively, at $T_1 = 1.09$ s. The probability of exceeding a damage state in the next T years can then be estimated using the Poisson distribution, given in Eq. [21.11].

Table 21.5 Mean annual rate of exceeding different damage states

Damage state	Mean annual rate of exceeding the damage state		Probability of exceeding the damage state in 50 years	
	Using Eq. [21.9]	Using Eq. [21.10]	Using Eq. [21.9] and Eq. [21.11]	Using Eq. [21.10] and Eq. [21.11]
Yielding	1.40E-03	1.42E-03	6.77%	6.84%
Serviceability	1.01E-03	1.01E-03	4.93%	4.93%
Bar-buckling	9.79E-05	9.28E-05	0.49%	0.46%
Collapse	6.84E-05	6.96E-05	0.34%	0.35%

$$P(\text{exceeding the damage state in } T \text{ years}) = 1 - \exp(-\lambda_{DS} \cdot T) \quad [21.11]$$

The mean annual rate of exceedance, along with the total probability of failure in 50 years, are presented for different damage states in Table 21.5 using the integration of the fragility and hazard curves (Eq. [21.8]) and also using the simplified equation (Eq. [21.10]). The predictions using the two equations are very close and therefore Eq. [21.10] can be used with sufficient accuracy to estimate the mean annual rate of failure.

21.8 Conclusions and future trends

The application of the IDA method for assessing the seismic performance and risk of bridges was presented. The IDA method is analytically intensive, requiring many nonlinear analyses and currently is not widely used in practice. However, it is possible to develop IDA algorithms that can significantly reduce the computation time enabling the derivation of the key information required to assess the seismic performance and risk. With the availability of powerful computers, it is possible to perform a large volume of analyses in a short amount of time. This makes the applicability of IDA more promising. The application of the IDA method enables a more thorough and systematic assessment of the seismic performance of structures, taking into account different sources of variability and assessing the performance for a wide range of possible ground motion records with different frequency contents and for different levels of seismic intensity. This makes the results from the IDA method an invaluable tool for seismic risk assessments. It is possible to enhance the IDA method by improving the scaling procedures (e.g., by means of incorporating natural or simulated records corresponding to different hazard levels rather than scaling a fixed record) and by incorporating the important aspects of spectral shape, higher mode effects and

period elongation. On the other hand, advanced structural modelling techniques enable the modelling of stiffness and strength degradations in different structural elements up to collapse including combinations of possible failure modes. Once such advanced structural models are used in IDA, a more precise prediction can be achieved by reducing the sources of uncertainty due to modelling and incorporating different collapse modes directly in the simulations. More experimental studies are required for different structural elements, exciting them to collapse, to improve the structural modelling.

21.9 Sources of further information

The development of the IDA method and more details regarding the concepts were described by Vamvatsikos and Cornell (2002, 2004, 2005, 2006). More information regarding predictions of drift capacity, ductility capacity and damage states for RC columns that is useful in structural modelling and prediction of limit states was given by Berry and Eberhard (2007); Berry *et al.* (2004), Dutta and Mander (1998); Priestley *et al.* (1996, 2007); Panagiotakos and Fardis (2001); Fardis and Biskinis (2003); and Haselton *et al.* (2007).

Information on the financial risk assessment of bridges was provided by Mander *et al.* (2007) and Dhakal and Mander (2006). Information regarding the general aspects of modelling bridge structures was given by Priestley *et al.* (1996, 2007); Berry and Eberhard (2007); and Aviram *et al.* (2008a,b).

Information concerning the evaluation of the modelling uncertainty is available from Liel *et al.* (2009). More information regarding the probabilistic basis for seismic performance evaluation is available from Cornell *et al.* (2002) and Jalayer and Cornell (2003).

Ground motion data for Japanese subduction earthquakes can be obtained from the K-NET and KiK-NET databases at www.k-net.bosai.go.jp and www.kik.bosai.go.jp; other crustal ground motion data can be obtained from the PEER-NGA database available at <http://peer.berkeley.edu>.

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Effect of soil–structure interaction and spatial variability of ground motion on seismic risk assessment of bridges

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Abstract: This chapter discusses the recent advancements in the field of soil–structure interaction and spatial variation of earthquake ground motion in terms of the related analysis aspects and the corresponding implications in seismic assessment and design of existing and new bridges. Various modeling techniques are described with respect to the efficient decoupling of the kinematic and the inertial soil–pile interaction while various issues related to ground motion selection and generation are critically assessed (i.e., site response, varying angle of wave incidence, multiple support and rotational excitation). The chapter concludes with recommendations for efficient finite element modeling for both research and design purposes.

Key words: soil–structure interaction, spatial variability, bridge engineering.

22.1 Introduction

Although reinforced concrete (RC) bridge structures might seem simpler structural systems than ordinary buildings, their response under earthquake loading is more complicated (Kappos, 2009): bridges have an order of magnitude larger cross-section dimensions; they cross non-uniform soil profiles; their connections are not necessarily monolithic (i.e., bearing type deck–pier connections); their boundary conditions introduce issues of geometrical non-linearity (due to the presence of stoppers, gaps, shear keys, etc.); and contribution of higher modes of vibration is often significant (Sextos and Kappos, 2009). Thus, the above structural characteristics and variability of *in situ* soil condition make bridges more sensitive to the assumptions made regarding the supporting soil and its dynamic interaction with the superstructure through the foundation of their piers and abutments.

Extensive research has been conducted over the past 30 years to investigate the impact of the soil–structure interaction (SSI) on the dynamic and seismic response in a wide range of bridge engineering applications (Kwon and Elnashai, 2008; Mylonakis and Gazetas, 2000; Mylonakis, *et al.*, 2006a; Pecker, 2007). Nevertheless, common practices and code-based approaches

on these issues still remain approximate and require improvements, as the phenomenon is often treated as either beneficial or is completely neglected (Mylonakis and Gazetas, 2000). One reason for this oversimplification is based on the anticipated period elongation of the structure with respect to the fundamental period of vibration of a fully fixed system and the corresponding monotonic decrease of spectral accelerations foreseen, at least on the basis of the design spectrum. Another reason is the energy dissipation that is known to take place at the foundation level due to the radiation of seismic waves and hysteretic damping, both contributing to the reduction of seismic loading when the presence of soil is considered (Gazetas, 2006; Makris *et al.*, 1994). The above arguments support the common perception that essentially any structure (bridges included) can be conservatively considered to be fixed at its base during analysis, design and assessment.

Contrary to the above state-of-practice, however, current research has shown other evidence (Finn, 2005; Gazetas, 2006; Pecker, 2007; Mylonakis and Gazetas, 2000; Wolf and Preisig, 2003). The foundation is indeed flexible and dissipates energy but interacts with the surrounding soil and the superstructure in such a way that it filters seismic motion (*kinematic interaction*), while it is affected by inertial forces generated by the vibration of the superstructure in a complex manner (*inertial interaction*). This depends on various factors (Mylonakis and Gazetas, 2000; Wolf and Preisig, 2003): intensity of ground motion, dominant wavelengths, angle of incidence of the seismic waves, stratigraphy, stiffness and damping of soil, and size, geometry, stiffness, slenderness and dynamic characteristics of the structure. This complex and coupled interaction may lead to a significant increase of displacement and ductility demand under certain circumstances (Finn, 2005; Gazetas, 2006; Pecker, 2007).

Bridge structures are also sensitive to the assumptions made regarding the selection of an earthquake ground motion scenario in terms of frequency content, duration, amplitude and directivity. Moreover, spatial variability of earthquake ground motion (SVG) may be significantly different at support points of long bridges, due to (a) the *traveling of the waves* at a finite velocity (Der Kiureghian *et al.*, 1997), (b) the *loss of their coherency* in terms of statistical dependence due to multiple reflections, refractions and superposition of the incident seismic waves (Zerva, 2009), (c) *attenuation of motion* due to geometrical spreading of the wave front and the loss of kinematic energy during propagation, (d) the effect of local site conditions, (e) pier-dependent kinematic interaction at the soil–foundation interface (Sextos *et al.*, 2003a) and (e) non-uniform liquefaction along the bridge length (Elnashai *et al.*, 2009).

The SVG phenomenon can lead to significant spatial and temporal variations of seismic motion in terms of amplitude, frequency content and arrival time, thus inducing significant forces and deformations under certain

circumstances (Sextos *et al.*, 2003b; Zerva and Zervas, 2002). Similar to SSI, the perception often prevails that the uncertainty in modeling the variation of earthquake ground motion in space is higher than the uncertainty induced by ignoring the problem itself. Hence in most cases, asynchronous excitation of long structures is either neglected or taken into consideration in a very approximate manner (Sextos, 2009). Considering the two problems from a physical point of view, it can be made clear that SSI and SVGM are essentially two strongly coupled phenomena.

The scope of this chapter is to critically discuss methods for considering the above two phenomena in the design and assessment of RC bridges. An effort is therefore made to demonstrate a simple yet accurate means to account for SSI during the design process of bridges based on state-of-the-art scientific publications, seismic code provisions, related background commentary as well as expert engineering judgment. Additionally, a series of modeling issues are discussed in terms of ‘optimal’ selection and application of seismic input (with emphasis on site response, varying angle of wave incidence, multiple support and rotational excitation). The seismic response of the soil–structure system (particularly in terms of soil–pile–pier and embankment–backfill–abutment–superstructure interaction and various sources of material and geometrical nonlinearity) is also discussed. The chapter concludes with a critical overview of the methods presented and a discussion of the future trends in research and design.

22.2 Soil–foundation–pier–superstructure interaction

The design of the foundation of piers and abutments primarily depends on (a) the depth, stiffness and bearing capacity of the bearing stratum, (b) the structural system and the anticipated vertical and lateral loads, and (c) potential restrictions of differential settlement due to the type of bridge deck (i.e., multi-span continuous decks can tolerate smaller differential settlements than simply supported ones). These foundations generally fall into two categories: strip footings and pile foundations. Combination of the two (i.e. piers supported on piles and abutments on surface foundation) is also feasible given the soil inhomogeneity. In the following subsections, only pile foundations will be examined as it represents the most complex case of the foundation systems, in terms of modeling, analysis, design and construction.

22.2.1 Piers founded on pile shafts (single or groups)

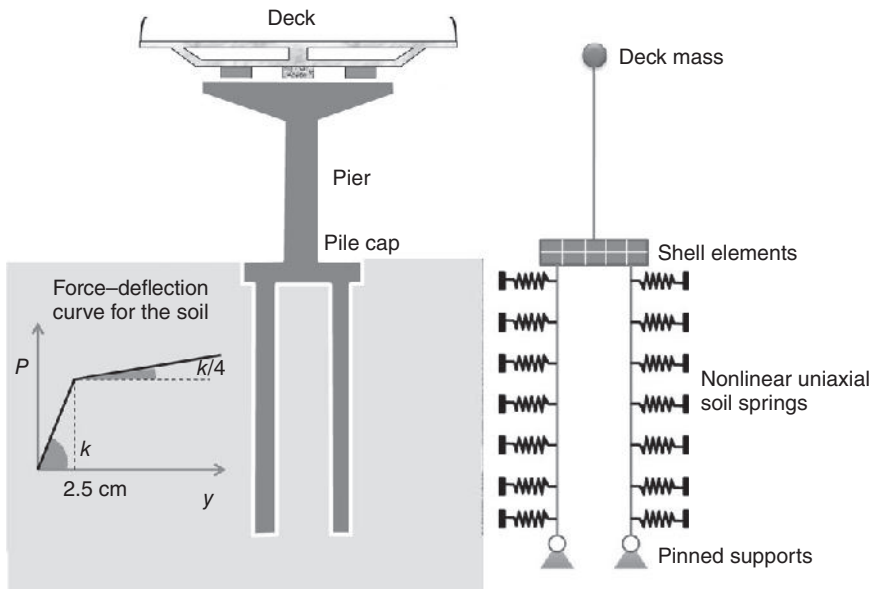
Static loading

Numerous studies have been made on the effect of SSI under static loading. The most commonly adopted engineering method for calculating the

pseudo-static interaction between pile and soil is the Winkler model (Matlock and Reese, 1960) in which the soil reaction to pile movement is represented by independent (linear or non-linear) unidirectional translational spring elements distributed along the pile shaft to account for the soil response in the elastic and inelastic range, respectively (Fig. 22.1). Although approximate, Winkler formulations are widely used not only because their predictions are in good agreement with results from more rigorous solutions, but also because the variation of soil properties along the pile length can be relatively easily incorporated. Moreover, they are significantly more efficient in terms of computational time required, especially compared to continuum finite element (FE) or finite difference methods, thus permitting focus to be placed on the structural inelastic response.

The mechanical parameters for the spring elements are frequently obtained from experimental results (leading to P - y curves for lateral and T - z curves for axial loading) as well as from simplified models. An expression that is still widely used for the lateral soil resistance–deflection relationship is (API, 1993):

$$P = 0.9p_u \tanh \left[\frac{kH}{0.9p_u} y \right] \quad [22.1]$$



22.1 Overview of finite element (FE) modeling of pile groups using uniaxial soil springs.

where p_u is the ultimate bearing capacity at depth H , y is the lateral deflection and k is the initial modulus of subgrade reaction which is both depth- and diameter-dependent despite the fact that in many cases (Pender, 1993) the modulus of the subgrade reaction is assumed to be independent of diameter.

The particular expression is also applied both for dynamic or non-linear problems after a simple transformation to a bi-linear relationship by assuming a specific threshold deformation Δ_y for entering the inelastic range (typically equal to 2.5 cm for cohesionless soils) and a second branch stiffness reduced to a quarter of the initial soil stiffness (Kappos and Sextos, 2001). As an alternative to the above procedure, the static stiffness extrapolated from the complex dynamic stiffness matrix is also used in practice as discussed in the following section.

In addition to the springs attached along the pile shaft, a horizontal inelastic soil spring can be used at the top of the pile to represent the strength and stiffness provided by passive soil resistance against the pile cap, while a vertical, uniaxial, inelastic spring is commonly used at the pile tip to account for downward and upward capacity of the supporting soil (Pender, 1993).

For the case of non-uniform, liquefaction susceptible soil profiles, the lateral subgrade reaction of piles and the maximum reaction force of the laterally spreading soils have to be appropriately reduced at the corresponding locations along the pile length. This reduction factor lies in the range of 0.1–0.2 (Finn, 2005) or 0.05–0.2 (Elgamal *et al.*, 2006; Suzuki *et al.*, 2006). As a rule of thumb, the lateral stiffness provided along the liquefied soil layers is reduced to the level of 10–20% depending on the estimated shear deformation.

Based on the above discussion, from a static response point of view, it can be concluded that the use of lateral soil resistance–deflection curves is a convenient approach for estimating the dynamic characteristics of the bridge–structure system (Kappos and Sextos, 2001). Nevertheless, despite the wide application of the P – y approach for the assessment of the structural response in the design practice, there are certain limitations that have to be stressed:

- Uncertainty of estimating the parameters involved when load tests are not available (especially of defining p_u and k), is disproportionally high compared to the simplicity of the approach. It is notable that although eq. 22.1 is adopted by both the Multidisciplinary Center for Earthquake Engineering Research and American Technology Council (MCEER/ATC, 2003) as well as the California Department of Transportation (CALTRANS, 2006) guidelines, the proposed sets of the required subgrade moduli differ on average by a factor of 4 (Finn, 2005).

- Relationships between one-dimensional soil stiffness (expressed in terms of modulus of subgrade reaction k) to two-/three-dimensional soil stiffness, expressed in terms of modulus of elasticity E_s and Poisson's ratio ν , has not been established. The verification using 2D and 3D FE analyses is not straightforward. As a result, a set of calibration assumptions is required for establishing a correspondence between the Winkler and plane-strain FE approaches (Kappos and Sextos, 2001) based on the initial formulations proposed (Vesic, 1961):

$$k = Dk_h = \frac{0.65E_s}{(1-\nu^2)^2} \sqrt[12]{\frac{E_s D^4}{(1-\nu^2)E_p I_p}} \quad [22.2]$$

where k_h is the modulus of subgrade reaction and D the pile diameter. Moreover, the transformation from one soil parameter to another is straightforward only in a case where they are assumed constant with depth; a set of additional and case-dependent calibrations is required to obtain agreement in the inelastic range.

- Pile group effects are essentially neglected. Even if the piles are statically connected using appropriate single valued springs to represent the increased flexibility of a pile group compared to the summation of the stiffness of all individual piles, the estimation of the connecting spring stiffness is highly subjective.
- Use of the (particular statically based) P - y method often leads to the extension of its application for the case of inelastic dynamic analysis in the time domain. Such an extension, although tempting for special cases of structural design (i.e. performance-based design of new or retrofit of existing important bridges), leads to the misleading perception of modeling refinement without proper understanding and consideration of the complex dynamic nature of SSI phenomena.

It can be concluded that as soil–foundation–structure interaction is a multi-parametric and strongly frequency-dependent phenomenon, it has to be investigated from a dynamic point of view, through a very careful selection of FE models, associated parameters, and modeling assumptions. As a result, the static Winkler spring approach is deemed appropriate only for cases where:

- bridges are analyzed linearly solely under static loading and the response is assessed accordingly;
- (standard or modal) pushover analysis is performed in order to quickly assess the inelastic mechanisms that are expected to be developed under earthquake excitation;
- response spectrum analysis and/or linear time history analysis is performed for a relatively low level of seismic forces;

- inelastic dynamic analysis is conducted in the time domain but the energy absorption is expected to be mainly concentrated on the superstructure (i.e., through bearings or other dissipating devices) while the material and radiation damping at the soil–foundation interface is a-priori judged of secondary importance (i.e. in cases where the underlying soil formations are stiff and uniform with depth).

Dynamic loading

Owing to the large dimensions of bridge structures, consideration of SSI effects is often made by implementing finite element discretization, either through a complete FE or substructure approach. The complete FE approach is primarily used when the overall wave propagation in 2D or 3D space is of primary interest and the response of the soil–foundation–superstructure is mainly linear elastic. It permits modeling of the entire bridge–soil system and a balanced refinement between the numerical representation of the structure and the subsoil domain; however, it has significant limitations as regard to the accuracy of seismic wave propagation especially in cases of inhomogeneous media where the finite difference method is deemed preferable. Realistic boundary conditions (Novak and Mitwally, 1988) and optimal mesh dimensions (Lysmer and Kulemeyer, 1969) are also a prerequisite.

On the other hand, according to the substructure method, the soil–foundation–bridge system is divided into distinct substructures, typically the superstructure, near field soil domain inclusive of the foundation, and far field domain. Owing to considerably reduced computational cost, the substructure method has been extensively used in the past in one of the following forms:

- *Coupled finite element method/boundary element method (FEM/BEM) approaches* (Renault and Meskouris, 2005): the advantage of this approach is that the soil can be discretized only in the interaction horizon, while the boundary conditions are consistent and hence, the wave propagation in the free-field can be accurately calculated considering non-relaxed boundary conditions.
- *Uncoupling and superposition of kinematic and inertial interaction* (Makris *et al.*, 1996; Mylonakis *et al.*, 1997): the dynamic stiffness matrix of the superstructure is attached to an additional impedance matrix representing the underlying unbounded soil region and the superstructure is then excited by the response history (denoted as foundation input motion – FIM) of a hypothetical soil–foundation subsystem without superstructure mass. In case of deep (pile) foundation, the procedure can be summarized in four independent steps:

- Analysis of free-field soil response (i.e. without the presence of piles) to vertically propagating incident shear waves.
- Analysis of interaction of the single pile or pile group with the surrounding soil, driven by the free-field response of the previous step.
- Computation of the dynamic impedances (in the form of dynamic springs and dashpots) at the pile head or the pile-group cap, associated with the swaying (R_x, R_y), rocking (R_{ry}, R_{rx}) and cross-swaying-rocking ($R_{x,ry}, R_{y,rx}$) motion of the foundation. It is noted that specifically for pile groups, the dynamic impedance of the foundation cannot be computed by simply adding the dynamic stiffness of the individual piles. The reason is that, pile-to-pile interaction is frequency-dependent, as resulting from waves that are emitted from the periphery of each pile and propagate to strike the neighbouring piles (Nikolaou *et al.*, 2001). A variety of numerical and analytical methods have been developed (El Naggar, 1995; Makris and Gazetas, 1992; Mylonakis and Gazetas, 2002; Nogami *et al.*, 1992) to compute the dynamic response of this interaction. Especially for the rotational stiffness of foundations supporting bridge piers that are expected to develop plastic hinge at their base, a non-linear moment–rotation relationship can be implemented (Sextos *et al.*, 2003a) combining the rotational compliance of the foundation with a lumped plasticity model for the RC section.
- Analysis of dynamic response of the superstructure supported on the previously defined impedance system, subjected to the kinematic pile-head motion already computed. It is noted that the above approach is similarly applied for the case of piers and abutments that are supported on surface foundations. A summary of closed-form solutions of dynamic stiffness of spread footings that have been derived by regression analysis, based on finite- and boundary-element data can be found elsewhere (Mylonakis *et al.*, 2006a). Commonly, due to their small embedment, the cross-swaying-rocking stiffness of the spread footings can be neglected.

22.2.2 Soil–structure interaction and characteristics of seismic ground motion

In the four steps described above, the highest level of uncertainty is associated with the identification of the incoming wavefield. It has long been shown that the dynamic response of both subsystems (i.e. massless soil–foundation and superstructure with flexible foundation) is strongly frequency-dependent. Hence the characteristics of the earthquake ground motion play a predominant role in the response of the soil–bridge system. Consideration of dynamic soil–structure interaction effects cannot be seen

independently from the characteristics of the incoming seismic motion. On the other hand, the dynamic characteristics of a bridge subjected to strong ground motion are also evolutionary in time due to damage accumulation in terms of frequencies of vibration and contribution of individual modes. The collapse of the Hanshin Expressway during the Kobe (1995) earthquake due to significant detrimental SSI effects (up to 100% increase) is a notable example of the critical interplay between the soil, bridge and earthquake characteristics (Mylonakis *et al.*, 2006b). Other studies (Sextos *et al.*, 2003b) have also highlighted the importance of local soil and site conditions when considering coupled soil–bridge interaction issues. The impact of SSI effects on seismic bridge displacements may be underestimated by a maximum factor of 1.5–2.5, if the local 1D/2D/3D basin site effects have not been properly incorporated in SSI analysis. Numerous works (Zhang and Makris, 2002a,b) have also shown that mechanical properties and geometry of the abutments significantly affect the dynamic response of short to moderate length overpasses; thus it may be highly unconservative to introduce the abutment–embankment system flexibility without due consideration of the incoming seismic energy to the bridge boundaries.

22.2.3 Consideration of soil nonlinearity

As the earthquake intensity increases, the behavior of the supporting soil deposits quickly becomes nonlinear, thus introducing additional flexibility and damping at the soil–foundation interface. A large number of sophisticated constitutive laws for soil materials have been developed and incorporated into modern FE codes (Sextos, 2011). Nevertheless, the numerical simulation of concurrent inelastic mechanisms developing at the soil, foundation and structure simultaneously is still computationally demanding and a major challenge due to material and epistemic uncertainties. This can be primarily attributed to the following:

- Despite the significant advances in modern computer software, it is difficult to achieve a balanced modeling refinement for all materials as most computer codes do not simultaneously support built-in features for yielding mechanisms specialized for soil, concrete, steel and rubber. Furthermore, coupling of different material laws often leads to significant analysis convergence difficulties.
- The data required to prescribe various yield parameters (i.e. von Mises, Willam–Warnke, Mohr–Coulomb, Drucker–Prager) cannot always be reliably assessed, thus, implicitly undermining the foreseen analysis refinement. This can be to some extent tackled through a probabilistic analysis framework.

- Even though general purpose numerical analysis software can now efficiently solve geometric non-linear problems (i.e. sliding at the pile–soil interface, sliding and rocking at the strip footing–soil interface, closure of deck-abutment gaps, etc.), analysis convergence is not ensured during simultaneous material and geometrical non-linear phenomena, neither can the results be easily verified.

One solution to the above-described problem is to assess in advance the locations of the soil–foundation–superstructure system that are anticipated to be subjected to the highest level of inelastic demand and develop the FE modeling strategy accordingly (for instance by prescribing refined mesh and nonlinear capabilities solely to specialized areas of interest, i.e. upper soil layers of the pier foundations and backfill).

22.2.4 Effect of soil liquefaction on the dynamic bridge response

The vulnerability of bridge foundations to liquefaction-induced lateral spreading has been clearly demonstrated by the extensive damage observed in many past earthquakes, i.e., Alaska (1964), Loma Prieta (1989), Costa Rica (1991) and Kobe (1995), among many others. Since then, extensive experimental testing (on large-scale single piles and smaller centrifuge samples) and numerical research have been conducted. The numerical research can be classified into three main categories: p -multiplier method (Baziar and Dobry, 1995), p – y spring coupled with continuum soil material models (Kulasingham *et al.*, 2004), and three-dimensional continuum method (Elgamal *et al.*, 2008; Varatharaj and Muraleetharan, 2004). More recently, liquefaction has also been considered in the framework of vulnerability analysis (Bayram *et al.*, 2011; Brandenberg *et al.*, 2011; Elnashai *et al.*, 2009; Kwon *et al.*, 2008), clearly demonstrating the significant effect of liquefaction on the seismic fragility of bridges through multiple different excitation mechanisms. An example of the effect of liquefaction on ground motion and soil stiffness (Zhang *et al.*, 2008) involves strong shaking, non-linear, response history analyses where lumped spring and damping models are used to represent the soil and foundation and consideration of lateral spreading through incremental static analyses with prescribed displacement of the laterally spreading soil. It is believed that future research will focus on improved computational methods to establish a more reliable relationship between ground motion intensity measures (IMs) and structural demand in the presence of liquefaction. However, determining the optimum IM is still a challenge especially for systems where variability in soil, struc-

tural material and liquefaction potential are jointly present with geometrical non-linearities (Brandenberg *et al.*, 2011).

22.3 Embankment–backfill–abutment–superstructure interaction

22.3.1 Static embankment–backfill–abutment interaction

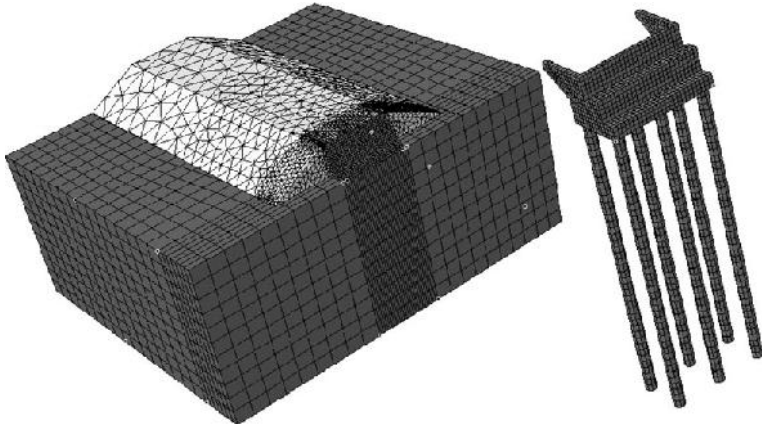
Various static spring-based approaches have long been developed for the study of soil–abutment–deck interaction (Shamsabadi *et al.*, 2007; Spyarakos and Ioannidis, 2003; Stojadinovic and Mackie, 2002). Despite the importance of modeling complex bridge lateral boundary conditions and the existence of specific guidelines in the US (Caltrans, ATC, MCEER) and in Europe (Eurocode 8-2) for the design of pile foundations and abutments, only minor guidance is provided by codes for numerical modeling of the entire coupled soil–bridge system. As a result, engineers resort to the simplified relationships prescribed by Caltrans to estimate the abutment–backfill soil capacity and stiffness (CALTRANS, 2006). Based on passive earth pressure tests and the force deflection results from large-scale abutment testing at UC Davis, a value for the initial stiffness is derived proportionally to the backwall height:

$$K_{\text{abut}} = K_i \times w \times (h/1.7) \quad [22.3]$$

where w is the width of the backwall and $(h/1.7)$ is the proportionality factor based on the 1.7 m height of the UC Davis abutment specimen. The ultimate abutment load can be assumed to be limited by a maximum static soil passive pressure of 239 kPa. In the transverse direction, the abutment stiffness and strength obtained for the longitudinal direction can be modified using factors corresponding to a wing wall effectiveness and participation coefficients of 2/3 and 4/3, respectively (Maroney and Chai, 2004).

Another simplified alternative is the performance of a separate pushover analysis that can be performed for the abutment and foundation systems in order to quantify the lateral support stiffness of the bridge (Kappos *et al.*, 2007). The advantage of this approach is that both soil and (abutment and foundation) concrete nonlinear behavior can be considered as a means to provide case-specific force–deflection (i.e. P – y) relationships that can, in turn, be used as nonlinear spring boundary constitutive models in the push-over analysis of the overall bridge structure.

Another effort was recently made (Sextos *et al.*, 2008) to extend the above concept by performing a set of 3D nonlinear FE analyses on typical Californian overpass abutment–embankment systems in order to provide simplified P – y relationships of the lateral supporting system as a function

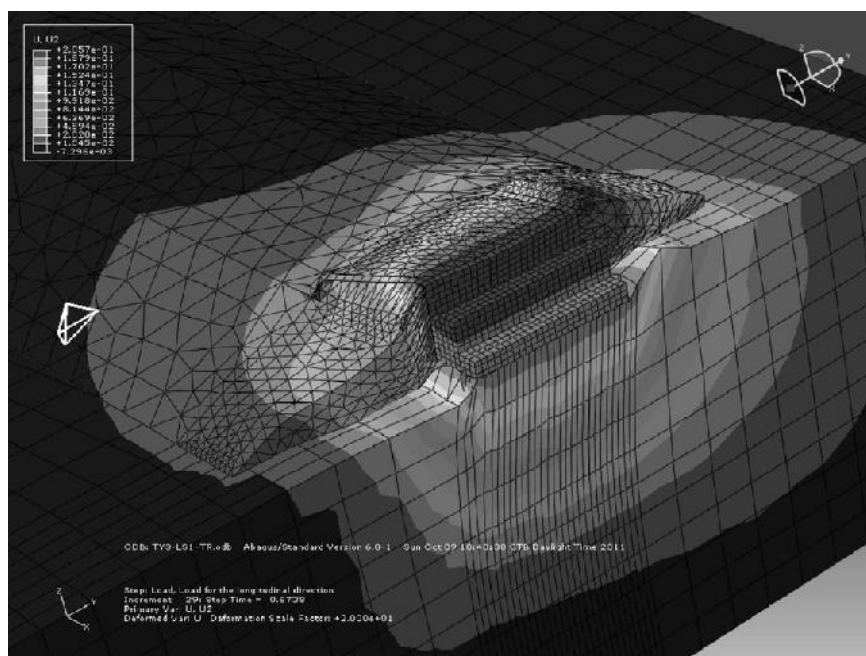


22.2 Finite element models of the abutment-embankment system of the TY3 overpass along the Egnatia Highway, Greece.

of abutment type, foundation-embankment-backfill geometry, and soil properties. The idea was to derive relationships that can be potentially used in cases where more accurate data are not available. Six typical RC Californian bridges (namely Route 14, LADWP, W180, MGR, Adobe and La Veta) and a Greek one (TY3 overpass along the Egnatia Highway) consisting of box-girder superstructures, seat-type abutments and shallow pile foundations were studied. Given the short spans and relatively high deck stiffness of the particular structures, the embankment mobilization and the inelastic behaviour of the soil material under high shear deformation levels was found to have a significant effect on the response of the bridge under seismic loading. The geometry of the TY3 case studied is illustrated in Fig. 22.2, while indicative distribution of plastic strains in the 3D space are depicted in Figs 22.3 and 22.4 for the transverse and longitudinal direction, respectively. The relative stiffness of the embankment-abutment system of all bridges examined is indicatively presented in Fig. 22.5 in the form of pushover curves reaching an ultimate displacement of 10 cm, assumed to be a limit value of abutment shear failure.

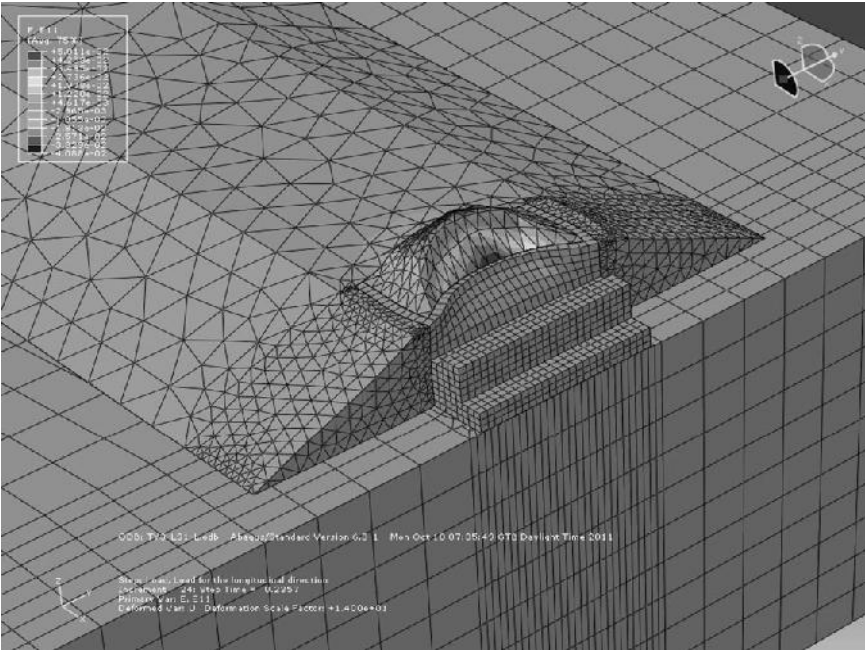
22.3.2 Dynamic mobilization of bridge embankments under seismic excitation

Recent research studies (Goel and Chopra, 1997; Kotsoglou and Pantazopoulou, 2007, 2009; Makris and Zhang, 2004; Mwafy *et al.*, 2010) showed that for flexible abutment-pile foundation systems, conventional pushover analysis studies may be insufficient. A notable reason is the nonlinear

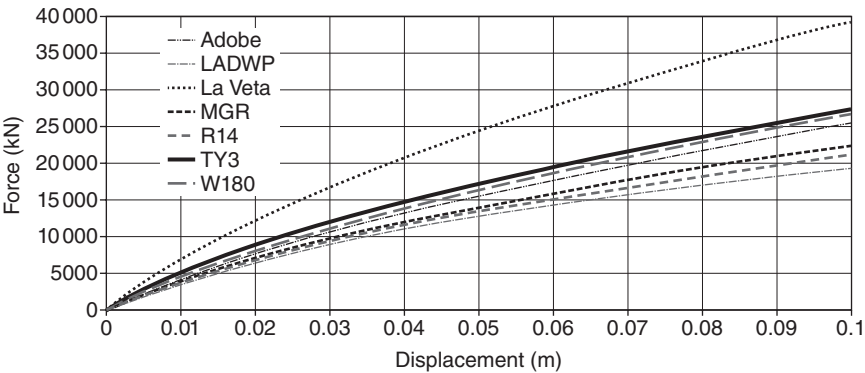


22.3 Plastic strains developed at the backfill of the approach embankment of the TY3 bridge (transverse direction).

behavior of the embankment and backfill soil under high levels of shear strain, even if the bridge superstructure remains linear. Another reason is the mobilization of the embankment mass during strong ground shaking which may act as a significant source of seismic energy that is transmitted to the bridge through the embankment–abutment–deck boundaries. In such a case, inertial and kinematic interaction effects between the bridge and the approach embankments may be significant, especially in the transverse direction. Moreover, in case of short bridges, the embankments may drive the overall response of the bridge (Zhang and Makris, 2002b), thus forcing the central piers follow the large, almost rigid body, displacements imposed by the deck–abutment system. This is a problem as it is strongly frequency dependent and has the significant implication that conventional response modification factors prescribed in bridge design codes (R in the US and q in Europe) in order to reduce pier forces depending on the bridge system period and ductility, may not be applicable (Kotsoglou and Pantazopoulou, 2009). Notwithstanding the above advances in understanding dynamic embankment–abutment–deck interaction therefore, further research is warranted.



22.4 Plastic strains developed at the backfill of the approach embankment of the TY3 bridge (longitudinal direction).



22.5 Force-displacement (pushover curve) of the embankment-abutment system of the TY3 bridge (longitudinal direction).

22.4 Realistic earthquake excitation scenarios for interactive soil–bridge systems

As all different aspects of soil–foundation–bridge interaction already presented are found to be frequency-dependent, it is clear that a reliable esti-

mate of the characteristics of incoming seismic motion is of paramount importance. Along these lines, there are at least five key factors in forming a realistic excitation scenario for bridge design and assessment: (a) overall seismotectonic features of the area, (b) quantitative criteria provided in modern seismic codes and design guidelines (typically related to rules for matching a prescribed spectrum), (c) modification of seismic motion to consider kinematic soil–structure interaction, (d) direction of excitation and consideration of the rotational component of earthquake ground motion, and (e) spatial variability of seismic motion. All these issues are discussed below.

22.4.1 Selection and generation of earthquake ground motion scenarios

Earthquake record selection tools

Numerous computational tools have been developed for selecting suites of earthquake records from available strong ground motion record databases. Most common earthquake record selection procedures involve spectral matching of the average response spectrum of the records to be used, with a target, code-prescribed or seismic hazard-defined elastic response spectrum (Beyer and Bommer, 2007), or a conditional mean spectrum (Baker *et al.*, 2011). Recent work evolved to develop methods and computational tools for quantifying and/or optimizing the spectrum compatibility (Naeim *et al.*, 2004; Kottke and Rathje, 2008). In case of the performance-based design approach, the selection of acceleration time series is considered with the goal of accurate prediction of the structural response at a specified ground motion IM. Most commonly, the peak ground acceleration (PGA) of the eligible records and some other characteristic parameters (i.e. the spectral acceleration, S_a) have been used as suitable IMs. Nevertheless, advanced intensity measures, including information about the spectral shape and structural characteristics, are preferable for records selection and scaling procedures as they result in a more accurate and reliable estimate of the seismic demand (Baker *et al.*, 2011; Huang *et al.*, 2009).

Despite the aforementioned state-of-the-art evolution in the research field, a rather rough framework is prescribed by modern seismic codes and guidelines (inclusive of Eurocode 8, CEN, 2004, and FEMA P-750, FEMA, 2009) concerning the motions to be used for time history analysis. In fact, most of the aforementioned record selection methods proposed in the literature have not yet been incorporated in any seismic code worldwide, despite the fact that evidence exists that this leads to large dispersions of structural response (Katsanos *et al.*, 2010). Along these lines, numerous computational tools have been developed to raise the limitations imposed

by the oversimplifying approach of the code-based, earthquake record selection procedures. REXEL (Iervolino and Galasso, 2010; Iervolino *et al.*, 2009) is the first software introduced for this purpose and facilitates the search for suites of waveforms compatible with target spectra that are either user defined or automatically generated according to Eurocode 8 and the recently issued Italian seismic code. An alternative web-based software for earthquake record selection is SeEQ (Dias, 2010) offering various filtering options. More recently, a MATLAB-based software ISSARS (Integrated System for Structural Analysis and Record Selection) has been developed (Katsanos *et al.*, 2011) connecting through the selection engine Internet to the aforementioned PEER-NGA ground motion database, to form suites of records that comply with specific criteria. These suites of records are then ranked based on their compatibility with the design spectrum by maintaining the resulting level of dispersion of structural response quantities as low as possible. This is made feasible by using the Applications Programming Interface of the finite element program SAP2000 (C&S, 2008) to run numerical analyses at the background and quantify the discrepancy of structural response as a part of the earthquake record selection process.

Generation of synthetic and artificial earthquake motion tools.

Design and assessment of extended structures often require the generation of spatially variable earthquake motions to be used as multiple-support inputs. A review of the methods most widely used can be found in Zerva and Zervas (2002). Site effects and SSI can also be accounted for in the signal generation process (Sextos *et al.*, 2003a). The use of existing, appropriately modulated, records to prescribed target earthquake characteristics (nonstationary in amplitude, frequency content, earthquake magnitude, local site conditions, duration, etc.) is another advantageous alternative (Liao and Zerva, 2006). Wavelets analysis, a powerful technique which uses various mother wavelets, usually different from sinusoidal function like Fourier method, and then adjusts the wavelet by shifting and scaling, thus achieving an effective way of characterizing a non-stationary time history data, has also been used for generating spectrum compatible synthetic accelerograms (Giaralis and Spanos, 2009; Mukherjee and Gupta, 2002) for investigating the seismic response of both structures (Giaralis and Spanos, 2010) and geotechnical systems (Haigh *et al.*, 2002).

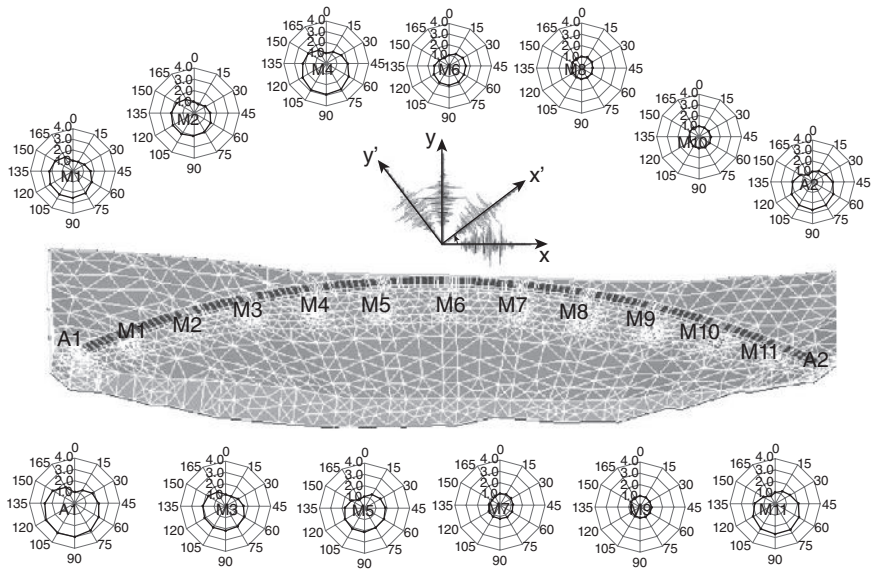
22.4.2 Rotational component of the foundation response:

Kinematic interaction between pile and soil, apart from filtering the high-frequency content of the incident seismic waves, induces rotations at the

pile head, especially in the case of single piles or footings with a very small number of piles (Gazetas and Kavvadas, 1993; Mylonakis *et al.*, 1997), leading to additional rotations of the piers in the transverse direction. This is a crucial issue, though commonly neglected during the excitation of the bridge, despite the fact that numerous bridges in the US have been founded on such single, cast-in-drilled hole (CIDH) piles. Recent work (Sextos, 2011) makes use of the analytical expressions proposed by Maravas *et al.* (2007) to simultaneously consider the translational and rotational component of seismic excitation, produced by the kinematic interaction between the pile foundation, the soil and the vertically propagating S-waves. It is found that the effect of the rotational component is higher for cases of soft soil profiles and flexible, taller piers. Most importantly, the kinematic interaction factor $I_{u+\omega}$ under both seismic translational and rotational components may exceed 1.5 times the interaction predicted based on the assumption that translational motion alone in certain frequencies associated with the dynamic characteristics of the structure and the incoming motion. The issue of the rotational excitation of bridges supported on CIDH pile foundations is another promising field of research which presents major practical interest.

22.4.3 Direction of excitation

The importance of the angle of ground motion incidence has been highlighted by many researchers, working primarily on a principal direction model (Kubo and Penzien, 1979; Penzien and Watabe, 1975) where translational components of ground motion along the principal directions of ground motion records are uncorrelated. Recent research from Athanatopoulou and Doudoumis (2008) and Taskari *et al.* (2008), among others, has also shown significant discrepancy between the response quantities produced when the seismic components are applied along the structural axes and those resulting from the excitation of the structure for different angles of wave incidence. Figure 22.6 indicatively illustrates the variation of response quantities for various angles of seismic motion incidence for the case of the long (640 m) and curved Krystallopigi Bridge in Greece (it is noted that 180 degree rotations of two horizontal components are sufficiently used to define all exhaustive rotation cases). It is seen that both displacements and bending moments may vary by up to 100% depending on the direction of excitation, while it is evident that a ‘critical’ angle of incidence that would simultaneously maximize all response quantities at all structural members cannot be defined. Unfortunately, the current seismic design framework is unclear as to the principal axes of excitation especially in the case of curved bridges. The direction of base excitation is usually



22.6 Finite element model of the Krystallopigi Bridge and definition of excitation direction.

assumed to be parallel and perpendicular to the chord for the two seismic input components, respectively.

22.4.4 Asynchronous excitation

Sources of spatial variation of earthquake ground motion

The spatial variation of earthquake ground motions is another aspect that is not accounted for in the vast majority of the design cases. This is a reasonable approximation for buildings or short bridges, but it may considerably alter the overall seismic response of more extended structures especially in the case where the local site conditions vary significantly with length (Deodatis *et al.*, 2000; Der Kiureghian *et al.*, 1997; Sextos *et al.*, 2003b). The particular problem is very complex and various methods have been proposed for tackling the uncertainty related to the definition of a 'realistic' support-dependent earthquake input, as well as for predicting the conditions under which the effect of asynchronous input is indeed critical for the structure.

Seismic code framework

Despite the major practical relevance of generating artificial spatially varying motions and the associated research carried out over the last years,

the multi-parametric nature and the complexity of the problem have not yet permitted the development of specific design provisions to be included in modern seismic codes, hence the effect of asynchronous excitation is only partially or indirectly considered. More specifically, most modern codes in the US and Japan deal with the problem solely on the basis of seating length provisions. AASHTO prescribes a minimum bearing support length N_s for the expansion ends of all girders which is a function of the length of the deck L (in meters), the height H of the column or pier (in m), and the skew angle α of the support (in degrees), based on the following dual relationship and the Seismic Performance Categories (SPCs, in particular A, B, C and D, as defined in AASHTO):

$$N_s(\text{in mm}) = \begin{cases} (203 + 1.67L + 6.66H) \cdot (1 + 0.000125\alpha^2) & \text{for : SPC}_{\text{AandB}} \\ (305 + 2.50L + 10.0H) \cdot (1 + 0.000125\alpha^2) & \text{for : SPC}_{\text{CandD}} \end{cases} \quad [22.4]$$

A similar expression is provided by MCEER/ATC-49 (MCEER/ATC, 2003) wherein the minimum seating length N_s is prescribed as a function of the distance between joints L , the tallest pier height H between the joints, the width of the superstructure B and the skew angle α :

$$N_s(\text{in mm}) = \left[0.10 + 0.0017L + 0.007H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(2\frac{B}{L}\right)^2} \right] \cdot \frac{1 + 1.25F_v S_1}{\cos \alpha} \quad [22.5]$$

where the ratio B/L should not exceed $3/8$, F_v is the site coefficient for the long-period branch of the design response spectrum, and S_1 the one-second period spectral acceleration.

Moreover, it is recommended that if geotechnical conditions at abutments and intermediate piers result in different soil classifications, then response spectra should be determined separately for each abutment and pier, having a different site classification and then the design response spectra can be taken as the envelope of the individual spectra.

An effort to relate the expected relative displacements of the deck δ_a of a multiply excited bridge system to the overall length L , has also been made through a statistically derived amplification factor R_D (Sextos *et al.*, 2003b):

$$\delta_a = R_D \delta_s = (0.8 \ln(L) - 2.8) \delta_s \quad [22.6]$$

where δ_s is the relative displacement that would result from ‘standard’ synchronous motion analysis and L is the overall bridge length (in m). A model to compute the differential displacements of points on the ground and on the top of a SDOF linear elastic system has also been proposed while

Albansesi *et al.* (2007) have suggested an alternative approach based on the use of a ‘relative displacement spectrum’.

Eurocode 8 on the other hand, in the latest version of its Part 2 referring to Bridges (CEN, 2005), also prescribes increased levels of minimum seating (or ‘overlap’) lengths for the abutments, to ensure the function of the support under extreme relative displacements that may result due to spatial variability of seismic motion. It is also noted that the relevant seating lengths are estimated with an additional safety margin equal to $\sqrt{2}$ compared with the safety factor imposed for the stresses induced at the bridge system. The most important development in Eurocode 8 is that it is currently the only seismic code worldwide that provides a clear and detailed framework for considering the effect of spatial variability of ground motion in bridge design, prescribing both a simplified and an analytical approach, the latter being included as an ‘informative’ annex.

In particular, Eurocode 8 clearly recognizes that since spatial variability of seismic action is a situation wherein the ground motion at different supports of the bridge differs, the seismic action cannot be based on the characterization of motion at a single point. According to EC8, spatial variability has to be considered for bridges of continuous deck when one or both of the following conditions apply: (a) soil properties along the bridge vary in such a way that the soil at the various supports corresponds to more than one category (as specified in Eurocode 8 – Part 1), and (b) soil properties along the bridge are approximately uniform, but the length of the continuous deck exceeds a specified limit, L_{lim} ; the recommended value of L_{lim} is equal to $L_g/1.5$, where the length L_g is the distance beyond which motions can be considered as completely uncorrelated and is given in Table 22.1 as a function of the ground conditions (to visualize, see Sextos and Kappos, 2009).

For the general case, the potential maximum values of the considered seismic action effect (i.e. member force or deformation) can be estimated through an adequate (albeit simplified) procedure. This method should be followed, unless a more accurate analysis is carried out. To this end, a more detailed procedure for the assessment of the asynchronous motion effects in the frequency domain (Der Kiureghian *et al.*, 1997) is proposed in an

Table 22.1 Limiting length to consider spatial variability effects, as a function of ground type according to Eurocode 8

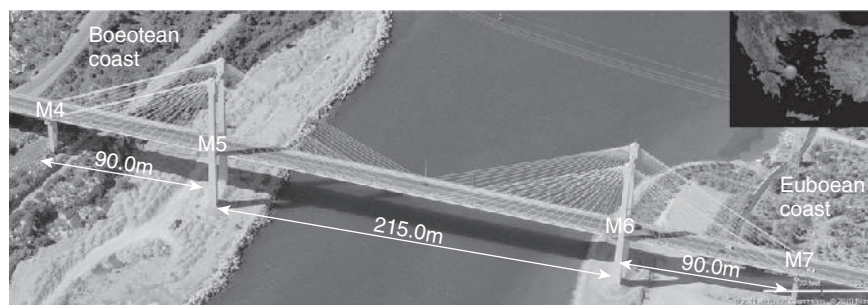
Ground type	A	B	C	D	E
L_g (m)	600	500	400	300	500
L_{lim} (m)	400	333	266	200	333

informative annex to the Code, while guidance on the generation of artificial spatially variable ground motions is also provided. The idea (put forward by the EC8-2 drafting panel) to compute, in a simplified way, the necessary stress or displacement increase due to SVGM effects is simple, practical and physically motivated. In short, various bridge supports are subjected to different values of (location-dependent) earthquake accelerations, which are partially correlated; as a result, pseudo-static internal forces develop. From the numerous possible combinations of relative support vibration, two cases are identified as the most critical: (a) all piers are subjected to ground displacements of the same sign (but varying magnitude), and (b) the two piers in each pair of two successive piers are displaced in opposite directions.

The structure is therefore subjected to two sets of pseudo-static forces called Set A and Set B displacements, whose effects are then combined with those result from a typical uniform excitation analysis using the SRSS (square root of the sum of squares) rule. It is also noted that the simplified method is based on the conventional data defining the design seismic action according to Eurocode 8 Part 1, i.e. the design spectrum estimated on the basis of seismic intensity (PGA) and ground type (five common types are identified, A to E in Table 22.1, plus two ‘special’ ones). The basic steps of the procedure as well as its implications in design are described in detail in Sextos and Kappos (2009).

Excitation of higher modes based on measured data

The main disadvantage of the above simplified approach is that it does not take into consideration the dynamic component of asynchronous excitation and the excitation of higher modes of vibration. To illustrate this, another long bridge structure was examined, utilizing data acquired by the permanent, multi-sensor monitoring system operating. The Evripos bridge, is a 694.5 m RC structure, which connects the Euboean coast in the island of Evia to the Boeotean coast in continental central Greece (Fig. 22.7). The

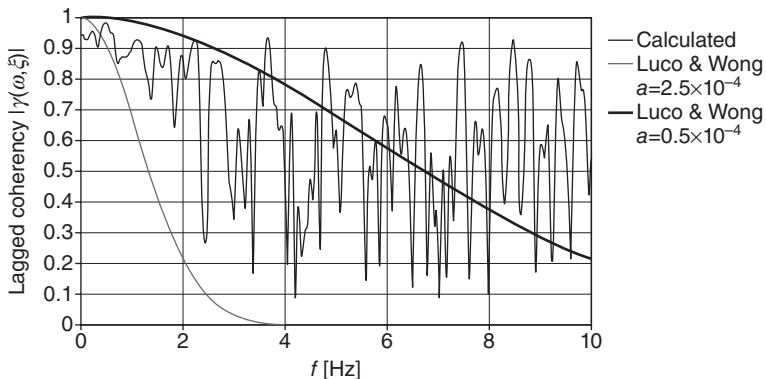


22.7 The central section of the Evripos cable-stayed bridge.

bridge is composed of three parts, the central cable stayed section and two side (approaching) parts made of pre-stressed RC beams that rest on elastomeric bearings. The central section of the bridge is divided into three spans of lengths 90 m, 215 m and 90 m, while the deck (of 13.50 m width) is suspended with cables by the 90 m height pylons at M5 and M6. The displacements of the deck along the longitudinal direction are permitted in piers M4 and M7 while those in the transverse direction are restrained (Lekidis *et al.*, 2005). The behavior of the Evripos cable-stayed bridge is constantly monitored through a special accelerometer array installed by the Institute of Engineering Seismology and Earthquake Engineering (ITSAK). The network is composed by four triaxial accelerometers installed at the base of the bridge on the pile caps of piers M5 and M6 and on soil surface in areas adjacent to piers M4 and M7. There are also 31 additional uniaxial accelerometers installed on the superstructure, for system identification purposes.

To study the spatial variation of earthquake ground motion, the bridge response recorded at four locations of the accelerometer network during the $M_s = 5.9$, 7 September 1999 Athens earthquake was utilized (Karakostas *et al.*, 2011). The records were filtered to remove inertial interaction effects and their coherency was computed for all available record pairs as seen in Fig. 22.8. A detailed FE model of the cable-stayed bridge was also developed and its response was computed using both the recorded motions and four synchronous excitation scenarios.

For the particular bridge studied, spatial variability of seismic ground motion has a generally favorable effect, at least on the pier base bending moments and the displacements at the middle of the central span deck.



22.8 Comparison between observed and predicted coherency loss for different values of the Luco and Wong model (Luco and Wong, 1986).

Apparently, the extent of this beneficial phenomenon is very much dependent on the assumptions made regarding the definition of the ‘synchronous’ excitation, which, in contrast to the actual, recorded asynchronous case, is not obvious. There are specific cases (i.e., out-of-plane bending moments and displacements at the top of the two bridge pylons) where the asynchronous excitation has a critical effect.

In all cases, the observed deamplification or amplification of the bridge displacements was verified by the reduced or increased amplitude of the Fourier spectra, at selected frequencies, which correspond to specific modes that have a strong impact on the vibration of the structure along the directions examined. Given the complexity of the problem studied, the excitation of higher modes due to asynchronous excitation may be the key tool for understanding the role of spatial variability of earthquake ground motion on the overall seismic response of bridges.

22.5 Conclusions

The present chapter is an attempt to critically review the currently available structural analysis capabilities for the assessment of SSI and spatial variability effects in the framework of seismic design and assessment of bridges. Having discussed the most widely applied approaches and highlighted a set of reasonable approximations together with their limitations, it was aimed to contribute towards filling the gap between state-of-the-art research and state-of-practice.

As a closure it can be stated that despite the major advances in terms of computational modeling and software development and powerful hardware, there is lack of ready-to-use features that would allow for efficient computation of the overall dynamic soil–foundation–superstructure response under realistic earthquake ground motion scenarios especially in the case of multiple inelastic mechanisms developing simultaneously within the soil and the structure. Tools for developing realistic ground motion scenarios are also valuable due to the inherent frequency dependence of the soil–structure system response.

As a result, it is necessary that the wide expert knowledge gained during the last decades is reflected in both ready-to-implement analysis tools and in modern seismic codes. Moreover, it is critical to utilize the large bulk of experimental soil data available worldwide together with the extensive databases of recorded motions during strong earthquakes. Finally, improving the methods for quick *in situ* assessment of the soil material properties is of utmost importance in order to minimize material-related uncertainty and focus primarily on the optimal numerical approximation required for ensuring the reliability of both analysis and design.

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Seismic risk management for water pipeline networks

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Abstract: This chapter discusses a seismic risk management method for a deteriorating lifeline network system based on the probability of system-performance failure. The chapter first reviews seismic damage to buried water pipelines and their facilities, and then discusses seismic risk assessment methods of lifeline networks in which structural and functional failures are correlated. Numerical studies are carried out for a water distribution network system by considering several seismic investment strategies to support the decision-making process in seismic disaster mitigation planning.

Key words: existing lifeline systems, performance-based design, seismic risk assessment, disaster prevention investment, probability of value loss.

23.1 Introduction

Since a lifeline network system is responsible for the lives and health of its users in addition to providing a social benefit, it is essential to keep the supply service functioning as much as possible even when a severe seismic event occurs. Originally, lifelines in Japan were designed to satisfy seismic load requirements that were based on the past significant events. Such an approach may underestimate possible damage in future scenarios, as past experiences are not necessarily the worst situation. Therefore, it is important to enhance seismic performance of existing lifelines by implementing reinforcement and retrofitting actions (Eidinger and Avila, 1999). Furthermore, these lifelines fail due to various causes, such as corrosion, traffic load, and third-party accidents (Imai and Koike, 2011), and so the original design seismic strength is not maintained.

Any existing lifeline system is composed of many pipe segments, i.e. various mixtures of old- and new-type joints (Kubota, 1992). Generally speaking, old-type joints have poor resistance against large ground motions. Approximately 80% of existing lifeline networks in Japan have old-type joints, and so it will take considerable time to replace all of them. Since many unit pipes have a very long life, replacing old-type with new-type joints will require an enormous investment. Therefore, it is necessary to consider a partial and selective retrofitting scheme, which means that some

parts of the network are replaced by new-type joints, while leaving the remaining portions intact.

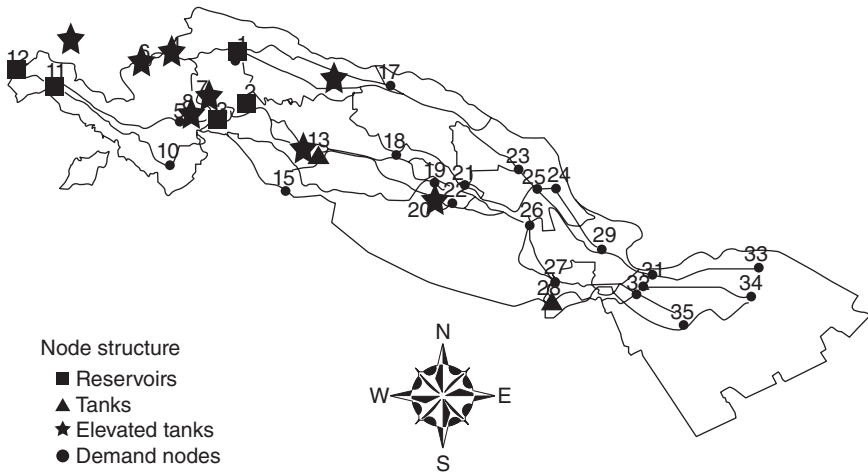
Retrofitting activities allow efficient management and operation of a deteriorating system by maintaining a favorable seismic performance level (Imai and Koike, 2009). In planning retrofitting of a system to mitigate seismic damage effectively, it is necessary to develop a procedure to quantitatively evaluate the effect of partial retrofitting on the fragility of the lifeline network system (FEMA, 1999). Some indicators, such as structural failure, performance failure, and economic loss, facilitate the evaluation of the seismic performance of the lifeline system (Imai and Koike, 2009), and are closely interrelated (Hernandez-Fajardo and Duenas-Orsorio, 2011). In particular, the supply damage rate (ratio of unserviceable users to all users of a water supply system) after an earthquake is one of the most appropriate indicators for evaluating the seismic performance of a lifeline network system from the users' viewpoint. However, since a network system normally has a hierarchical structure, with a considerable number of links and nodes, it is not simple to estimate the effect of partial retrofitting on the seismic performance.

In this chapter, a simple model is used to estimate the relationship between the probability of structural failure and loss of service without using the topological characteristics of a network system (Hernandez-Fajardo and Duenas-Orsorio, 2011; Kang and Song, 2011). A seismic-assessment procedure for a lifeline network system is proposed, using the probability of system-performance failure based on the supply damage rate as the index for evaluation, and an example seismic assessment is carried out for an actual water distribution network model by taking into account the pipeline deterioration.

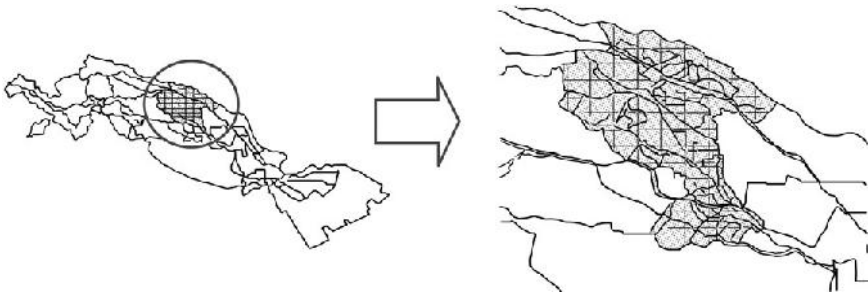
23.2 Seismic failure of a lifeline system

23.2.1 Seismic damage to a lifeline system

A water supply lifeline network system located in a metropolitan area of Japan is shown in Fig. 23.1. The water lifeline network system has a hierarchical structure, composed of transmission lines, distribution, and service network systems. The whole area is divided into several subzones, as depicted in Fig. 23.2, which are the distribution districts designated by the city's waterworks bureau. The shaded area in Fig. 23.2 is one of the water supply service areas in this model. The detailed distribution network of this area is shown on the right-hand side. Table 23.1 lists the pipe materials and their total lengths for different diameter classes within the sample pipeline network presented in Fig. 23.2.



23.1 Transmission network of the water supply lifeline system.



23.2 Distribution network in a service district of the water supply system (Wada *et al.*, 2010).

The transmission pipelines have been constructed and maintained effectively with a high-quality assurance, while the water supply distribution and service networks are operated in a sustainable manner, for both normal daily situations and emergencies, where partial leakage is tolerated. Since the lifeline system is composed of pipeline networks and control facilities, including various buildings and types of equipment, seismic damage to the lifeline system is often evaluated in terms of structural components, such as pipelines and building facilities (Straub and Der Kiureghian, 2009). In the transmission network, which has a water-purification treatment plant, water-storage tanks, pumping facilities, and control valves, structural failure of these facilities due to possible strong earthquakes needs to be prevented.

Table 23.1 Pipe materials used in the distribution area of the water-supply system

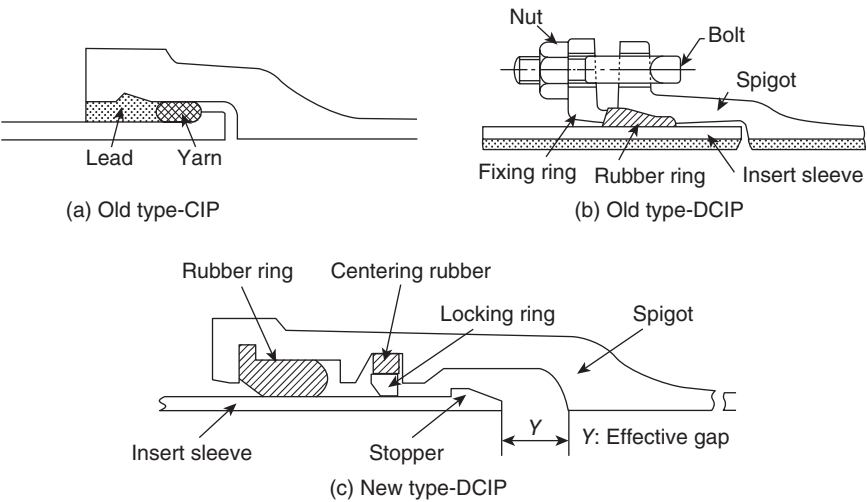
Dia. (mm)	Pipe length (m)				
	CIP	DCIP		Steel Pipe	
	Old	Old	New	Old	New
200	4 738	20 091	0	1 390	0
250	52	53	0	0	0
300	3 620	14 898	0	925	0
350	2 820	14 049	0	2 121	0
400	0	0	0	879	0
500	0	7 000	5 260	0	23 023
Subtotal	11 230	56 091	5 260	5 315	23 023
Total			100 919		

Source: Wada *et al.* (2010).

If transmission pipelines are broken, the water supply may be suspended due to pipe damage and repair works. On occasion, seismic damage within the distribution network may be limited to particular service areas by means of emergency shut-off valves. These different characteristics of transmission and distribution pipelines are reflected in the pipe materials and their joint systems.

Transmission pipelines have to be strong enough to withstand earthquake loads; as a result, arc-welded high-grade steel pipes are often adopted. The distribution and service networks, on the other hand, include old cast-iron pipes (CIP) with bell and spigot joints as well as ductile cast-iron pipes (DCIP) which are classified into two different joint types: unrestrained joint (old type) and new restrained joint (new type) (Kubota, 1992). As a result, the weakest joints of CIP are particularly vulnerable to seismic damage.

The main target of seismic retrofitting activities is to replace old-type CIP and DCIP with new DCIP or arc-welded steel pipes in the distribution network system. Recently in Japan, newly developed restrained-segmented joint pipes have been widely used to replace old unrestrained pipes as part of seismic disaster-prevention measures. Mechanical features of old- and new-type CIP and DCIP joints are illustrated in Fig. 23.3. Since CIP and DCIP with old-type joints do not have any restraining mechanism for axial force, they are vulnerable to seismic displacement. As a result, these unrestrained pipes with old-type joints are replaced by new-type DCIP, which is equipped with a locking mechanism and thus cannot be easily pulled out



23.3 Old and new types of CIP and DCIP joints.

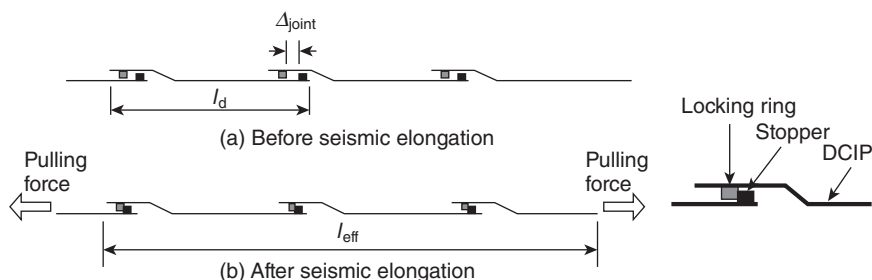
Table 23.2 Seismic performance of old-type and new-type joints

Type	Resistance of joint (mm)		
	Major	Moderate	Major
Mean			
Old joint	5	10	15
New joint	30	45	60
Coefficient of variation			
Old joint	0.2	0.25	0.3
New joint	0.05	0.05	0.1

Source: Wada *et al.* (2010).

even in the case of large displacement induced by an earthquake. The seismic performances of these joints are compared in Table 23.2 in terms of acceptable displacement in the axial direction.

Figure 23.4 shows a restraining mechanism of the new-type DCIP joint. If the ground displacement is larger than Δ_{joint} that can be accommodated by a single joint, the other two joints neighboring the first one can move together to absorb the remaining ground displacement over the interval of l_{eff} as shown in Fig. 23.4b.



23.4 Restraining mechanism of the new-type DCIP joint (Wada *et al.*, 2010).

23.2.2 Seismic performance of pipelines

There are two useful seismic design guidelines of segmented pipelines, which are issued from the American Lifeline Alliance (ALA, 2005) and the Japan Water Works Association (JWWA, 1997). These current seismic design methods are reviewed and compared below and on page 635. These procedures are suitable for existing segmented pipelines. On page 635, a design procedure for new-type segmented pipelines is discussed. Subsequently, on page 638, a general analysis method for evaluating seismic fragility of segmented pipelines is summarized.

Seismic performance design procedure of existing segmented pipelines in the USA

The ALA (2005) has developed a design procedure for segmented joint pipes for earthquake loading. An unrestrained and segmented pipe has joints with low strength and stiffness relative to the pipe barrel. An example is a DCIP with push-on, bell-and-spigot gasketed joints as shown in Fig. 23.3(b). The axial displacement that is accommodated by the joint can be taken as follows (El Hamadi and O'Rourke, 1990):

$$\Delta_{\text{joint}} = 7l_d \epsilon_{\text{soil}} \quad [23.1]$$

where Δ_{joint} is the capacity against the joint pull-out at the time of the earthquake, l_d is the length of the pipe segment as shown in Fig. 23.4 (a), and ϵ_{soil} is the ground strain in the axial direction of the pipe. For design purposes, the ALA guidelines recommend the use of seven times the average joint movement, based on El Hamadi and O'Rourke (1990). The ALA seismic design method of the joint suggests that the pull-out movement

should not exceed the capacity of the joint, so the seismic safety is assessed as follows:

$$\Delta_{\text{joint}} \geq \Delta_{\text{design}} \quad [23.2]$$

where Δ_{joint} is the capacity against joint pull-out at the time of the earthquake, and Δ_{design} is the design displacement which is given by

$$\Delta_{\text{design}} = \Delta_{\text{seismic}} + \Delta_{\text{operational}} + 6.35 \text{ mm} \quad [23.3]$$

in which Δ_{seismic} and $\Delta_{\text{operational}}$ are predicted seismic joint displacement and daily operational joint displacement, respectively, and 6.35 mm is factor used to provide a margin measure of safety.

Furthermore, the ALA prepared a seismic design procedure for restrained and segmented pipes, which can absorb a large earthquake ground displacement and have a locking ring to prevent pull-out failure from the joint. When one locking ring is insufficient to resist the applied force, the neighboring joints can support the residual force. If n joints are necessary to resist the applied force, the total axial tensile force F_{stop} is given by

$$F_{\text{stop}} = \pi D \tau_G l_d (n + 1) \quad [23.4]$$

where D is the pipe diameter, τ_G is the shear stress acting between the pipe surface and surrounding soil, and n is the number of joints necessary to resist the applied force. The ALA recommends that n should have a value of 3. So for restrained and segmented pipes, the seismic safety is assessed as follows:

$$R_y \geq F_{\text{stop}} \quad [23.5]$$

where R_y is the critical strength for the pull-out force.

Seismic performance design procedure of existing segmented pipelines in Japan

The JWWA (1997) published the *Seismic Guidelines for Water Pipelines*, in which it developed a design procedure for unrestrained and segmented joint pipes, that is different from the one suggested by the ALA. Note that the JWWA procedure is applicable to unrestrained and segmented pipes only, whereas the ALA method can be used for both unrestrained and restrained pipes.

In a very strong earthquake, which is defined as a Level 2 earthquake in Japan, or a maximum considered earthquake (MCE), the following analytical procedure can be used to estimate the joint displacement Δu_j :

$$\Delta u_j = \lambda \cdot \alpha_1 \cdot \Delta u_G \quad [23.6]$$

where

$$\alpha_1 = \frac{1}{1 + (\gamma_1/\beta_1)^2}, \quad \lambda = \frac{|\cosh \beta_1 - \cos \gamma_1|}{\frac{1}{2}\beta_1 \sinh \beta_1} \quad [23.7]$$

$$\beta_1 = \sqrt{\frac{K_{g1}}{EA}} l_d, \quad \gamma_1 = \frac{2\pi l_d}{L'} \quad [23.8]$$

in which E , A , and K_{g1} are the elastic rigidity, cross-section area, and spring modulus of a segmented pipe, respectively; L' is the apparent wavelength along the pipe; and $L' = \sqrt{2}L$ for the incident wavelength L . Δu_G is the relative ground displacement which is given by

$$\Delta u_G = \frac{u_G}{\sqrt{2}} \left[1 - \cos \left(\frac{2\pi}{L'} l_d \right) \right] \quad [23.9]$$

in which U_G is the horizontal displacement at the ground surface and can be calculated by the following equation:

$$u_G = \frac{2}{\pi^2} S_v(T_G) \cdot T_G \quad [23.10]$$

where T_G is the characteristic site period of the soil deposit in which a buried pipeline is installed, and S_v is the spectral response velocity at the characteristic site period, respectively.

In the seismic design procedure for unrestrained DCIP, the seismic displacement of a single DCIP joint is assessed by

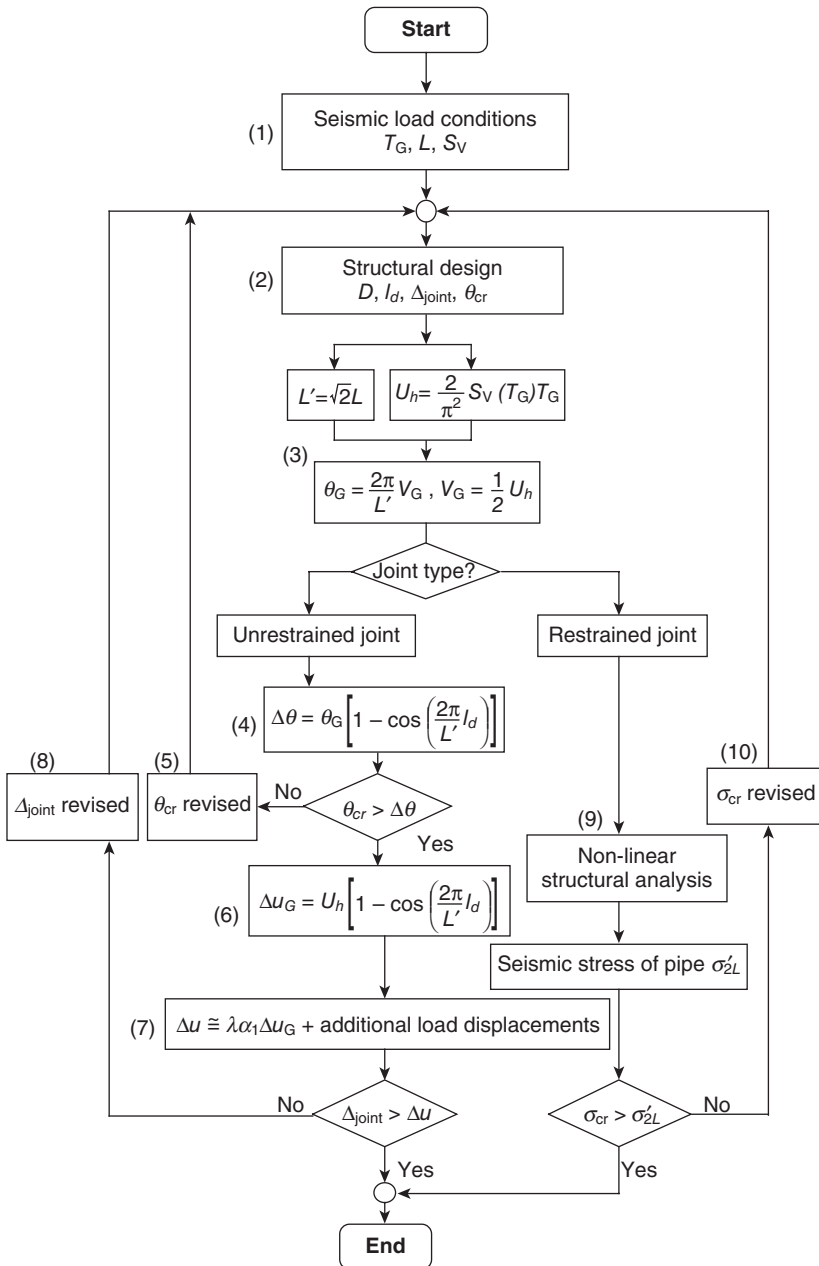
$$\Delta_{\text{joint}} \geq \Delta u_J + \text{additional load displacements} \quad [23.11]$$

in which Δ_{joint} is the critical joint displacement.

Figure 23.5 shows a flowchart of JWWA's seismic design procedure, in which the calculation steps for the unrestrained joints are described in detail, but those for the restrained joints request non-linear structural analysis to obtain the seismic stress of pipe joints without any detailed instruction on its calculation methods.

In this design procedure, the following steps must be executed:

1. Assume the seismic load conditions at the site; T_G , L , S_v .
2. Assume the structural design conditions; D , l_d , Δ_{joint} , θ_{cr} .
3. Calculate the apparent wavelength L' , the horizontal displacement u_G in Equation (23.10), the transverse displacement v_G , and its corresponding angle θ_G .
4. Calculate the bending angle $\Delta\theta$ at an unrestrained joint.



23.5 Flow chart of a seismic design procedure (JWWA, 1997).

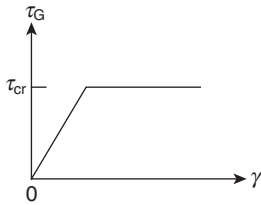
5. If the bending angle is more than the critical angle, a new DCIP is selected to obtain a greater critical bending angle θ_{cr} than the calculated one.
6. If the bending angle is less than the critical angle, then calculate the relative ground displacement Δu_G in Equation (23.9).
7. Estimate the seismic displacement Δu of a single DCIP joint given in Equations (23.6), (23.7), (23.8) and (23.11).
If the seismic displacement is less than the critical joint displacement Δ_{joint} , the seismic design of the unrestrained joint pipeline is completed.
8. If the seismic displacement is greater than the critical joint displacement, a new DCIP is selected to obtain a greater critical displacement than the calculated one.
9. In case the pipe joint is restrained, non-linear structural analysis is requested to obtain the seismic stress σ'_{2L} at the joint.
10. If the seismic stress is greater than the critical stress σ_{cr} , a new DCIP is selected to obtain a greater critical stress than the calculated one. If the seismic stress is less than the critical stress, the seismic design of the restrained joint pipeline is completed.

Note that the JWWA method is not based on observed seismic damage data of restrained and segmented joint pipes. Thus, it cannot be used as a seismic safety-assessment procedure for restrained and segmented joint pipes in Japan. To solve this difficulty, a newly developed seismic assessment procedure is proposed for restrained and segmented joint pipes, which is detailed in the following subsection.

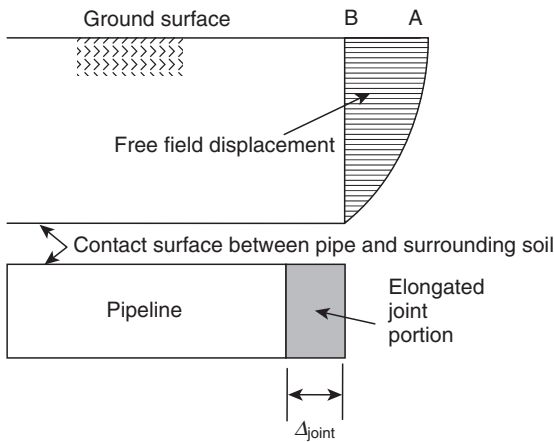
Seismic performance analysis of the new segmented joint

From the past earthquakes in Japan, because there are no extensive damage data for restrained and segmented pipes, seismic performance analysis needs to rely on an engineering mechanics-based theoretical approach. Owing to the superior seismic performance of the new segmented joint, water pipeline engineers often adopt this type of joint when the retrofitting action is planned. In this subsection, the ultimate strength of the new-type joint of DCIP is investigated by the theoretical approach to obtain the seismic capacity of this joint for the Level 2 ground motion (MCE) in Japan.

The axial force S that is necessary to pull out from the joint can be calculated from the shear stress, which is sinusoidally distributed along the pipe axis. If the shear stress τ_G (i.e. shear stress acting between the pipe surface and the surrounding ground) due to ground strain ϵ_G is less than the critical shear stress τ_{cr} , as shown in Fig. 23.6, slippage is initiated at the pipe surface, and the maximum pull-out force is calculated from the shear



23.6 Shear stress and strain acting between the pipe surface and the surrounding ground.



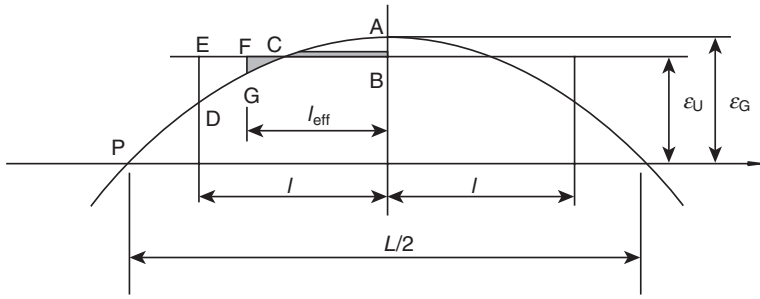
23.7 Pipe elongation induced by the surrounding soil.

stress τ_G . When the shear stress τ_G is greater than the critical shear stress, on the other hand, the maximum pull-out force can be computed from the following critical shear stress:

$$S = \begin{cases} \pi D \cdot l_{\text{eff}} \cdot \tau_G & \text{for } \tau_G < \tau_{\text{cr}} \\ \pi D \cdot l_{\text{eff}} \cdot \tau_{\text{cr}} & \text{for } \tau_G \geq \tau_{\text{cr}} \end{cases} \quad [23.12]$$

where l_{eff} is the effective length loaded by the shear stress acting on the pipe surface.

When a seismic excitation is applied to the ground surface and pipeline as shown in Fig. 23.7, the ground is deformed. Note that the deformation profile in the ground is not uniform in the vicinity of the pipeline because of the difference between the pipe stiffness and the soil stiffness. The pipe elongation is forced by the shear stress acting on the pipe surface which is transmitted by the shear deformation of the surrounding soil.



23.8 Ground and structural strains along the pipe-stretching axis (Wada *et al.*, 2010).

Figure 23.8 shows the shear strain distribution along the pipe-stretching axis when the ground strain is larger than the critical strain ε_U that is necessary to initiate the pull-out deformation of the segmented pipe joints. If a group of segmented joints can be smoothly pulled out without slippage at the pipe surface, the pipes along the section BCFE are necessary to absorb the excessive strain between the ground and structural strains. If this group of segmented joints makes a slippage before the pull-out deformation of the joints along the section CFE, however, the structural strain distribution is decreased from BCFE (l) to BCF (l_{eff}) because the excess strain of ABC (shaded area) is balanced by the additional strain of CFG. This effective length, l_{eff} , is estimated with the connection factor ρ as follows:

$$l_{\text{eff}} \equiv l_d + \rho(l - l_d) \quad [23.13]$$

in which l and l_d are the maximum length of BCFE in Fig. 23.8 and the unit pipe length in Fig. 23.4, respectively. The connection factor ρ is derived from the following relationship (Imai and Koike, 2010):

$$\rho = \frac{\frac{L}{2\pi} \alpha^* \left(\frac{q_{\text{eff}}}{q^*} - 1 \right) - l_d}{l - l_d} \quad [23.14]$$

The relative displacement between the ground and structural strains Δ can be formulated as:

- if a relative displacement is absorbed by the pull-out-deformation and slippage effect

$$\Delta = l_{\text{eff}} \varepsilon_G + (1 - q_{\text{eff}} \alpha_0) u_G \quad [23.15]$$

- if a relative displacement is absorbed by the slippage effect over the assumed continuous pipe

$$\Delta = (1 - q^* \alpha_0) u_G \quad [23.16]$$

where ε_G is the ground strain, α_0 is the conversion factor of the pipe strain from the ground strain without slippage, α^* is the conversion factor of the segmented pipe by considering it as a continuous pipe, and q_{eff} is the slippage factor for the segmented pipe. In particular, q^* is computed as follows by assuming that it is a continuous pipe:

$$q^* = \sin \xi \cdot \left(1 + \frac{\pi^2}{8} - \frac{\xi^2}{2} \right) - \xi \cos \xi \quad [23.17]$$

where

$$\xi = \arcsin \left(\frac{\tau_{\text{cr}}}{\tau_G} \right), \quad \tau_G = \frac{2}{L} \frac{EA}{D} \alpha_0 \varepsilon_G \quad [23.18]$$

As a simplified assumption, the following formula can be used as a conservative estimate:

$$S = \pi D \cdot l_{\text{eff}} \cdot \tau_{\text{cr}} \quad [23.19]$$

The resisting force for pull-out, which is produced by the compression force at the contact surface between the locking ring and the stopper shown in Fig. 23.3, is given by the following equation:

$$R_Y = \pi D \cdot r \cdot \sigma_{\text{cr}} \quad [23.20]$$

where σ_{cr} is the critical axial stress to initiate pull-out failure from the pipe joint, and r is the height of the locking ring.

The pull-out failure of the new DCIP is initiated when the joint displacement $\Delta u_{G,\text{eff}}$ exceeds the critical pull-out displacement Δ_{joint} . Therefore, the pull-out failure can be assessed using the following criteria:

$$\begin{aligned} \Delta_{\text{joint}} < \Delta u_{G,\text{eff}} & \text{ pull-out failure occurs} \\ \Delta_{\text{joint}} > \Delta u_{G,\text{eff}} & \text{ pull-out failure does not occur} \end{aligned} \quad [23.21]$$

where the joint displacement is given by:

$$\Delta u_{G,\text{eff}} \cong l_{\text{eff}} \cdot \varepsilon_G \quad [23.22]$$

Fragility curves for segmented pipelines

The pipe damage severity due to an earthquake can be measured in terms of the ratio of pipe damage numbers per kilometer. There are many observation data on the pipe damage ratio from the past earthquakes in Japan. From these site investigation data, fragility curves for segmented pipelines

Table 23.3 Seismic spectral response velocity parameters

Seismic response (mm)		
Type	Mean	COV
EQ1	8	0.3
EQ2	37	0.4

Source: Wada *et al.* (2010).

were determined by obtaining the pipe damage ratio for a given seismic load level.

In this assessment, T_G (i.e. natural period of the ground surface) is simply assumed to be 1.0 second as a typical value over the entire area of the network system. The seismic responses of buried pipe joints for two types of seismic load are summarized in Table 23.3, in which the mean value and coefficient of variation (COV) of the axial displacement at the pipe joint are calculated from Equation (23.22).

The potential damage points are identified for all mechanical joints at every unit length l_d , and so the number of potential damage points is given by $2l_{eff}/l_d$. Therefore, the damage rate, which is defined as the number of damage points per kilometer, is estimated by:

$$v = E[\text{number of damage points per wavelength}] \cdot \frac{1000}{L} \quad [23.23]$$

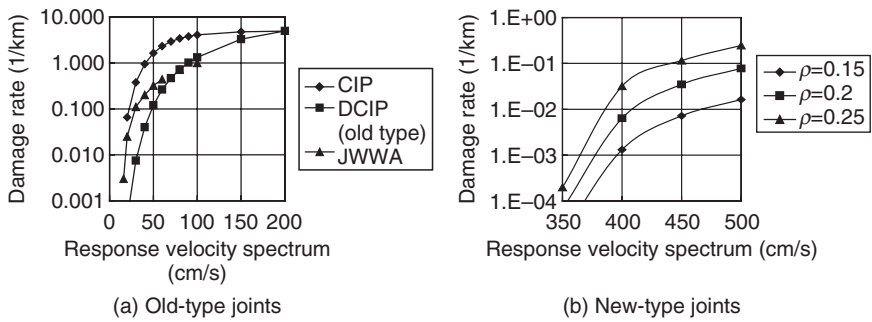
$$= \frac{2l_{eff}}{l_d} \cdot P[R(C_s, T_p) < S|EQ] \cdot \frac{1000}{L}$$

From Equation (23.23), the structural damage rates for CIP and DCIP (old- and new-type joints) are estimated for various response velocities at the ground surface, as shown in Fig. 23.9. In particular, Fig. 23.9(a) compares the structural damage rate for old-type joints of CIP and DCIP; that for the JWWA formula, which is given by an empirical curve based on the damage data of water pipelines in the 1995 Kobe earthquake (Ishihara and Yoshimine, 1995; Isoyama *et al.*, 1998), is as follows:

$$v_{JWWA} = C_p C_d C_g C_l v_{JWWA}^0(S_v) \quad [23.24]$$

where C_p , C_d , C_g , C_l and $v_{JWWA}^0(S_v)$ are the coefficients of pipe joint type, diameter, geotechnical condition, liquefaction hazard, and standard damage rate for the ground response velocity S_v .

The damage rate of the JWWA formula is between the damage rates of CIP and DCIP (old-type joint), which suggests the applicability of the proposed method. Fig. 23.9(b), on the other hand, shows the damage rates for



23.9 Damage rates for CIP and DCIP (Wada *et al.*, 2010).

DCIP (new-type joint) with three different connection factors ρ . The ρ is a factor to control the number of mechanically locked joints for the effective length l_{eff} along the seismic wave propagation. This damage rate for DCIP (new-type joint) suggests that the pull out joint failure can initiate for a large response velocity more than 350 cm/s, however, which is not possible to be produced by the Level 2 ground motion. This is the reason why any damage examples of DCIP (new-type joint) were not observed in the past earthquakes.

23.2.3 Effects of deterioration

The mechanical joints of the old segmented pipelines may be damaged by various reasons, such as ground shaking by traffic, uneven settlement of the alluvial ground, corrosion cracks in the corroded soil, and third party accident. such pipe damage is often reported as deterioration. The deterioration process of pipe joints under traffic load vibration is assumed to be modeled by the following deterioration factor:

$$\psi(t) = 1 - d_1 \left(\frac{t}{D_T} \right)^{d_2} \quad [23.25]$$

where d_1 and d_2 are parameters describing the deterioration behavior (typical values of d_1 and d_2 for new and old joints are shown in Table 23.4), and

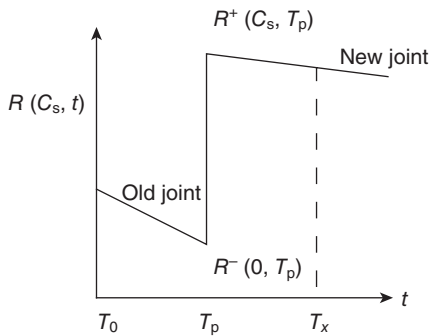
$$\begin{aligned} D_T &= T_p - T_0 \text{ before retrofitting} \\ D_T &= T_D - T_p \text{ after retrofitting} \end{aligned} \quad [23.26]$$

in which T_D is the service period of the pipeline system and T_p is the time at which the retrofitting is conducted as shown in Fig. 23.10. These two deteriorating factors for pipe joints as shown in Table 23.3 are derived from the corrosion rates of buried water pipes (Imai and Koike, 2009).

Table 23.4 Deterioration factors for pipe joints

Type	Deterioration factors	
	d_1	d_2
Old joint	0.5	0.25
New joint	0.1	0.05

Source: Imai and Koike (2009).



23.10 Schematic profile of the joint deformation capacity of deteriorating pipes.

In this assessment, the joint displacement resistance in the pull-off direction represents the residual strength R . This value can be estimated based on the deterioration factor (i.e. Equation (23.25)) as follows:

$$\begin{aligned}
 R^-(0, T_p) &= \psi(T_p - T_0) \cdot R(0, T_0) \\
 R^+(C_s, T_p) &= R_Y \\
 R(C_s, T_x) &= \psi(T_x - T_p) \cdot R^+(C_s, T_p)
 \end{aligned}
 \tag{23.27}$$

in which $R^-(C_s, T_p)$, $R^+(C_s, T_p)$, and $R(C_s, T_x)$ are the joint deformation capacity before retrofitting, the capacity after retrofitting, and the capacity at time T_x after retrofitting, respectively. Figure 23.10 shows the schematic profile of the deteriorating process of the segmented pipe joint before and after retrofitting with new-type DCIP.

23.3 Seismic risk assessment

Many existing parts of the infrastructure deteriorate because of external environmental conditions and other factors, and are vulnerable to various

natural hazards, including earthquakes. Many structures constructed prior to 1980 in Japan were designed for a particular seismic load level, i.e. Level 1 ground motion caused by the maximum operational earthquake (MOE). After the 1995 Kobe Earthquake, revised guidelines for defining Level 1 and Level 2 (MCE) ground motions as the designed earthquake load were introduced. This means that many old infrastructure designs fell below the acceptable limits according to the current seismic design procedures and practices. Thus, these structures are deemed vulnerable to strong earthquakes.

23.3.1 Definitions of seismic performance

In this section, the definition of seismic performance of the lifeline networks in Japan is described, in which seismic performance for the important network systems should be selected to fulfill the short restoration/recovery requirement for the severest seismic load in the past earthquakes. The seismic performance for the ordinary network systems, on the other hand, must meet the minimum requirement for saving human lives. In addition, for the frequent earthquakes during the service period, the seismic performance for both systems also needs to be operated without any water supply disruption.

The seismic performance of the lifeline network system can be specified in relation to the limit states of the system, such as the serviceability limit state, repairable limit state, and ultimate limit state for the corresponding seismic disasters. Table 23.5 shows typical definitions of the required seismic performance of a lifeline system. In these definitions, both the seismic-performance damage mode and the component damage mode are also defined quantitatively for seismic disasters of Level 1 (denoted by EQ_1) and Level 2 (denoted by EQ_2) ground motions, respectively. The most important facilities and transmission pipelines are required to have higher safety in seismic performance 2 than in seismic performance 3, because those structural components are always necessary for immediate recovery. The distribution pipelines and their facilities should have the capacity to recover from seismic performance 3 in due course.

In Table 23.5, the limit state function of Z for the component damage mode specifies the critical condition that the structural damage mode occurs when a seismic load exceeds the critical value. The limit state function of D for the performance damage mode provides the critical condition that the functional damage mode occurs in terms of the seismic load level. Once these limit state functions are defined, the probability of exceeding the limit state for the seismic performances can be formulated and developed in terms of seismic design information.

Table 23.5 Definitions of seismic performance and failure modes

Seismic performance	Definition	Performance damage mode	Component damage mode
1	The system performance can be maintained without any disruption for the Level 1 earthquake ground motion (EQ_1), if the system is slightly damaged or undamaged.	Minor $D_{EQ_1}^{minor}$	Minor $Z_{EQ_1}^{minor}$
2	The system performance can be restored after quick repair for the Level 2 earthquake ground motion (EQ_2), if the system is not significantly damaged.	Moderate $D_{EQ_2}^{moderate}$	Moderate $Z_{EQ_2}^{moderate}$
3	The system performance can be restored after recovering from disruption for the Level 2 earthquake ground motion (EQ_2), if the system is not completely damaged.	Major $D_{EQ_2}^{major}$	Major $Z_{EQ_2}^{major}$

Source: Imai and Koike (2009).

23.3.2 Seismic hazard surrounding the lifeline system

Generally the lifeline system is surrounded by many potential earthquakes which might be triggered by a large active fault located far from the system, or by a comparatively short active fault located in the vicinity of the system. There are two extreme approaches for seismic risk assessment with the hazard conditions of these various faults. One approach is that the most significant earthquake should be selected, and the others neglected. The second approach is that all the active faults should be collected for the seismic assessment. If there are very many active faults around the system, the second approach is not practical. The first approach, on the other hand, cannot erase the seismic risk due to the neglected faults (denoted by the residual risk).

When the most significant earthquake is selected as a seismic design load, a scenario earthquake approach may be adopted to assess whether the designed system survives without significant disruption and damage under the considered scenario. In this design procedure, the residual risk, which might not be accounted for when adopting one particular situation/case as the scenario earthquake, has to be taken into consideration. One solution to this difficulty is to adopt several scenario earthquakes as possible future earthquakes. If there are insufficient historical earthquake data in the

nearby region, the most probable ground motion must be predicted in a probabilistic manner.

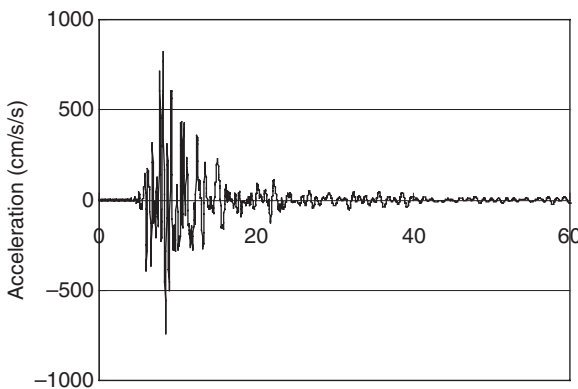
Since an active fault presents immediate seismic threat in the near future, a scenario earthquake approach is adopted in the present study. Figure 23.11 shows a sample time history of simulated accelerogram on the surface ground in the vicinity of the fault center (Yoshimi *et al.*, 2005).

In cases where there is no information about the active fault, for practical design requirements, the worst earthquake model can be assumed. One practical method was proposed by Takemura *et al.* (1999), who suggested adopting an envelope curve of the response spectral velocities of historical earthquake records with at least $M_w = 6.5$ – 6.7 , as illustrated in Fig. 23.12. Figure 23.13 shows a sample simulated accelerogram (JARL, 1992); its response spectral density is compatible with that of the worst earthquake model. However, this conventional approach cannot always guarantee that this simulated ground motion of Fig. 23.13 is a comprehensive and probabilistic model of the worst earthquake ground motion. More rigorous modeling approaches have been recently proposed by Goda and Atkinson (2010); this method utilizes conventional ground motion prediction equations with extended capability to simulate spatially distributed seismic excitations, which are particularly suitable for lifeline network models.

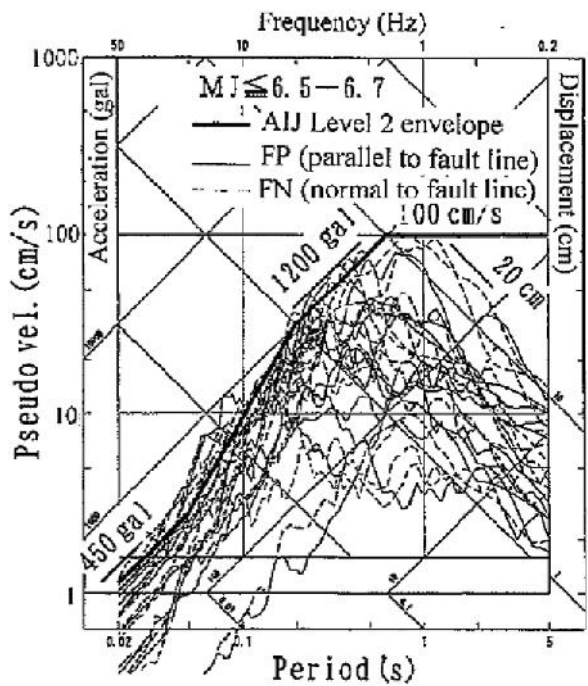
If the seismic performance damage mode of a pipeline D_{EQ}^{mode} as well as the component damage mode Z_{EQ}^{mode} is given, the probability of failure is defined as the combination of all potential active faults in the following form (McGuire, 2004):

$$p_f = \sum_{k=1}^N \bigcup_{s \in S_k(M, R, G)} P[D_{EQ}^{mode} | Z_{EQ}^{mode}] P[Z_{EQ}^{mode} | S_k(M, R, G) = s] \quad [23.28]$$

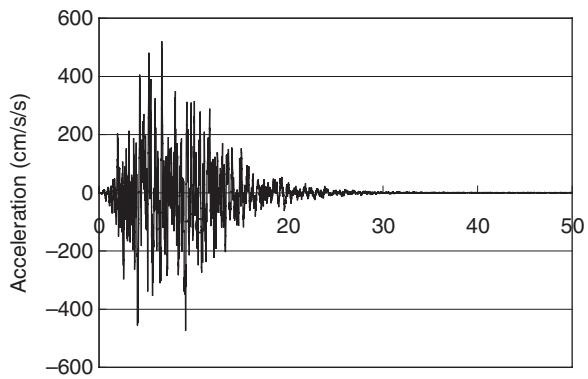
$$P[S_k(M, R, G) = s | EQ_k] P[EQ_k]$$



23.11 Scenario earthquake accelerogram generated from an active fault.



23.12 Velocity response spectra of the worst ground motion (Takemura *et al.*, 1999).



23.13 Simulated accelerogram of the worst ground motion.

where the system is surrounded by N active faults, and the seismic intensity from each active fault (k) is predicted as $S_k(M, R, G)$, where M , R , and G are magnitude, focal distance, and site ground condition, respectively. If earthquakes are ordered according to their acceleration from the largest to

the smallest based on the attenuation formula, equation (23.28) can be rewritten as

$$\begin{aligned}
 p_f = & \bigcup_{s \in S_k(M, R, G)} P[D_{EQ}^{\text{mode}} | Z_{EQ}^{\text{mode}}] P[Z_{EQ}^{\text{mode}} | S_1(M, R, G) = s] \\
 & P[S_1(M, R, G) = s | EQ_1] P[EQ_1] \\
 & + \sum_{k=2}^n \bigcup_{s \in S_k(M, R, G)} P[D_{EQ}^{\text{mode}} | Z_{EQ}^{\text{mode}}] P[Z_{EQ}^{\text{mode}} | S_k(M, R, G) = s] \\
 & P[S_k(M, R, G) = s | EQ_k] P[EQ_k] \\
 & + \sum_{k=n+1}^N \bigcup_{s \in S_k(M, R, G)} P[D_{EQ}^{\text{mode}} | Z_{EQ}^{\text{mode}}] P[Z_{EQ}^{\text{mode}} | S_k(M, R, G) = s] \\
 & P[S_k(M, R, G) = s | EQ_k] P[EQ_k]
 \end{aligned} \tag{23.29}$$

In this equation, the first term corresponds to the critical scenario earthquake, and the second term is a group of earthquakes that are of moderately severe seismic intensity generated from known active faults; the last term denotes a group of earthquakes that will be less seismically severe.

Currently, the majority of research groups have estimated seismic risk from all active faults. But in actual design schemes, seismic assessment for all active faults is impractical. A simple approach would be preferable to the current approach (Equation (23.29)). The worst spectral method (JARL, 1992), dealing with many earthquakes caused by unknown faults, is adopted in the present study as a conservative estimate for dealing with this issue. Since each seismic intensity $S_k(M, R, G)$ is less than the worst seismic intensity S_{worst} shown as

$$S_k(M, R, G) \leq S_{\text{worst}} \tag{23.30}$$

the probability of failure in Equation (23.29) can be replaced by

$$\begin{aligned}
 p_f = & \bigcup_{s \in S_k(M, R, G)} P[D_{EQ}^{\text{mode}} | Z_{EQ}^{\text{mode}}] P[Z_{EQ}^{\text{mode}} | S_1(M, R, G) = s] \\
 & P[S_1(M, R, G) = s | EQ_1] P[EQ_1] \\
 & + \sum_{k=2}^n \bigcup_{s \in S_k(M, R, G)} P[D_{EQ}^{\text{mode}} | Z_{EQ}^{\text{mode}}] P[Z_{EQ}^{\text{mode}} | S_k(M, R, G) = s] \\
 & P[S_k(M, R, G) = s | EQ_k] P[EQ_k] \\
 & + \bigcup_{s \in S_k(M, R, G)} P[D_{EQ}^{\text{mode}} | Z_{EQ}^{\text{mode}}] P[Z_{EQ}^{\text{mode}} | S_k(M, R, G) = s] \\
 & P[S_{\text{worst}} = s | EQ_{\text{worst}}] P[EQ_{\text{worst}}]
 \end{aligned} \tag{23.31}$$

where EQ_1 , EQ_k , and EQ_{worst} are the critical scenario earthquake, the k th earthquake belonging to the 2nd group, and the worst earthquake, respectively.

The probability of the earthquake from an active fault can be predicted using the Brownian passage time (BPT; Ellsworth *et al.*, 1999) distribution model as

$$P[EQ] = \int_{T_p}^{T_p+T_D} f(t; T_R, \kappa) dt \left/ \left[1 - \int_0^{T_p} f(t; T_R, \kappa) dt \right] \right. \quad [23.32]$$

in which T_R , T_p , T_D , f , and κ are the return period of the earthquake, the non-occurrence period, the design period, probability density function of the BPT model, and the coefficient of variation of the return period, respectively. The probability density function of the BPT model is given by:

$$f(t; \mu, \alpha) = \sqrt{\frac{\mu}{2\pi\kappa^2 t^3}} \exp \left[-\frac{(t-\mu)^2}{2\mu\kappa^2 t} \right] \quad [23.33]$$

In particular, with the return period and non-occurrence period of the worst earthquake, it might be assumed that $T_R = 475$ years and T_p = middle point of the return period as the average non-occurrence period of many earthquakes.

23.3.3 Structural failure

Let us define the damage modes for structural components in the following way; R , D , L , S , C_s , T_p are the residual strength for each damage mode, dead load, live load, seismic load, retrofitting investment, and present time, respectively:

$$\begin{aligned} Z_{EQ}^{\text{minor}}(C_s, T_p) &= R^{\text{minor}}(C_s, T_p) - (D + L) - S \cdot 1_{EQ} < 0 \\ Z_{EQ}^{\text{moderate}}(C_s, T_p) &= R^{\text{moderate}}(C_s, T_p) - (D + L) - S \cdot 1_{EQ} < 0 \\ Z_{EQ}^{\text{major}}(C_s, T_p) &= R^{\text{major}}(C_s, T_p) - (D + L) - S \cdot 1_{EQ} < 0 \end{aligned} \quad [23.34]$$

where,

$$1_{EQ} = \begin{cases} 1 : \text{an earthquake occurs at } t_{EQ} \\ 0 : \text{an earthquake does not occur at } t_{EQ} \end{cases} \quad [23.35]$$

The expected value of 1_{EQ} equals the probability of an earthquake occurring in the residual service period ($T_D - T_p$), and it is given by the following equation using the return period of the possible future earthquake:

$$E[1_{EQ}] = P[EQ] \quad [23.36]$$

A pipeline is characterized as a long, stretched structure, in which potential cracks that initiate a leakage failure due to seismic load effect are randomly located along the pipeline. Assuming that damage in a pipeline follows a Poisson process, the probability of pipe failure in the i th link is given by (Imai and Koike, 2009):

$$\begin{aligned} P[Z_{\text{link}}^{\text{minor}}(l_i)|EQ_1] &= 1 - \exp\left[-\int_0^{l_i} v_{\text{minor}}(x)dx\right] \\ P[Z_{\text{link}}^{\text{moderate}}(l_i)|EQ_2] &= 1 - \exp\left[-\int_0^{l_i} v_{\text{moderate}}(x)dx\right] \\ P[Z_{\text{link}}^{\text{major}}(l_i)|EQ_2] &= 1 - \exp\left[-\int_0^{l_i} v_{\text{major}}(x)dx\right] \end{aligned} \quad [23.37]$$

where the damage rate per unit pipe length at point x is given by:

$$v_{\text{mz}}(x) = v_0 P[Z_{\text{EQ}}^{\text{mz}}|EQ] \quad [23.38]$$

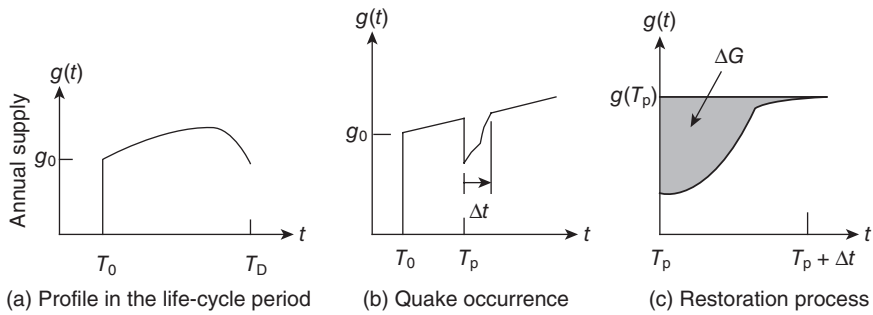
where mz indicates the structural damage mode, i.e. minor, moderate, and major, and v_0 denotes the number of pipe joints per unit pipe length.

Most structural facilities, including river-crossing bridges, pumping facilities, storage tanks, and control-valve equipment, are located along the link, while other important facilities, such as water-purification plants, are represented as individual nodes. Because these structural facilities are on the ground surface, a response-analysis approach can be applied to estimate the maximum seismic loads, which should be compared with the structural resistance to assess the probability of failure for different seismic loads probabilistically by using suitable fragility curves.

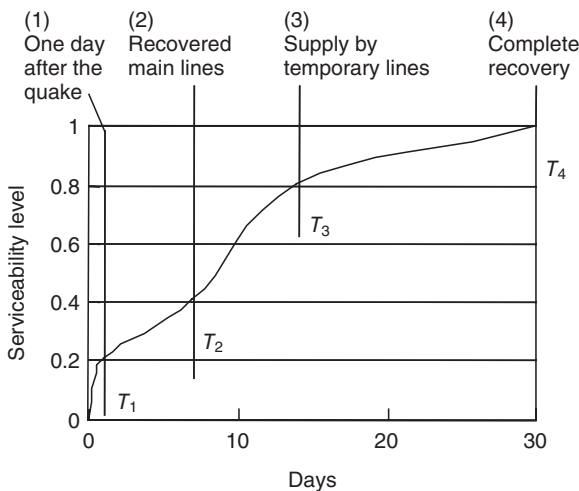
23.3.4 Performance failure

Performance failure can be estimated as a loss of water supply immediately after an earthquake and/or as a restoration period in the recovery process several weeks after the earthquake. Figure 23.14(a) shows a temporal profile of water serviceability in the life-cycle stage between the initial time point T_0 and the terminal time point T_D . Figure 23.14(b) is a conceptual illustration of a short-term disruption in the lifeline service during the restoration period after seismic damage has occurred (Imai and Koike, 2009). When an earthquake occurs at T_p , some restoration time Δt is necessary, as indicated in Fig. 23.14(b). The detailed restoration process is presented in Fig. 23.14(c), in which ΔG is the loss of serviceability caused by seismic damage.

A detailed restoration process is shown in Fig. 23.15, where the process is divided by the four time points T_1 , T_2 , T_3 , and T_4 . The time point T_1 is the day after the earthquake, when the water serviceability level is provided to



23.14 Schematic profile of water serviceability in the life-cycle stage and malfunction during the restoration process after an earthquake.



23.15 Schematic profile of the restoration process after an earthquake.

undamaged areas only; such undamaged areas can be identified by a site survey of damage points along all the pipelines. The time point T_2 is when the transmission mains have recovered. The time point T_3 is when temporary pipelines have been installed for emergency use in the damaged areas. T_4 is the completion time when all the damaged pipelines have been restored. During the time interval from the earthquake occurrence to the time point T_1 , a water company collects the damage information of the distribution networks as well as the control facilities. The head office sets up the emergency headquarter immediately after the earthquake, and they will classify the damaged and undamaged networks to control the water supply. In this stage, leakage cannot be stopped from damaged pipelines.

During the time points between T_1 and T_2 , the most important transmission pipelines and their relevant facilities are preferentially checked and repaired. If the water purification plants are operational, the water will be supplied to the nodes connected to the important demands. At the time point of T_3 , the water will be supplied to the nodes connected to the distribution networks, and the damaged pipelines are replaced by temporary pipelines. At the time point of T_4 , the whole water supply system, including all kinds of pipeline networks, control facilities, storage reservoirs, and pumping facilities, is completely recovered.

The performance level of the network system after the seismic event can be estimated by the supply damage rate. The supply damage rate immediately after a seismic event strongly depends on the structural damage rate of the water distribution network system. The water unserviceability over the distribution ($\Gamma_{\text{major}}(T_p)$) areas is quantified as follows:

$$\Gamma_{\text{major}}(T_p) = \begin{cases} 1 - \frac{n}{N} & \text{at } T_p = T_1 \\ 1 - g_T\left(\frac{T_p}{T_4}\right) & \text{at } T_2 \leq T_p \leq T_4 \end{cases} \quad [23.39]$$

where n/N is the water supply ratio and g_T is a restoration function showing the recovery ratio at time point T_p . Therefore, the supply damage rate in the damaged network system can be expressed as:

$$P\left[D_{\text{EQ}}^{\text{major}} \left| \bigcup_{i=1}^{NL} \{Z_{\text{link}}^{\text{major}}(l_i)\} \right| \right] = \Gamma_{\text{major}}(T_p) \quad [23.40]$$

where $D_{\text{EQ}}^{\text{major}}$ is the major damage state in terms of serviceability of water supply.

23.4 Seismic risk mitigation

23.4.1 Decision making for seismic mitigation

Generally, an insurance company can manage many kinds of risks on the basis of expected risk evaluations. The public sector managing a water pipeline system, however, has to take proactive measures to reduce the seismic risk of the system by seismic retrofitting activities. Communication with regard to risk is necessary among stakeholders to obtain a final acceptance of the investment in seismic retrofitting activities; to this end, the engineer must provide with an appropriate solution in monetary terms that can be understood by the stakeholders. A viable approach to establishing decision-making measures is outlined below.

The final balance of cost and income over the life-cycle period (Porter *et al.*, 2004) should be the key factor in evaluating the feasibility of a water pipeline-maintenance project. This balance, which is referred to as the value index V_0 , is the resulting value that can be produced by the project operation, and can be expressed as (Imai *et al.*, 2010):

$$V_0 = B + I - E - (C_0 + C_M) \quad [23.41]$$

in which B , I , E , C_0 , and C_M are the accumulated total amounts of social benefit, income gain, operational expense, initial cost, and maintenance cost, respectively, during the life of the project. If a seismic investment, C_S , is taken into account, the value index V_1 can be represented by

$$V_1 = B + I - E - (C_0 + C_M + C_S) = V_0 - C_S \quad [23.42]$$

It should be noted that the seismic vulnerability of the lifeline system causes not only structural damage to the network links but also functional damage to the demand nodes in terms of serviceability loss. This loss results, in turn, in the loss of social benefit, ΔB , and income gain, ΔI . The value index V_2 in this situation is given by

$$\begin{aligned} V_2 &= B + I - E - (C_0 + C_M + C_S) - (C_R + \Delta B + \Delta I) \cdot 1_{EQ}(t_{EQ}) \quad [23.43] \\ &= V_0 - [C_S + C_{EQ} \cdot 1_{EQ}(t_{EQ})] \end{aligned}$$

where C_R is the restoration cost and C_{EQ} is the seismic damage cost, which is defined as

$$C_{EQ} = C_R + \Delta B + \Delta I \quad [23.44]$$

and

$$1_{EQ}(t_{EQ}) = \begin{cases} 1 & \text{: an earthquake occurs over some time duration } t_{EQ} \\ 0 & \text{: an earthquake does not occur over some time duration } t_{EQ} \end{cases} \quad [23.45]$$

In particular, for the major damage mode, Equation (23.43) can be expressed as

$$V_2^{\text{major}} = B + I - E - (C_0 + C_M + C_S) - C_{EQ}^{\text{major}} \cdot 1_{DEQ}^{\text{major}} = V_0 - \{C_S + C_{EQ}^{\text{major}} \cdot 1_{DEQ}^{\text{major}}\} \quad [23.46]$$

in which C_{EQ}^{major} is the seismic damage cost in the major damage mode. It is noted that in the formulation of the value index of Equations (23.41) to (23.46), a discount rate is not taken into account because the public budget accounting system for water supply projects cannot manage free cash flow for its own financial investment.

23.4.2 Decision-making criteria for seismic risk mitigation

For a water pipeline system, the probability of performance failure should be less than the target value. On the other hand, the manager of the facility is often concerned about the probability of value loss being less than the target value (in addition to the chance of performance failure). The optimal solution of the seismic investment must satisfy these two design criteria:

$$p_{D_{EQ}}^{major} \leq p_{major, D_{EQ}}^{Target} \quad [23.47]$$

$$p[V_2^{major} < 0] \leq p_{major, V_2}^{Target} \quad [23.48]$$

in which

$$p[V_2^{major} < 0] \equiv P[V_{2, EQ} < 0] = P[V_0 < (C_s + C_{EQ}^{major}) | D_{EQ}^{major}] P[D_{EQ}^{major}] \quad [23.49]$$

23.4.3 Case study for a sample network system

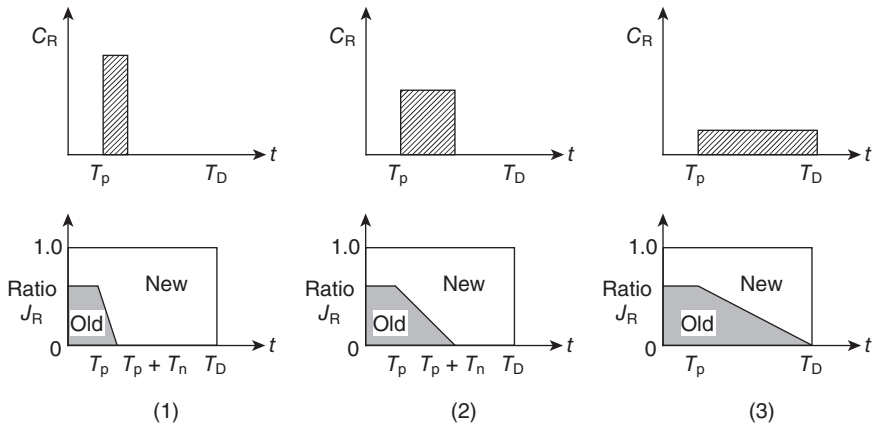
Numerical example

Numerical simulations are carried out for several alternatives to obtain the optimal seismic risk mitigation strategy; the numerical example set-up is summarized in Table 23.6 (Wada *et al.*, 2010). Let us define C_s , T_0 , T_p , T_D , T_n , and J_R as the seismic investment, the starting point of supply service, the present point, the point of service termination, the end point of retrofitting activities, and the ratio of old-type joint pipes to all pipes, respectively. To assess the effectiveness of different retrofitting strategies, three alternative investment schemes are considered. Figure 23.16 shows schematic diagrams of these investment schemes; scheme 1 is an intensive retrofitting plan, while scheme 3 is a long-term plan, and scheme 2 is an intermediate plan.

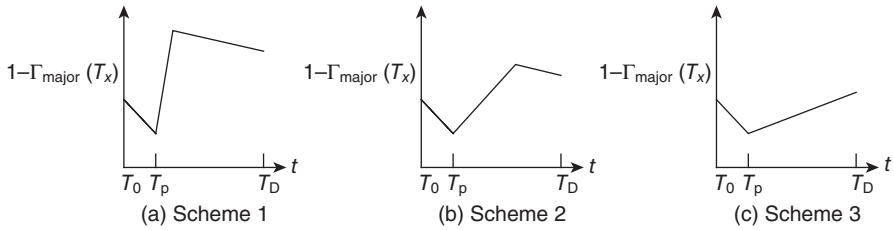
Table 23.6 Numerical example setup

Item	Symbol	Unit	Amount
Total pipe length	L	km	100
Unit length of pipe	u_L	m	5
Restoration period	Δ_t	month	0.5
Return period of EQ ₁	T_{R1}	year	50
Return period of EQ ₂	T_{R2}	year	475
Retrofitting period	T_n	year	10, 30, 70

Source: Wada *et al.* (2010).



23.16 Illustration of three seismic investment schemes.



23.17 Schematic profiles of the water-supply level for three retrofitting schemes.

The ordinate J_R in Fig. 23.16 is a ratio of the old joints for the total joints, which is defined by

$$J_R(t) = \frac{\sum_{j=1}^{NL} L_j^{\text{old}}(t)}{\sum_{j=1}^{NL} [L_j^{\text{old}}(t) + L_j^{\text{new}}(t)]} \quad [23.50]$$

where L_i^{new} , L_i^{old} , and NL are the stretching length of the new and old joints in the i th network link and the number of links in the network.

$$L_{\text{old}}(t) = \sum_{i=1}^{NL} l_i^{\text{old}}(t), \quad L_{\text{new}}(t) = \sum_{i=1}^{NL} l_i^{\text{new}}(t) \quad [23.51]$$

Figure 23.17 shows a schematic profile of the water-supply level for the three investment schemes. Scheme 1 can provide higher performance of the

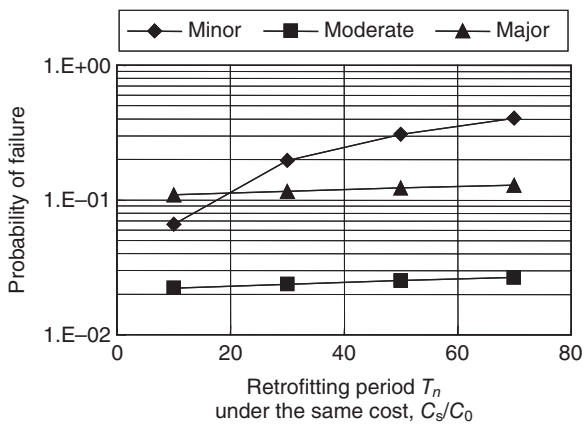
water-supply service after the retrofitting, because the deterioration effect due to old joints rapidly decreases. Scheme 3, on the other hand, cannot achieve higher performance of the water-supply service, because the deterioration effect due to old joints is maintained over the whole residual period.

It is noteworthy that the investments required for schemes 1–3 are same, i.e. scheme 1 rapidly can improve the seismic performance, but in the remaining longer period, the deterioration process for DCIP (new-type joint) will start again. On the other hand, scheme 3 can gradually improve the seismic performance during the whole service period, but potentially vulnerable old-type joints of CIP/DCIP cannot be replaced during the whole service period.

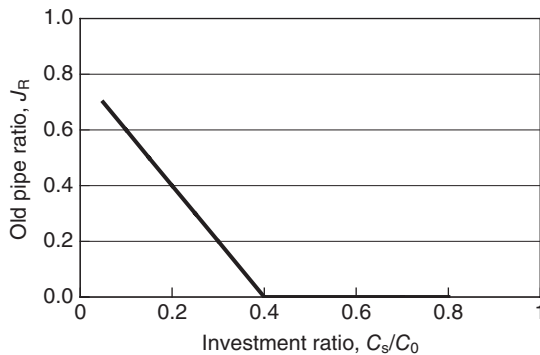
Simulation results

Figure 23.18 shows the probability of system-performance failure for various retrofitting periods under the seismic investment ratio $C_s/C_0 = 0.5$, in which C_s and C_0 are the seismic investment cost and initial cost, respectively. As shown in Fig. 23.19, all existing old pipes are replaced when C_s/C_0 exceeds 0.4. Therefore, this is the case when sufficient seismic investment has been achieved.

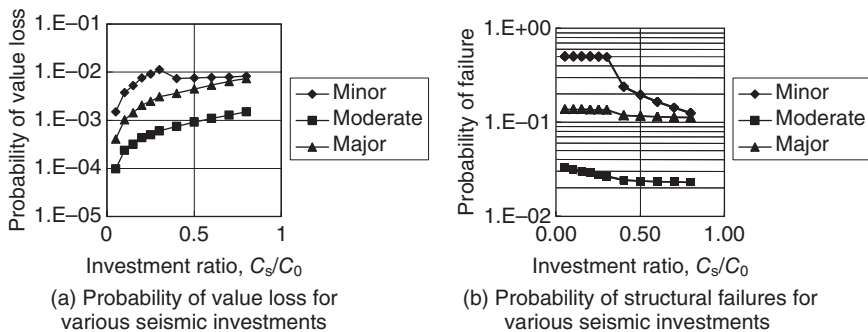
Rapid retrofitting cases indicate that the improvement in seismic performance is remarkable in the minor damage mode (Fig. 23.18). This result indicates that the intensive retrofitting plan is effective in reducing the seismic risk when sufficient investment has been made. Figure 23.20 compares the probability of value loss and that of system-performance failure



23.18 Probability of system failure for retrofitting periods under $C_s/C_0 = 0.5$.



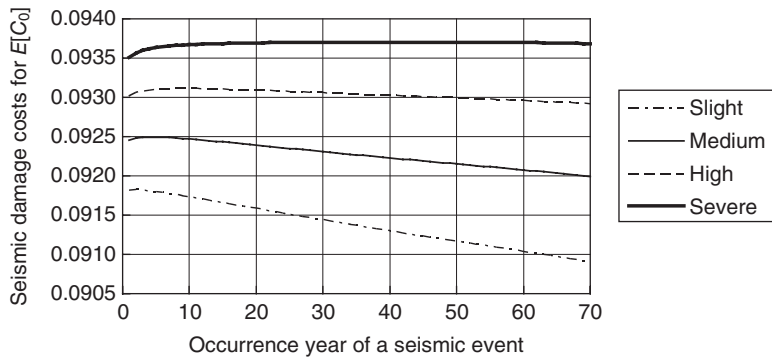
23.19 Old-pipe replacement for seismic investment (Wada *et al.*, 2010).



23.20 Probability of system failure for various seismic investments (Wada *et al.*, 2010).

for various C_s/C_0 values when the retrofitting period T_n is 30 years. The probability of value loss increases for larger C_s/C_0 , while the probability of system-performance failure are monotonically decreased. The seismic performance clearly changes at $C_s/C_0 = 0.4$. Notably, this probability in the minor-damage mode shows a sudden drop. This result indicates that the existence of old-type joint pipes affects the seismic performance of a deteriorated network system. It is important that the stakeholders can make a decision on the optimal seismic investment by comparing these probabilities for various C_s/C_0 .

Figure 23.21 indicates the effects of deterioration on seismic-retrofitting cost due to occurrence timing of a seismic event. Since the severe deterioration case shows a higher retrofitting cost, durability of the pipe material is also an important factor in the seismic performance of the lifeline network.



23.21 Effect of deterioration on seismic retrofitting cost (Wada *et al.*, 2010).

23.5 Future trends

In the previous sections, an assessment procedure using the probability of system-performance failure for the existing lifeline network system is proposed, and case studies are carried out in order to evaluate the effectiveness of retrofitting strategies for an existing water-distribution network model.

Several important results can be summarized as follows:

- Old segmented pipes with poor mechanical joints should be replaced by newly developed segmented pipe joints with locking mechanisms, whose remarkable strength makes them appropriate in seismic-disaster prevention.
- Damage-monitoring technology is useful in identifying seismically damaged areas in the restoration stage immediately after the seismic event.
- For optimal decision making with seismic investment, our proposed measure of the value index is useful for all stakeholders in understanding key features relating to financial planning.
- Effective planning of seismic-retrofitting activities and disaster mitigation for the existing lifeline network system can be achieved using the newly proposed assessment method.

In further studies, the following items should be investigated:

- For rapid restoration/recovery, the emergency action teams from all the company engineers working on the water-pipeline systems have to be prepared/organized in advance.
- New technology will be necessary to identify small leakage accidents at many locations.

- Seismically strong segmented pipe joints have to be supplied at low cost, because partial replacement of pipe joints is not effective. A complete replacement of old pipes over a limited period will be useful to guard against any future strong earthquakes.
- Since peak ground deformation results in significant failure of buried pipelines, further study should be devoted to the seismic design of buried pipelines crossing liquefaction-sensitive areas or active fault zones.

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Seismic risk assessment of water supply systems

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Abstract: Water supply is vital to the function of an industrialized society and important to emergency response and recovery after earthquakes. Water supply systems and other civil infrastructure systems are intricately linked with the economic well-being, security, and social fabric of communities. This chapter deals with seismic risk assessment of water supply systems. A general framework for evaluating seismic risk of water supply systems is presented. The framework contains five steps and includes seismic hazard characterizations, system property characterizations, component response evaluation, system property evaluation, and consequence assessment. The assessment makes use of probabilistic seismic hazard analysis, theoretical and empirical models to estimate pipeline response, hydraulic analysis for heavily damaged water systems, and multi-scale simulations of complex water systems. Each of these steps is described separately, and the water supply system in Los Angeles, one of the largest water supply systems in the world, is used to illustrate these assessment steps. In addition, sources of further information and future trends are briefly commented.

Key words: earthquake, GIRAFFE, Los Angeles water supply system, risk curves, Monte Carlo simulations.

24.1 Introduction

A water supply system delivers water from sources to customers, and provides services vital to the function of an industrialized society and important to emergency response and recovery after disastrous events (e.g., earthquakes). Taken individually, or in aggregate with other lifeline systems (e.g., electric power, natural gas and liquid fuels, telecommunications, transportation, and waste disposal), water supply systems are intricately linked with the economic well-being, security, and social fabric of the communities. Significant advances in seismic risk assessment of water supply systems, or more broadly lifeline systems, have occurred in the last two decades (e.g., O'Rourke *et al.* 2004a). Early studies were focused on component behavior and simple system models (e.g., Hwang *et al.* 1998; O'Rourke and Liu 1999; Li and He 2002). As more advanced experimental and computational modeling was developed, network simulations were explored to assess system reliability (e.g., Wang and O'Rourke 2008; Wang *et al.* 2010). Most recently,

the economic and community consequences of earthquake damage have been integrated with system simulations to link component behavior through system reliability assessments all the way to regional economic impact (e.g., Chang *et al.* 2002; Rose and Liao 2005; Chang *et al.* 2008; Rose *et al.* 2011).

This chapter deals with seismic risk assessment of water supply systems and illustrates its recent advances through the water supply system in Los Angeles, one of the largest water supply systems in the world. After the introduction in this section, Section 24.2 presents a general framework for evaluating seismic risk of water supply systems. There are five principal steps in the framework, consisting of seismic hazard characterizations, system property characterizations, component response evaluation, system response evaluation, and consequence assessment. Each of these steps is described separately in Sections 24.3–24.7, and the water supply system in Los Angeles is used as an illustrative example in each section. Sources of further information and future trends are also briefly commented in Sections 24.9 and 24.8, respectively.

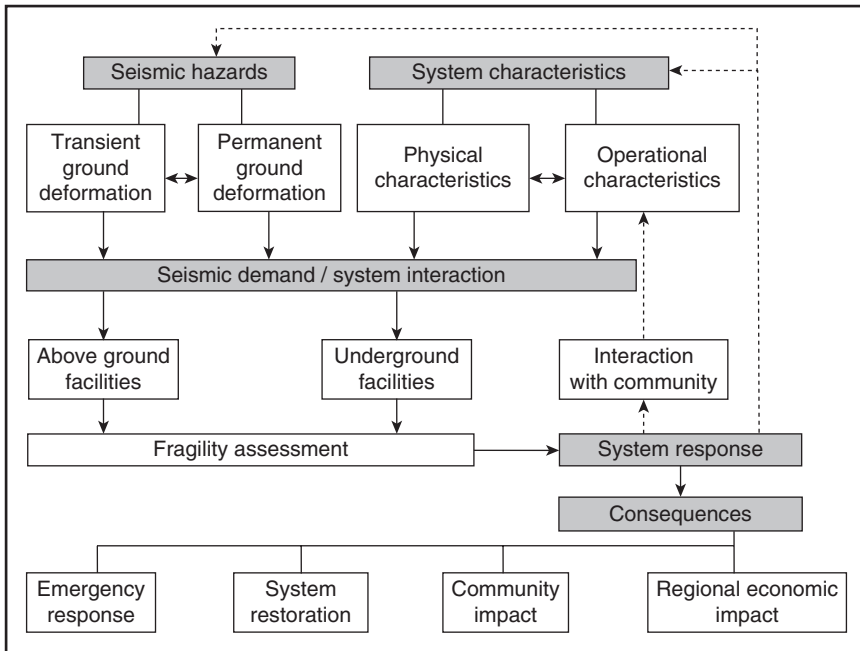
24.2 General framework for evaluating seismic risk

A general framework for evaluating the seismic risk of water supply systems is shown schematically in Fig. 24.1 (O'Rourke *et al.* 2004a; Wang and O'Rourke 2008). Seismic hazards are combined with system characteristics to account for the effects of transient ground motion and permanent ground deformation on both above ground and underground facilities. Fragility analyses of system components are used to assess the response of system components and overall system as a whole, from which the consequences with respect to the broader user community of the water supply system are derived. Based on results of the risk assessment, performance goals of the water supply system can be set accordingly and achieved through interaction with the community by altering operational and physical characteristics of the system, and by mitigating seismic hazards.

There is a basic chain of activities that predominates in this framework. The chain starts with characterizations of seismic hazard and system properties, then proceeds to analyses of interactions between them, from which system response and evaluation of community impacts follow. This basic chain is emphasized in Fig. 24.1 and its principal features are discussed under the sections that follow, using the Los Angeles water supply system as an illustration.

24.3 System characteristics

System characteristics involve both physical and operational attributes. As summarized in Table 24.1, physical characteristics involve types and numbers



24.1 Framework for seismic risk assessment of water supply systems (after O'Rourke *et al.* 2004a; Wang and O'Rourke 2008).

Table 24.1 System characteristics of water supply systems

Physical characteristics	Operational characteristics
Components	Planning
Connectivity	Mapping and GIS
Functionality	Monitoring and maintenance
Interaction	Operational procedures

of components that constitute the system, connectivity among components, required functionality for each component, and how different components interact with one another to achieve the designated functionality of the system as a whole. In general, a water supply system consists of a network of pipes, pumps, and valves that move water from sources to customers, tanks and reservoirs that store water to accommodate fluctuations in demand as a result of varying rates of usage or fire protection needs, and other supporting infrastructures. The operational characteristics of the

system include its organizational and social aspects. Those who operate the system are the interface between the system and the user communities. Operational characteristics involve the system operating procedures, mapping, monitoring, maintenance, and planning. Because each water supply system has its unique system characteristics, both physically and operationally, proper characterization of the system is an important step in seismic risk assessment of water supply systems.

The water supply system in Los Angeles, operated primarily by the Los Angeles Department of Water and Power (LADWP), is utilized as an illustration in this chapter. The system characteristics necessary for the seismic risk assessment are described in this section. The physical characteristics of the LADWP system are described first, followed by the system operational characteristics, which are integrated into a hydraulic network model. The system characteristics described in this section are reproduced from Wang and O'Rourke (2008), and the information is mostly provided by LADWP in the form of hydraulic network model (LADWP 2002a), water flow diagrams (LADWP 2002b), water maps (LADWP 2002c), and digital ortho photographs of the greater Los Angeles area.

24.3.1 Physical characteristics of the LADWP system

LADWP acts as a water 'retailer' in the greater Los Angeles area, supplying water directly to individual customers. Plate II (between pages 452 and 453) shows an overview of the LADWP system. The LADWP water supply system provides water to about 660 000 customers, representing 3.8 million people in a service area of approximately 1200 km². The system contains approximately 10750 km distribution lines and 1000 km trunk lines. The total water consumption of the LADWP system in a typical summer and winter day is about 2.5×10^6 and 1.2×10^6 m³, respectively. Two-thirds and one-third of the water consumption is accounted for by residential and commercial/governmental uses, respectively. Only a very small portion of water is consumed by industries (Toprak 1998).

The general water flow pattern in the LADWP system is from north to south, primarily by gravity flow. LADWP has three sources to meet its water demands: the local groundwater basins (about 15%), the Los Angeles aqueducts, and purchases from Metropolitan Water District (MWD) that acts as a regional water 'wholesaler' providing water to water supply agencies, such as LADWP. Water from northern California, including the Los Angeles aqueducts and MWD, is the major water source (about 85%) for the LADWP system. The water from northern California is first transferred to the water treatment and distribution hub, Van Norman Complex, in the northern Los Angeles, treated in a water treat-

ment plant, and then distributed to customers through trunk lines and distribution lines.

24.3.2 Operational characteristics of the LADWP system

The LADWP system operational characteristics, such as the system operating procedure, monitoring, maintenance, and planning, are consolidated into a hydraulic network model (LADWP 2002a) by LADWP engineers, using a commercial software, H2ONET (MWH Soft, Inc. 1999). H2ONET is an interactive, multi-application software program for the modeling of water distribution pipeline systems. It combines a point and click interface for network construction, drawing, and database management, highly advanced and computationally efficient hydraulic and water quality simulation modules based on EPANET (EPA 2005), and a graphical interface running within the AutoCAD (Autodesk 2005) for the Windows environment. H2ONET not only is capable of construction and maintenance of the comprehensive water supply system data inventory with reference to spatial coordinates, but also offers flexible data exchange with other software, such as EPANET and GIS, enabling integration with other relevant information and data.

The LADWP H2ONET hydraulic network model contains all kinds of physical components in the system, such as tanks, reservoirs, pump stations, regulation stations, pipe, and user demands, as well as operational aspects of the system, such as behavior and control parameters for pump stations and regulation stations. It contains 9287 nodes and 10665 links, representing about 2186 km of trunk lines, 230 regulation stations, 110 tanks and reservoirs, 151 local groundwater wells, and 73 pump stations. To control the water flows in different pipes or to provide redundancy for enhancing the performance reliability of regulation stations, a regulation station usually contains multiple control valves, resulting in 591 valves in the 230 regulation stations, including 493 pressure-reducing valves, 50 throttle control valves, 35 flow control valves, 8 user-defined valves, and 5 pressure-sustaining valves. Similarly, 73 pump stations contain 284 pumps, in addition to 151 pumps in 151 local groundwater wells. Since the H2ONET hydraulic network model was originally used by LADWP engineers for system planning and management purposes, only pipelines with relatively large diameters or significant serviceability are included in the model. Most local distribution pipelines with relatively small diameters are included implicitly in the H2ONET hydraulic model by aggregating them to demand nodes.

The H2ONET hydraulic network model also contains a comprehensive data inventory describing operational aspects of the LADWP system. The

operational aspects include the pre-defined setting parameters for regulation valves, such as pressure setting for pressure reducing valves, and controls on links (pipes, valves, or pumps) for altering the operational status of the links.

24.4 Seismic hazards

Seismic hazard at a particular site can be estimated quantitatively by either deterministic seismic hazard analysis, DSHA (Reiter 1990), in which a particular earthquake scenario is assumed, or probabilistic seismic hazard analysis, PSHA (Cornell 1968; Reiter 1990), where uncertainties in earthquake magnitude, location, and occurrence time are explicitly considered. The seismic hazard analysis, regardless of deterministic or probabilistic, starts with identification and evaluation of earthquake sources by reviewing geologic evidence, tectonic evidence, historical seismicity, and instrumental seismicity (Kramer 1996). Then, a recurrence relationship, which specifies an average rate at which an earthquake of some magnitude will be exceeded, is used to characterize the seismicity of each source zone. The ground motion produced at each site is determined with the aids of suitable attenuation relationships, which describe the decrease of ground motion with increasing distance. Finally, PSHA combines the uncertainties in earthquake location, earthquake magnitude, and attenuation relationships to obtain the probability that a ground motion level will be exceeded during a particular time period.

The United States Geological Survey (USGS) has adopted the PSHA approach to all locations in the US, and prepared probabilistic seismic hazard maps in 1996, with revised maps in 2002 and 2008 (available at the USGS website <http://earthquake.usgs.gov/hazards/products/>). The seismic hazard information for Los Angeles is available at the same web sites, and the maps revised in 2002 are referred to as the USGS 2002 dataset. The USGS 2002 dataset represents a consensus within the geo-scientific community regarding earthquake parameters that contribute to the seismic hazards.

The USGS 2002 dataset is commonly used for site-specific seismic performance evaluation, in which only earthquake ground motion at a site is required and spatial correlation among earthquake ground motions across many sites is not of concern. In contrast, seismic risk assessment of water supply systems, which are distributed over a broad geographic area, requires seismic hazard information at various sites across the system. For example, seismic hazard information over an area of approximately 1200 km² is required in the seismic risk assessment of the LADWP water supply system. The earthquake ground motions with the same annual

probability of exceedance differ, but are correlated at various sites over the LADWP service area. The USGS 2002 dataset is for the aggregated hazard at a particular site associated with potentially thousands of earthquakes. If it was used simultaneously at a number of different sites distributed throughout a functioning system, it would overestimate the earthquake effects on system performance because the spatial correlation is ignored that exists among different sites when affected by the same earthquake.

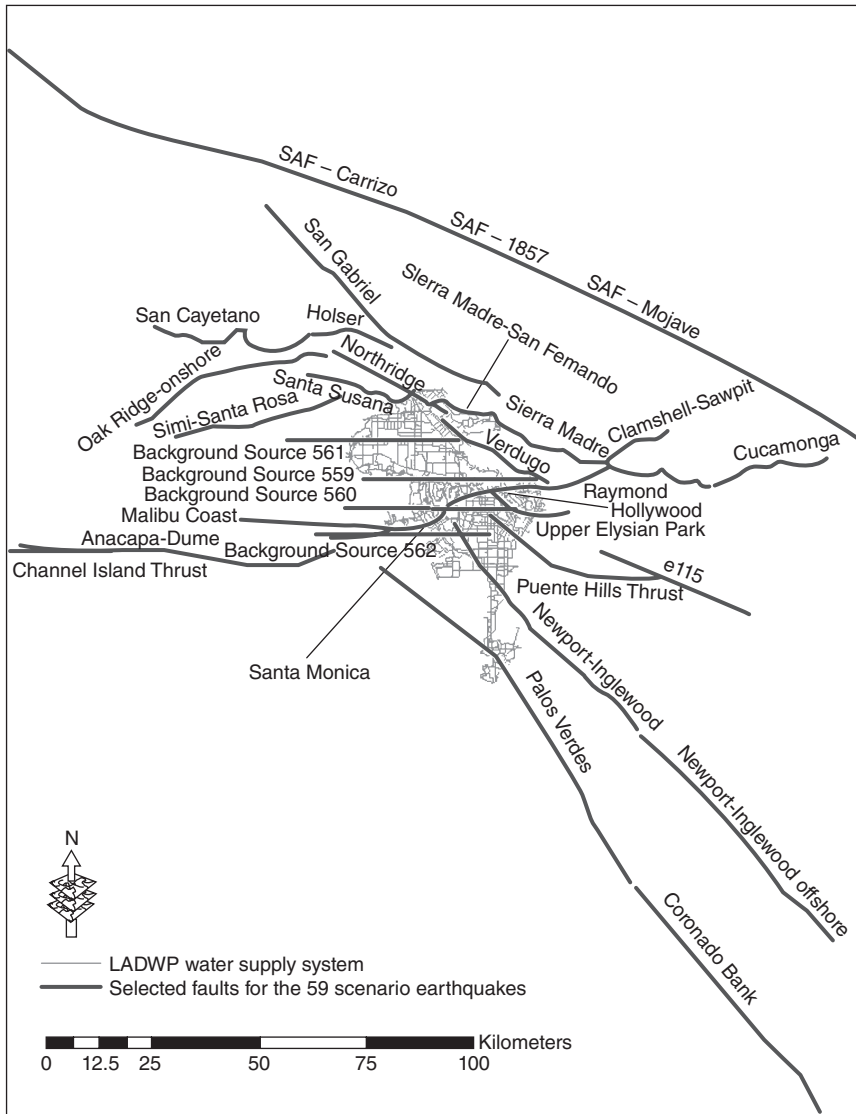
One potential approach to account for the spatial correlation is to simulate water supply system performance for a comprehensive set of scenario earthquakes, each of which is associated with an annual frequency of occurrence. Each scenario earthquake represents one branch of the logic trees in PSHA, and denotes a discrete choice of input parameters and models. However, since detailed models of a large lifeline system are often quite complex, it may not be practical to simulate the system response for an exhaustive set of scenario earthquakes (which may contain thousands of scenario earthquakes for high seismic hazard areas, such as Los Angeles). This raises a challenge on how to account for the large number of scenario earthquakes omitted and their contribution to the probabilistic hazard and risk analysis results.

Chang *et al.* (2000) proposed an approach in which a number of scenario earthquakes are chosen based on their combined contribution to the aggregate hazard, and the contribution of omitted earthquakes is accounted for by optimizing the annual frequencies of the earthquakes. Using this approach, a new set of event annual frequencies is sought such that the relevant ground shaking hazards computed from the selected set at the principal sites of interest match the target probabilistic hazards, such as those associated with the USGS 2002 dataset.

24.4.1 LADWP seismic hazard characterizations using 59 scenario earthquakes

An approach was developed by Lee *et al.* (2005), similar to the one proposed by Chang *et al.* (2000), to characterize the seismic hazards in the LADWP water supply system by choosing a suite of 59 scenario earthquakes so that their optimized annual frequencies of occurrence match the USGS 2002 probabilistic seismic hazards in the LADWP service area. Figure 24.2 shows spatial distribution of the faults for the 59 scenario earthquakes, together with the LADWP water supply system. Table 24.2 summarizes characteristics of the 59 scenario earthquakes.

The seismic hazard associated with the 59 scenario earthquakes was adjusted to match that of the USGS 2002 dataset over the LADWP water supply system by a multivariate, nonlinear optimization process. The 59



24.2 Spatial distribution of the faults for 59 scenario earthquakes (after Wang and O'Rourke 2008).

annual occurrence frequencies for the 59 scenario earthquakes are the optimized variables. The target function was selected by minimizing an error function for the sum of the differences between the hazard curves, i.e., variation of annual exceedance frequency as a function of the spectral acceleration, S_{A1} , at $T = 1$ s at equivalent rock sites, from the 59 scenario

Table 24.2 Characteristics of 59 scenario earthquakes

Scenario ID	Scenario Name	Magnitude M_w	Annual occurrence frequency
12	el15	6.8	3.60E-03
18	SAF – Mojave	7.3	4.13E-03
19	SAF – Carrizo	7.4	2.28E-03
21	SAF – All southern segments	8.1	3.00E-03
22	SAF – 1857	7.8	9.61E-03
23	SAF – Southern 2 segments	7.7	3.37E-03
118	Holser	6.5	1.66E-04
119	Hollywood	6.4	6.64E-06
120	Raymond	6.5	7.41E-04
122	Clamshell-Sawpit	6.5	1.06E-03
141	Newport-Inglewood offshore	7.1	2.56E-03
145	Coronado Bank	7.6	1.75E-03
159	Newport-Inglewood	7.1	8.10E-04
160	Newport-Inglewood	6.6	2.37E-03
161	Newport-Inglewood	6.6	5.58E-04
162	Newport-Inglewood	6.6	1.50E-04
166	Sierra Madre	7.2	7.45E-04
167	Sierra Madre	6.7	4.40E-03
168	Sierra Madre	6.7	2.21E-04
169	San Gabriel	7.2	1.53E-03
170	San Gabriel	6.7	9.97E-05
171	San Gabriel	6.7	1.27E-03
173	Malibu Coast	6.7	2.70E-06
174	Santa Monica	6.6	5.23E-04
175	Verdugo	6.9	9.65E-04
176	Verdugo	6.4	1.57E-05
177	Verdugo	6.4	2.84E-06
189	Oak Ridge – onshore	7	4.13E-03
191	Oak Ridge – onshore	6.5	3.86E-03
195	San Cayetano	7	6.86E-03
196	San Cayetano	6.5	6.03E-03
198	Santa Susana	6.7	3.01E-03
202	Simi-Santa Rosa	7	6.35E-04
203	Simi-Santa Rosa	6.5	2.87E-04
219	Anacapa-Dume	7.5	9.36E-04
220	Anacapa-Dume	7	5.70E-04
221	Anacapa-Dume	7	9.43E-04
222	Anacapa-Dume	6.5	1.29E-06
370	Northridge	7	1.43E-03
371	Northridge	6.5	2.88E-04
372	Northridge	6.5	2.37E-05
378	Channel Island Thrust	7.5	5.12E-04
388	Upper Elysian Park	6.4	6.13E-05
397	Puente Hills blind thrust	7.1	8.63E-04
398	Puente Hills blind thrust	6.6	1.04E-05
399	Puente Hills blind thrust	6.6	8.21E-05
440	Cucamonga	6.9	6.18E-03

(Continued)

Table 24.2 Continued.

Scenario ID	Scenario Name	Magnitude M_w	Annual occurrence frequency
443	Sierra Madre-San Fernando	6.7	9.41E-04
444	Palos Verdes	7.3	1.05E-03
446	Palos Verdes	6.8	8.20E-04
447	Palos Verdes	6.8	6.24E-04
451	Palos Verdes	6.3	3.27E-03
452	Palos Verdes	6.3	1.44E-03
453	Palos Verdes	6.3	2.07E-03
454	Palos Verdes	6.3	2.17E-03
559	Background Source	7	1.05E-03
560	Background Source	7	7.75E-04
561	Background Source	7	1.29E-03
562	Background Source	7	7.63E-04

earthquakes and USGS 2002 dataset at each of the 56 control points spatially distributed over the LADWP water supply system.

24.4.2 Earthquake-induced ground motion

The earthquake-induced ground motion results in ground deformation and affects lifeline systems by transient and permanent ground deformation (TGD and PGD, respectively). TGD refers to recoverable movement of the ground and soil strain as the seismic wave propagates through. The most common cause of TGD is the traveling ground waves by seismic body waves, either compressional or shear waves, propagating from a seismic source. The body waves are characterized by a relatively high apparent wave propagation velocity that results in low ground strains, generally insufficient for serious lifeline damage except for highly deteriorated lifeline systems and/or near source strong shaking. Surface waves may be amplified as seismic body waves propagate in some specific local geologic settings, such as large sedimentary basins, resulting in TGD capable of considerable lifeline disruption. Significant TGD-induced lifeline damage quite often occurs as the result of adverse ground conditions, such as narrow sediment-filled valleys with respect to relatively rigid valley boundaries, steep ridges and elevated topography accompanied locally by slip in fractured rock, and virtually level ground with liquefiable soils relative to adjacent and underlying competent materials.

PGD refers to the irrecoverable movement of the ground that often results from ground failure, but also may be the result of modest levels of

volumetric strain and shear distortion. The main sources of PGD include surface faulting, tectonic uplift and subsidence, liquefaction, landslides, and densification. Surface faulting and tectonic uplift and subsidence are associated with the crustal deformation from which the ground shaking originates. In contrast, local ground conditions are responsible for liquefaction, landslides, and densification. Liquefaction usually occurs in loose saturated granular soils, which transform to a liquefied state or condition of substantially reduced shear strength when subjected to seismic loading. Inertial forces from ground shaking may trigger mass movement of the ground, i.e., landslides, in the forms of rock falls, relatively shallow slumping and sliding of soils, or relatively deep translation and rotation of soil and rock. The earthquake-induced shaking can decrease the volume of dry or partially saturated granular soils, resulting in ground subsidence.

24.4.3 Seismic demand on system components

For each of the 59 scenario earthquakes, strong ground motion data at 572 points were generated, and the seismic demands on the system components in the LADWP water supply system were determined accordingly. Ground motion contour surfaces were developed from the data at the 572 points using local polynomial interpolation in a GIS software, and further corrected for the site conditions. Then, the seismic demands on the system components were determined according to their locations. Consider, for example, the water pipelines, the seismic performance of which is closely related to the peak ground velocity (PGV) they are subjected to. The PGV was therefore utilized as the primary seismic demand parameter. The PGV contour surfaces were interpolated from these 572 points, and site condition corrections were applied according to the NEHRP-HAZUS procedures. With the aid of GIS software, the spatial distribution of the LADWP pipelines were superimposed on the PGV contour surfaces to determine the seismic demand on each pipeline in accordance with its respective location. The GIS function discretized the pipeline spatially in accordance with the PGV contour and assigned the corresponding PGV value to each pipeline segment subsequently. Plate III (between pages 452 and 453) shows the LADWP pipeline system and the PGV contour surface for the Scenario 175 Verdugo earthquake. Similar procedures were applied repeatedly to other 58 scenario earthquakes, as well as other ground motion parameters, if necessary.

24.5 Component responses

The effects of TGD and PGD are evaluated for components of above-ground and underground facilities. The performance of underground

facilities under seismic loading is assessed primarily by geotechnical engineers, who have developed models for soil–structure interaction, including empirical models based on observations from the past earthquakes, closed-form analytical methods, and numerical simulations, such as finite element and finite difference analyses (e.g., O’Rourke and Liu 1999; O’Rourke *et al.* 2004a, 2004b; Shi 2006; Wang 2006). Structural engineers focus typically on the performance of above-ground facilities. The principles of structural dynamics (e.g., Chopra 2001) are routinely applied in the engineering analysis and design of above-ground structures. Theoretical models, as well as advanced numerical models, have been developed and applied for analyzing the above-ground structure responses to ground shaking (e.g., Naeim 2003; Chopra 2005).

To account for uncertainty with respect to component or facility response, seismic behavior is frequently characterized by fragility curves that provide the probability of failure as a function of the demand (e.g., PGV or peak ground acceleration, PGA). Fragility curves can be derived from either observations of past earthquakes or, more typically, Monte Carlo simulations that have special capability in quantifying uncertainty.

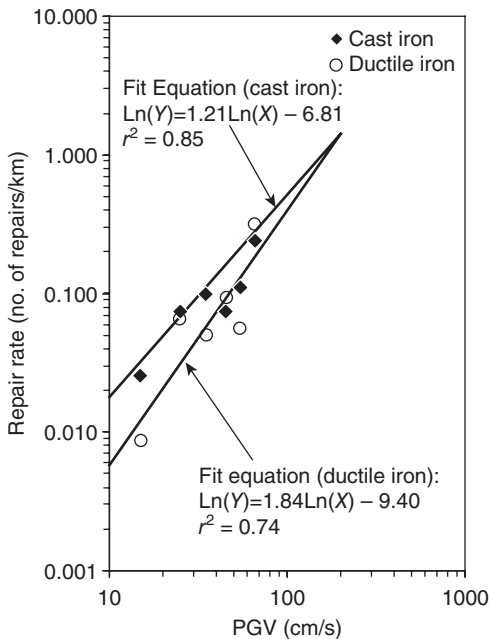
24.5.1 LADWP system component responses

Seismic risk assessment of the LADWP system has been focused on seismic wave (TGD) interactions with pipelines, which have a primary impact on system performance (Wang and O’Rourke 2008). Analytical and/or numerical methods have been developed to model the seismic wave interaction with pipelines (e.g., O’Rourke *et al.* 2004b; Shi 2006; Wang 2006), and they can be utilized to evaluate the seismic performance of pipelines. However, to apply those models, details are required on seismic wave characteristics (e.g., predominant period and apparent wave velocity), pipeline properties (e.g., cross-sectional area and axial deformation stiffness), and ground conditions (e.g., burial depth, unit weight of soils, and pipe-soil interface friction angle). Such information is quite often not available, particularly for a large and complex system, such as the LADWP system, which contains about 12000 km pipes covering an area of approximately 1200 km². One alternative method, which requires minimal input parameters, for estimating pipeline damage during earthquakes was to use regressions between observed repair rates and measured seismic parameters during previous earthquakes.

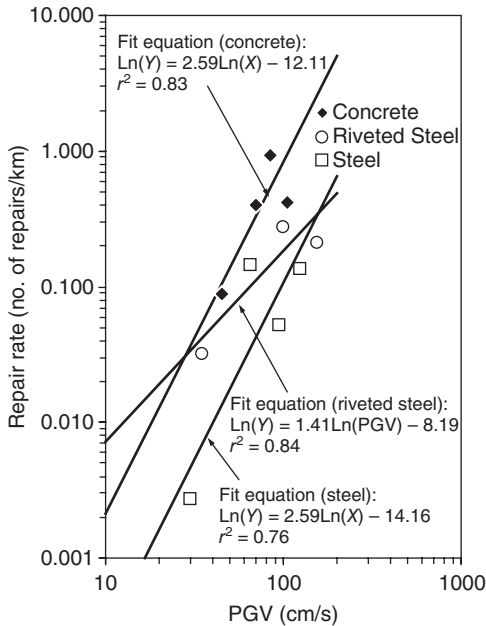
24.5.2 Regressions for estimating pipeline damage

The damage to cast iron and ductile iron pipelines was estimated using the regressions developed by Jeon (2002). Empirical data were collected about

pipeline repairs and locations of repairs after the 1994 Northridge earthquake in Los Angeles and incorporated in a large GIS database, consisting of over 12000 km of pipelines in the greater Los Angeles area and more than 240 strong motion records. Records from over 240 strong motion stations throughout the earthquake-affected area were analyzed with respect to various seismic parameters (Toprak 1998). The spatial distributions of different seismic parameters were estimated by interpolation and superimposed on the pipeline network and spatially distributed database of pipeline damage. Where possible, pipeline repairs in zones of documented PGD were screened from the repair rates so that the resulting statistics would reflect principally the effects of seismic waves or TGD. Using a GIS software, the repair rate was calculated for areas influenced by specific seismic parameters. Correlations then were developed through regression procedures to obtain the most statistically significant relationships among repair rate and values of different seismic parameters. It was found that the most statistically relevant parameter for correlation with repair rate is PGV. Figure 24.3 shows the linear regressions developed by Toprak (1998) and Jeon (2002) and utilized to calculate the repair rates from PGVs for cast iron and ductile iron pipelines. Interested readers are referred to Jeon



24.3 Regressions of repair rate vs. PGV for cast iron and ductile iron pipelines (after Jeon 2002).



24.4 Regressions of repair rate vs. PGV for concrete, riveted steel, and steel pipelines (after Wang and O'Rourke 2008).

(2002) and Jeon and O'Rourke (2005) for details on the regression analysis procedures. Following their procedures, regressions of repair rate vs. PGV for trunk lines composed of other materials were developed (Wang and O'Rourke 2008). Figure 24.4 shows the regressions for concrete, riveted steel, and steel pipelines.

24.6 System responses

After evaluating the component response of either above-ground or underground facilities, the seismic risk assessment proceeds to system integration, in which the system performance is evaluated according to the functionality and serviceability of the entire network. As the functionality of a water supply system is to deliver water with sufficient pressure to customers, its overall system performance is assessed using hydraulic network analysis, given the estimation of earthquake-induced damage at the component level (such as damage to trunk lines, distribution lines, and pump stations). Hydraulic network analysis is based on mass and energy conservation equations, and is employed to integrate the impacts of component damage into the assessment of the designated system functionality as a whole. More specifically, hydraulic network analysis is performed to determine where

the water outage area will be and how the customer demands are satisfied, given the earthquake-induced damage scenario at the component level.

A special hydraulic analysis computer program, Graphical Iterative Response Analysis of Flow Following Earthquakes (GIRAFFE), has been developed at Cornell University for hydraulic network analysis of damaged water supply systems (Shi and O'Rourke 2008; Wang and O'Rourke 2008). GIRAFFE is able to compensate for the limitations of commercially available hydraulic analysis software in predicting unrealistic negative pressures in a heavily damaged system by eliminating portions of the network containing negative pressures. GIRAFFE simulates trunk line damage explicitly by a Poisson process to model the occurrence of pipe damage and hydraulic models for both pipe break and leak to model the water outflow from the system. In addition, damage to distribution lines is simulated implicitly by adjusting the nodal demands in the trunk line system. GIRAFFE is capable of performing Monte Carlo simulation and compiling the simulation results in a format that can be used in GIS. The LADWP water supply system performance during the 1994 Northridge earthquake was modeled using GIRAFFE, and it was found that GIRAFFE provided simulation results consistent with the observed water outage and the recorded flows in the system after the earthquake. The favorable agreement between simulation results and system measurements shows that GIRAFFE is capable of modeling severely damaged water supply systems, and provides confidence in applying GIRAFFE.

24.6.1 LADWP system response

GIRAFFE has been used to assess the seismic system performance of the LADWP water system. Monte Carlo simulation was performed for each of the 59 scenario earthquakes. In each simulation, GIRAFFE simulated the trunk line damage explicitly by implementing the hydraulic models for pipe breaks and leaks, and modeled the damage to local distribution system implicitly by adjusting the nodal demands according to the repair rates in the local distribution systems. Then, GIRAFFE utilized the EPANET hydraulic engine to check network connectivity, perform hydraulic analysis, and eliminate negative node pressures until no negative node pressures are present in the system. Finally, GIRAFFE compiled the results from each simulation and provided performance index, such as mean of system serviceability index (SSI), which is defined as

$$SSI = \frac{\sum_{i=1}^{n_i} \bar{Q}_i}{\sum_{i=1}^{n_{i0}} \bar{Q}_i} \quad [24.1]$$

where $\bar{Q}_i$ is the customer demand at node i , and n_{i0} and n_i are number of satisfied customer demands before and after imposing damage scenario, respectively.

Plate IV (between pages 452 and 453) shows an example of the simulated spatial distribution of water outage areas after the Scenario 175 Verdugo earthquake according to GIRAFFE simulation. The unsatisfied demand nodes are shown by yellow dots and the pipes with no flow as a result of earthquake-induced damage are shown by red lines. Note that the pipes with no flow in the undamaged LADWP system to provide redundancy in the system are shown by pink lines. As a result of high PGV values in the upper right quarter of the figure and concentrated damage in those areas, most water outage clusters in the upper right quarter. Sparse water outage also occurs in other parts of the system and the overall system serviceability index $SSI = 0.81$. Similar procedures were applied for each of the 59 scenario earthquakes and the simulation results from the 59 individual scenario earthquakes were integrated in the form of risk curves, as described in the next subsection.

In addition, GIRAFFE was utilized to study the effects of water loss from storage tanks. Leaking and ruptured pipelines will draw down the water levels in tanks and local reservoirs, thereby further reducing flow and pressure in the pipeline network. Evaluating the effects of water losses from tanks and reservoirs may provide a more representative model for post-earthquake performance because it will take water utility crews some time to isolate leaks and pipeline breaks to reduce their impact on local water sources.

For each simulation that accounts for tank water losses, after all negative pressure nodes were eliminated and the simulation results were obtained, the water levels of the storage tanks in the system were updated according to the flow rates determined from the hydraulic analysis to simulate the impact of leakage from damaged pipelines for a 24-hour period after the earthquake. The system with the updated tank water level was reanalyzed in GIRAFFE to eliminate additional negative pressure nodes resulting from the loss of storage water in tanks, and the second set of simulation results with a 24-hour period of running tanks was generated.

24.6.2 Risk curves

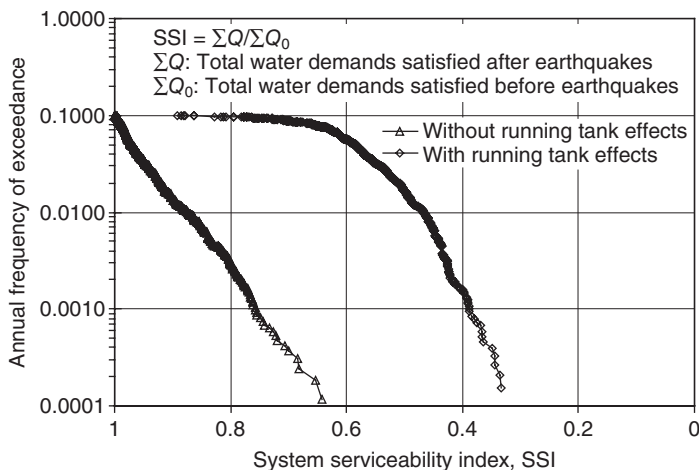
The simulation results for the 59 scenario earthquakes were integrated in the form of risk curves, plotting the variation of annual exceedance frequency as a function of system serviceability index SSI. Each scenario earthquake is characterized by an optimized annual frequency of occurrence, as shown in Table 24.2. If n_{mc} Monte Carlo samples were generated for each of the 59 scenario earthquakes and the corresponding n_{mc} set of

simulation results are assumed to occur with equal likelihood, i.e., equal annual frequency of occurrence, the annual frequency of occurrence for each set of Monte Carlo simulation results is one n_{mc} th of that for the associated scenario earthquake. Consider, for example, the Scenario 175 Verdugo earthquake, the annual occurrence frequency of which is $9.65\text{E-}04$. Then, the annual occurrence frequency for a SSI of 0.81, associated with the simulation results in Plate IV (between pages 452 and 453), is $9.65\text{E-}06$, if $n_{mc} = 100$. The similar process was applied to all $59 \times n_{mc}$ sets of simulation results, i.e., SSI, each of which is associated with its own annual occurrence frequency. The $59 \times n_{mc}$ SSI were then sorted in an increasing order of SSI, and the annual frequency of exceeding a given SSI, SSI_0 , is calculated by a summation of the annual occurrence frequencies for those $SSI \leq SSI_0$,

$$F(SSI_0) = \sum_{SSI \leq SSI_0} f(SSI) \quad [24.2]$$

where $F(SSI_0)$ is the annual frequency of exceeding SSI_0 , and $f(SSI)$ is the annual occurrence frequency of SSI.

Figure 24.5 shows the system risk curves plotting the variation of annual exceedance frequency as a function of SSI. The system risk curve without a 24-hour period of running tanks is shown by open triangles and that with a 24-hour period of running tanks is shown by open squares. The system risk curve without a 24-hour period of running tanks varies almost linearly in the semi-logarithmic scale, with a minimum SSI of 0.64 corresponding to an annual frequency of 0.0001, or 10000-year recurrence interval. The



24.5 System risk curves (after Wang and O'Rourke 2008).

system risk curve with 24-hour running tanks plots to the right of the first curve, indicating much lower SSI values for the same annual exceedance frequency. The linear portion of the curve, with lower SSI values, is approximately parallel to the system risk curve without 24-hour period of running tanks. The difference between the curve without 24-hour period of running tanks and its parallel portion in the curve with 24-hour period of running tanks signifies the effects of running tanks. After 24-hour period of running tanks, the system deteriorates rapidly and the SSI decreases significantly. The system risk curve with a 24-hour period of running tanks provides an estimate of LADWP system performance after earthquakes that reflects the deteriorating capacity of local tanks and reservoirs, where water levels drop because of pipeline damage after earthquakes.

24.7 Economic and social consequences

Economists and social scientists have contributed to a growing body of research and applications associated with the economic and social consequences of lifeline damage and loss of functionality. It is well recognized that lifeline disruption has a direct effect on business losses and social consequences. Consider, for example, water supply disruption, which adversely affects fire protection, the loss of which may trigger serious economic and social consequences. One catastrophic example is the 1906 San Francisco earthquake in which most of the City of San Francisco was burned down by the fires following the earthquake because of the water supply disruption and loss of fire-fighting capability (Gilbert *et al.* 1907).

The direct regional economic consequences of lifeline loss often set off a chain reaction of further production cutbacks among successive rounds of customers and suppliers that spread through the entire regional economy, which is described as ‘ripple effect’ or ‘multiplier effect’. Some well-established analysis methodologies in economics, such as input–output (I-O) analysis and computable general equilibrium (CGE) analysis, have been successfully utilized to assess the indirect economic effects of lifeline disruption.

The social and economic consequence evaluations have been performed for the LADWP water supply system. CGE models have been developed to assess the economic impacts caused by earthquake-induced losses of water supply (e.g., Rose and Guha, 2003; Rose and Liao, 2003, 2005; Rose *et al.* 2011). The community impacts of the seismic-induced lifeline loss have also been evaluated by other researchers (e.g., Chang and Miles 2003; Chang and Chamberlin 2004; Chang *et al.* 2008). Davidson and Cagnan (2004), Cagnan (2005), and Brink *et al.* (2010) modeled lifeline system restorations after earthquakes and simulated system repair and recovery

operations for both the LADWP water supply and electric power systems. Each of these simulations relies on information about damage states, as provided by the system simulation model developed in previous sections.

24.8 Future trends

The seismic risk assessment of the LADWP water supply system has focused on the seismic responses of pipelines and their impact on the overall system performance. The seismic responses of other components, such as pump stations, regulation stations, ground water wells, and tanks and reservoirs, are not included. It is useful to incorporate the seismic responses of those components and to integrate their impacts to the seismic risk assessment. The framework and procedures described in this chapter can be readily adapted to other components, provided that fragility information for those components is developed.

Because of the difficulty in predicting the occurrence of permanent ground deformation and delineating the spatial distribution of permanent ground deformation, the PGD effects on the system component performance during earthquakes were not incorporated in seismic risk assessment of the LADWP water supply system. It is also valuable to incorporate the PGD effects on the seismic responses of system components, as well as the overall system. The framework and procedures described herein can be easily modified to incorporate PGD effects.

Interdependencies among different lifeline systems (e.g., water and power systems) are one of the most prominent characteristics of lifeline systems. The damage and disruption of seismic hazards in other systems may affect the system being assessed because of physical proximity and/or operational interaction. For example, the loss of electricity affects water supply systems by rendering pumps and automatic valves inoperable, thereby affecting the flow and pressure in the water supply systems. Therefore, one important aspect in the lifeline seismic risk assessment is the interaction of the system under scrutiny with other lifeline systems.

In addition, since detailed simulation of a large water supply system is often complex and time consuming, the computational time and efforts are obviously demanding for seismic risk assessment of water supply systems. Such computational difficulty is particularly prominent for seismic risk mitigation in which repeated seismic risk assessments are needed for various mitigation options. Further studies on how to improve computational efficiency in seismic risk assessment and mitigation for water supply systems are therefore worthwhile to pursue. Some preliminary investigations have been performed using conditional samples from a single run of Monte Carlo

simulation and the concept of efficient frontier (Wang and Au 2009; Wang *et al.* 2010).

24.9 Sources of further information and advice

Further information on seismic risk assessment of water supply systems can be obtained from a long list of academic journals, conference proceedings, earthquake engineering research centers/groups, and professional bodies around the world. The list includes, but is not limited to, Earthquake Engineering Research Institute (EERI) in USA and its professional journal, *Earthquake Spectra*, and many other publications (e.g., proceedings of annual meeting, monographs, reconnaissance reports of major earthquakes), Technical Council on Lifeline Earthquake Engineering (TCLEE) of the American Society of Civil Engineers (ASCE) and its monographs and conference proceedings, the Multidisciplinary Center for Earthquake Engineering Research (MCEER <http://mceer.buffalo.edu/>) headquartered at the University at Buffalo, The State University of New York, and its monographs and technical reports, and the proceedings of world conference series on earthquake engineering. In addition, many earthquake engineering research centers/groups around the world conduct research relevant to the seismic risk assessment of water supply systems, e.g., the Pacific Earthquake Engineering Research (PEER) Center (<http://peer.berkeley.edu/>) headquarters at the University of California, Berkeley, the Mid-America Earthquake (MAE) Center (<http://mae.cce.illinois.edu/>) headquartered at the University of Illinois at Urbana-Champaign, the research groups at Kobe University (e.g., Kuwata *et al.* 2008) and Kyoto University (e.g., Javanbarg *et al.* 2008) in Japan and Tongji University (e.g., Li and He 2002, Li *et al.* 2007, Liu and Li 2009) in China.

24.10 Acknowledgments

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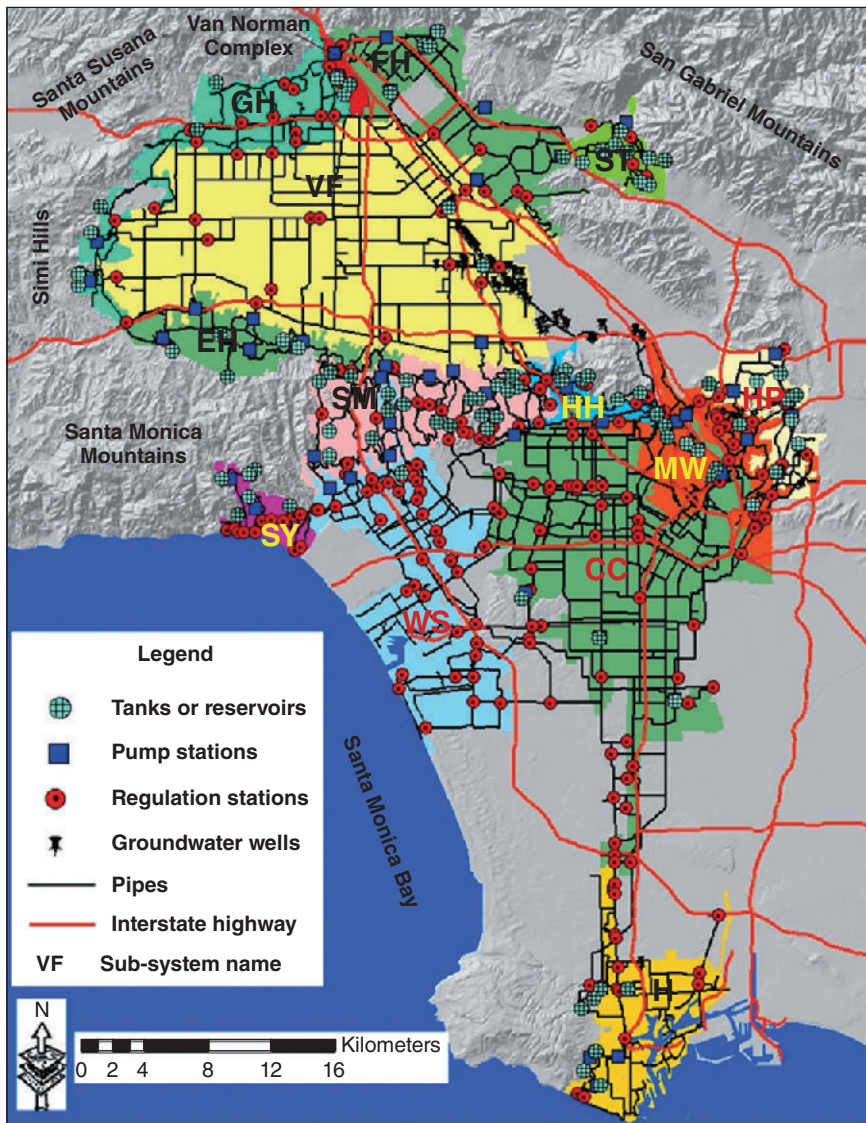


Plate II Overview of LADWP water supply system (after Wang and O'Rourke 2008).

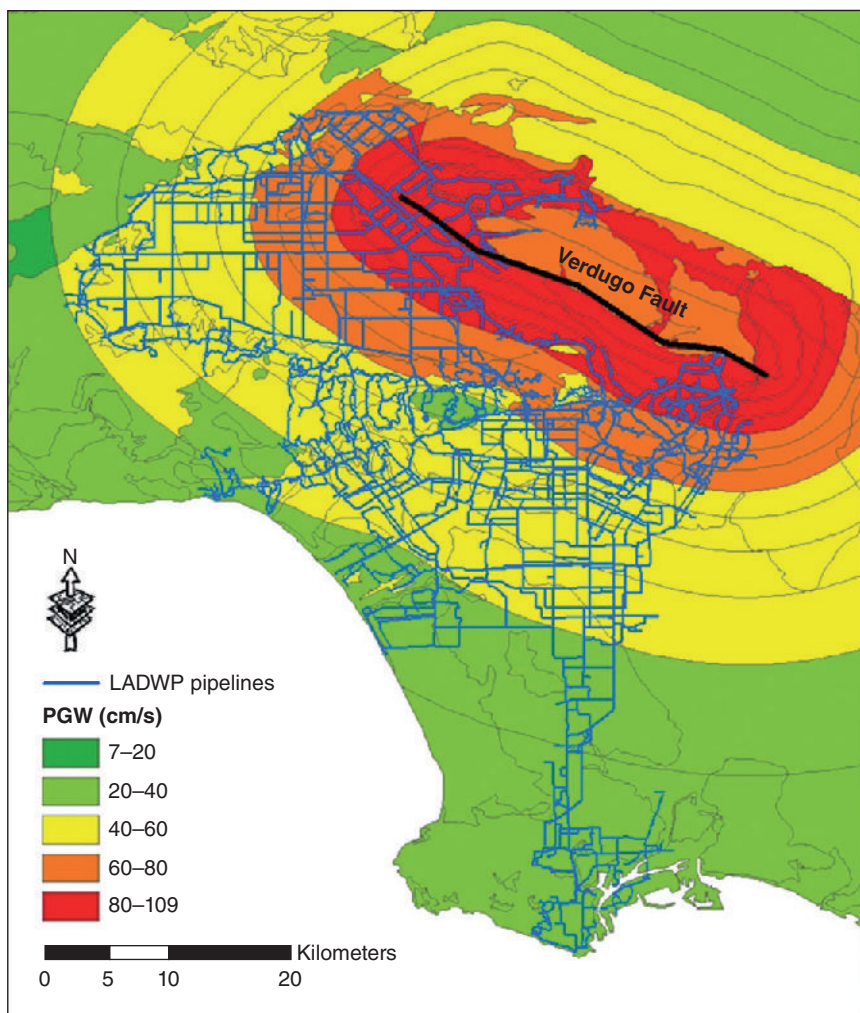


Plate III PGV contour surface for scenario 175 Verdugo earthquake (after Wang and O'Rourke 2008).

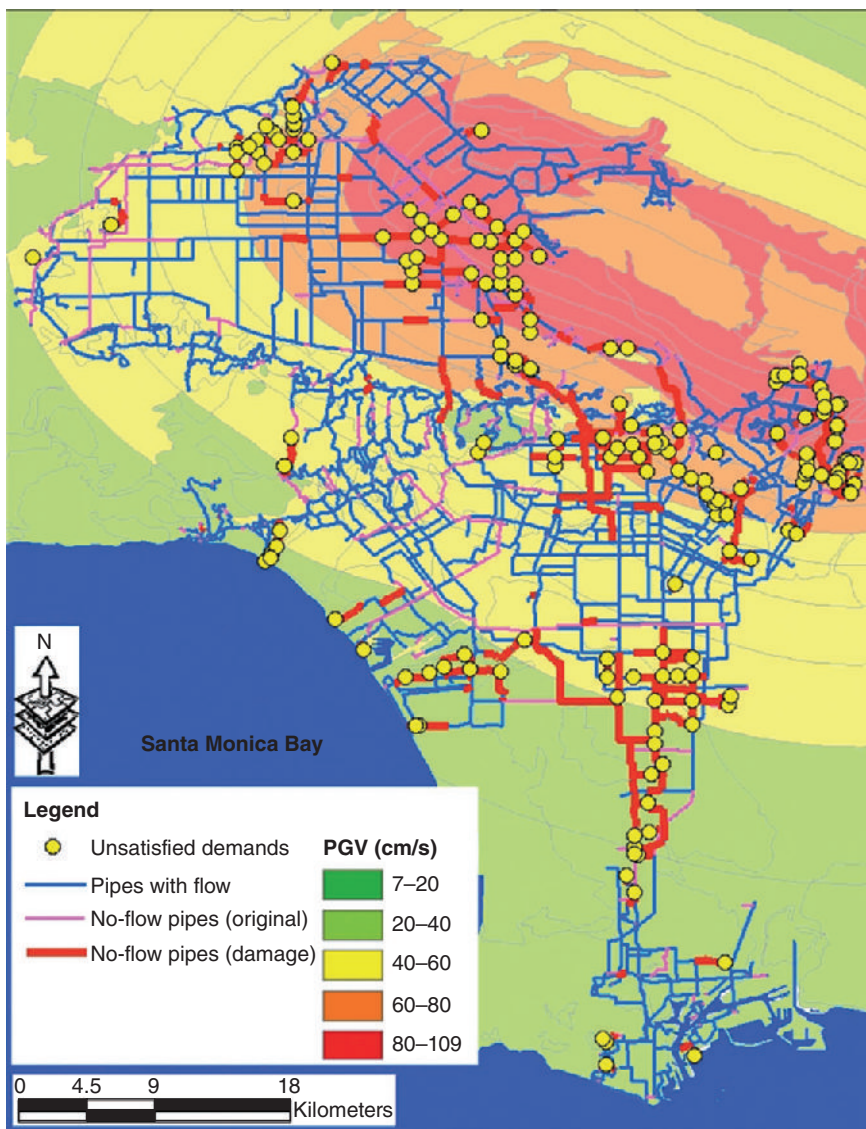


Plate IV Spatial distribution of water outage after scenario 175 Verdugo earthquake (after Wang and O'Rourke 2008).

Seismic risk assessment for oil and gas pipelines

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Abstract: Buried pipeline systems form a key part of global lifeline infrastructure, and any significant disruption to the performance of these systems often translates into undesirable impacts on regional businesses, economies, or the living conditions of citizens. This chapter addresses the considerations associated with the seismic risk assessment of pipelines providing transmission of natural gas or liquid hydrocarbons, and pipelines that are part of a gas distribution system serving a region. Particular reference is made with respect to philosophy, approaches, and technologies adopted in designing and operating pipelines to minimize pipeline damage during earthquakes.

Key words: pipeline, risk, soil–pipe interaction, performance goal.

25.1 Introduction

Quantification of anticipated seismic hazards is a key consideration in assessing performance of pipelines under seismic loading conditions. Evaluation of the performance of pipeline systems under such hazards commonly uses equations based on simplified assumptions or sophisticated numerical modeling techniques as described in guidelines dating back to the early 1980s (e.g., ASCE, 1984) and more recently updated (e.g., PRCI, 2009).

This chapter addresses the considerations associated with the seismic risk assessment of pipelines providing point-to-point transmission of natural gas or liquid hydrocarbons, and major distribution pipelines that are part of a natural gas distribution system serving a region or local community. Liquid hydrocarbons include a variety of products: crude oil, refined products such as gasoline, diesel fuel, jet fuel, lubricating oil, and gas liquids such as propane. For simplicity, hydrocarbon pipelines will be referred to as ‘oil’ pipelines in this discussion. The scope of this discussion does not include pipelines within facilities such as onshore drilling fields, offshore platforms, refineries, or major distribution facilities. The scope also does not include smaller pipelines incorporated into a municipal distribution system, which normally operate at lower pressures and are constructed of materials other

than steel. This scope is similar to that defined for liquid hydrocarbon and other pipelines in ASME B31.8 (ASME, 2007), with the only difference being the extension to the first pipeline connection within a facility in the scope of ASME B31.8.

25.2 Purpose of performing a risk assessment

Risk is formally defined as the product of the probability of an event occurring and the consequences of the event. This definition implies that risk be stated in a quantitative manner. For example, a 10% chance per year of an event causing the death of one person has a risk of 0.1 deaths per year. In the context of this chapter, a distinction is made between risk assessment and risk management or, as commonly referred to in the oil and gas pipeline industry, integrity management. The term risk assessment is appropriate for quantitative estimates of both the potential for damage and potential consequences. The basis for determining the quantitative value of risk is typically highly dependent upon qualitative descriptions that can be translated into numerical values. For example, the likelihood of earthquake-triggered slope movement sufficient to cause pipeline failure given a peak ground acceleration of 0.4g might be ranked as low, moderate, or high and subsequently assigned numerical values of 5%, 50%, and 85% for use in a risk calculation.

The goal of any risk assessment is to provide information to answer some specific questions that necessarily includes a scenario, a performance metric, and an acceptance criterion. Examples of questions with reliability scenarios include the following:

- What is the annual probability that earthquake damage (scenario) will interrupt service to customer XYZ (performance metric) for more than 24 hours (acceptance criterion)?
- What is the probability that costs (performance metric) to repair earthquake damage (scenario) will exceed \$2 000 000 over the next 15 years (acceptance criterion)?

Other categories of system performance metrics and acceptance criteria are provided in Table 25.1.

As of 2012, there are very few regulations for oil and gas pipelines that explicitly define risk acceptance criteria and require a quantitative seismic risk assessment to demonstrate compliance. Integrity management regulations in North America (PHMSA, 2011) are directed at long-term measurements of performance and gradual system upgrades and do not specify specific performance requirements. Some local agencies in the United States rely upon risk assessments for land use planning (e.g., County of Santa Barbara, 2000). Annex O of the Canadian pipeline code (CSA, 2007)

Table 25.1 System performance metrics and acceptance criteria

Acceptance criterion	System performance metrics					
	Capital loss (\$)	Revenue loss (\$)	Service interruption (% customers)	Downtime (hrs, days)	Casualties (injuries, deaths)	Lost product (volume)
Number of injuries or deaths					X	X
Duration of service interruption			X			
Number of customers served				X		
Amount of monetary loss	X	X	X	X		X
Quantity of release						X

provides an optional method that relies upon risk assessment to demonstrate suggested reliability targets have been met. More explicit reliance on risk assessments as a tool for gauging regulatory compliance are largely confined to government agencies, most prominently those agencies responsible for deciding the feasibility of recalls in the automotive and aviation sectors to correct potential safety issues. Outside the regulatory environment, the relevance of risk assessments for oil and gas pipelines is largely delegated to academic studies focused on regional or national economic and societal impacts of extreme events (e.g., O'Rourke *et al.*, 1992) and targeted studies performed for the insurance industry.

Information from the risk assessment can be used to gauge regulatory compliance, when regulations dictate specific reliability targets, but are most often used to support cost versus benefit decisions on the part of regulatory agencies or the pipeline owner. It is more common for oil and gas pipeline owners to rely upon vulnerability assessment in which some quantitative measure of the likelihood for pipeline damage is established. The information on vulnerability is primarily used to guide decisions on capital improvement projects related to increasing resiliency through pipeline replacement or redundant transmission paths. In this approach, utility personnel are responsible for relating the vulnerability to consequences, often based upon qualitative or ranking measures, to determine whether identified risks are unacceptable.

25.3 Key steps in performing risk assessments for oil and gas pipelines

Risk assessment of oil and gas pipelines involves many uncertainties. Understanding these uncertainties and their causes is required to interpret the results of the risk analysis. The analysis of uncertainties associated with data, methods, and models used to estimate the risks involved should play an important part in the overall risk analysis process.

The first step is to define the scope of the risk analysis. This involves defining the objective of the risk analysis and includes consideration of the configuration of the system being assessed, the physical and functional boundaries of the system, the environment in which the system operates, operating conditions for the system, and the technical, environmental, organizational, and human circumstances relevant to the system. It is also necessary to determine whether a qualitative or quantitative method of risk assessment is appropriate for the scope and goals of the risk analysis.

It is important to clearly define what constitutes a tolerable level of risk at the outset of the process. Defining tolerable exposure criteria early on in a site-specific risk analysis can greatly simplify the effort involved in the risk analysis as some events may be able to be quickly identified as having

no significant consequence without detailed calculation. Unfortunately, defining tolerable risk is often complicated by emotional issues related to explicitly defining an acceptable likelihood for injuries or deaths. Examples of acceptable risk levels adopted by various industries and organizations are provided in Table 25.2.

The second step is to define the potential hazards to pipelines in question and determine the likelihood of those hazards occurring. These are expected to include pipe damage from ground displacement (e.g., landslides, liquefaction, and settlement), pipe damage from third party activity, and pipe damage related to typical operational conditions (e.g., Kiefner and Trench, 2001). In many countries, regulators require oil and gas pipeline operators to report significant pipeline accidents and this information can provide the basis for estimating the likelihood of hazard occurrence. In some cases,

Table 25.2 Examples of acceptable annual probabilities

Reference	Annual exceedance probability (AEP)	1/AEP
Need for accident planning unwarranted or unnecessary except for catastrophic accident ¹	0.001000	1 000
Point for diminishing justification for reducing US Bureau of Reclamation dam failure ²	0.000100	10 000
California gas transmission pipeline incident rate (1984–2001) ³	0.000074	13 513
Building collapse from seismic load (US) ⁴	0.000040	24 750
Highest safety class for nuclear processing facilities ⁵	0.000010	100 000
DNV OS-F101 lowest failure probability for high risk of human injury ⁶		
Equivalent annual probability of earth impact by asteroid Apophis in 2036 ⁷	0.000004	250 000
Lower range of probability for core damage to existing nuclear power plants ⁸	0.000002	500 000
Estimate of lower range of probability for core damage for proposed new nuclear power plants ⁸	0.000001	1 000 000
Acceptable individual risk for student established by California Department of Education ³		

¹FEMA/DOT/EPA (1988).

²US Department of Interior (1997).

³California Department of Education (2007).

⁴ASCE 2010 (assumes Risk Category 2 and annual probability of MCE of 1/2 475).

⁵Kennedy (1992).

⁶Det Norske Veritas (2000).

⁷NASA (2009).

⁸Nuclear Energy Institute (2012).

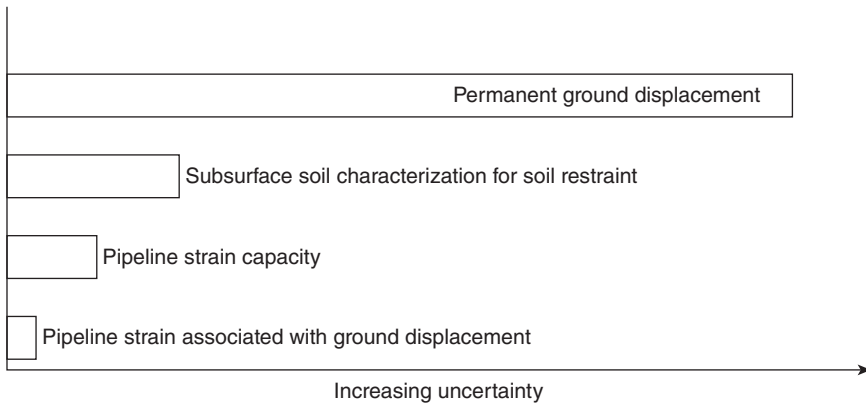
these hazards are well known. In other cases, hazards can be identified through formal methods that include hazard and operability studies, failure mode and effect analysis, fault tree analysis, and event tree analysis. Each of these methods has advantages and disadvantages associated with the ability to identify all relevant hazards, the level of complexity, and the necessary input data requirements. In many cases, a simple method that is well performed and meets the objectives and scope of the risk analysis can provide better results than that from a more sophisticated analysis. Ordinarily, the effort put into the risk analysis should be consistent with the potential level of risk being assessed. For seismic risk assessment, earthquake hazards are directly linked to a particular earthquake size, location, and style of faulting. They are either a direct hazard such as ground shaking and surface fault displacement or an indirect hazard such as triggered slope movement, liquefaction, lateral spread displacement, and post-cyclic consolidation settlement.

The third step is to estimate the level of pipeline vulnerability for each hazard event based upon an analytical assessment of the amount of stress or strain developed in the pipeline. A distinction is typically made between modest damage modes, such as leaks or holes in the pipe wall, and more catastrophic damage modes, such as a full line break. Determining the potential for unacceptable pipeline performance requires relating the likelihood of an earthquake to the severity of the earthquake hazard and then relating the severity of the earthquake hazard to pipeline response. There are several sources of significant uncertainty in determining the potential for earthquake induced pipeline damage:

- estimated earthquake-induced ground displacements which are related to:
 - earthquake recurrence rate,
 - earthquake location,
 - triggering of indirect earthquake hazards;
- subsurface soil characteristics for determining soil restraint on buried pipelines;
- pipeline strains produced by ground displacements;
- pipeline strain capacity for a particular performance level (e.g., continued operation and pressure integrity).

A qualitative comparison of the relative uncertainty in each of these parameters is illustrated in Fig. 25.1.

The fourth step in the risk assessment process involves estimating the likelihood of various consequences of pipeline damage. For natural gas pipelines, the effects of gas release typically include toxicity of released materials, such as hydrogen sulfide, thermal radiation due to ignited gas jets emanating from small holes or tears in the pipe wall, and external pressure



25.1 Qualitative depiction of relative uncertainty in key vulnerability assessment parameters.

and thermal exposure from the ignition of a gas cloud. For oil pipelines, the effects of oil release typically include thermal radiation from pool fires, property damage from inundation with oil, and environmental contamination. The assessment of consequences involves many uncertain parameters, such as those listed below:

- size of the pipe opening;
- location of the pipe opening around the pipe circumference;
- availability of ignition source;
- duration of leakage;
- vapor cloud dispersion parameters (e.g., wind speed, wind direction, and terrain roughness);
- number and location of persons near the pipe opening;
- number and location of structures near the pipe opening.

The measure of risk is defined both in terms of vulnerability and consequences (e.g., 0.01% chance per year for a fatality from thermal radiation). This risk measure is reviewed for acceptability and to identify possible actions to reduce unacceptable risk. This is normally an iterative process as the potential reduction in risk is measured by re-evaluating changes produced by the identified actions on the likelihood of a hazard occurrence, the consequences of the hazard, and combined risk.

25.4 Types of seismic hazard

In addition to the loadings under operational conditions, potential loads on buried pipelines from seismically induced hazards are of importance to assessing the performance of pipeline systems.

The typically seismic hazards to pipelines include:

- wave propagation;
- permanent ground deformation hazards:
 - differential movements at topographic discontinuities or faults,
 - slope failures,
 - liquefaction-induced permanent ground deformations;
- volcanic hazards;
- tsunami inundation.

25.4.1 Wave propagation

Seismic wave propagation is a ground motion phenomenon that relates to the passage of body waves, including compression waves and shear waves, radially from the source of earthquake energy release (hypocenter) into the surrounding rock and soil medium. Compression waves cause compressive and tensile strains in the ground in a radial direction away from the hypocenter. Shear waves cause shear strains in the ground perpendicular to these radial lines. When the compression waves and shear waves are reflected by interaction with the ground surface, surface waves (Love waves and Rayleigh waves) are generated. Except at large distances from the epicenter, the amplitudes of surface waves are much less than body waves. An earthquake at its source generates only compression and shear waves, and propagation of its radiated waves can be evaluated using ray theory (Pujol, 2003). Since the amplitude of shear waves is significantly larger than compression waves and thus generates greater strains in a pipeline, the examination of wave propagation can be limited to the effects of shear waves. Also, the strongest component of ground shaking in strong motion instrument records used to derive attenuation relationships is typically from shear waves (Bolt and Abrahamson, 2003).

A pipeline buried in soil that is subject to the passage of these seismic waves will incur longitudinal and bending strains as it conforms to the associated ground strains. In most cases, these strains are relatively small, and welded pipelines in good condition typically do not incur damage. Propagating seismic waves also give rise to hoop membrane strains and shearing strains in buried pipelines, but these strains are small and may be neglected.

25.4.2 Permanent ground deformation

Earthquake-induced permanent ground deformations have been recognized as one of the major causes of system damage and associated service disruption to lifeline facilities during past earthquakes (Hamada and O'Rourke, 1992). Essentially, large soil loads arising from permanent ground

movements can lead to potentially unacceptable strains in the pipelines. Extensive research has been focused in the last few decades to investigate the response of the pipes under permanent ground deformations. Common causes of permanent ground displacement are related to surface fault displacement, liquefaction-induced lateral spreading and flow slides, slope instability and landslides, and ground subsidence.

Ground movements at fault crossings

Pipelines crossing faults can be subjected to displacements ranging from a few centimeters to several meters. In addition to the lateral movements significant vertical movements can occur in the crossings involving reverse-thrust faults. Large ground movements at the Trans-Alaska Pipeline crossing of the Denali fault near Glennallen, Alaska, during the 2002 M7.9 Denali fault earthquake is a classic example of the type of ground movements expected at fault crossings.

Liquefaction-induced ground movements

In saturated loose or soft granular soils, liquefaction occurs when the shear strains induced due to seismic shaking cause transient pore water pressures to increase in the soil mass. As a result, intergranular contact stresses will reduce to negligible levels. In this transient state, the soil mass is subject to significant reduction in shear strength and behaves essentially as a viscous fluid that could deform or flow under gravitational or inertia forces. Areas susceptible to liquefaction could undergo significant vertical and lateral movements even in gently sloping terrain. The extent of permanent ground displacements is expected to increase with the increasing amplitude of earthquake accelerations and with the duration of seismic shaking. The extent of ground movements can be classified as: flowslides (more than ~5 m), lateral spreading flowslides (~5 m to ~0.3 m), and ground oscillation (less than ~0.3 m). In addition to liquefaction-induced ground movements, flotation and soil uplift could also be identified as another potential concern in relation to the reduction in soil strength associated with liquefaction. This would be of particular concern if the pipe trench backfill materials are poorly compacted and susceptible to liquefaction.

Liquefaction can produce overall volume changes in the liquefied soil mass that take place due to the dissipation of earthquake-shear-induced excess pore water pressures. The volume changes manifest in the field as post-liquefaction settlements, and they may occur both during and after earthquake shaking. The adverse impacts of these settlements on the performance of structural foundations and linear lifelines (such as buried pipelines and bridges) have been well recognized (Tokimatsu and Seed, 1987; Wijewickreme and Sanin, 2010).

Ground movements due to landslides

Significant ground movements could occur due to landslides triggered (without soil liquefaction) during earthquake shaking. This would become a concern in areas of steep terrain and saturated slopes with soft soils, or in areas where there is ongoing relatively slow moving landslide activity.

25.4.3 Volcanic hazards

In terms of volcanic hazards, exposure to the hazard is generally related to the type and nature of the volcano (e.g. impact of volcanic hazards tends to be greater for andesitic volcanoes than for basaltic volcanoes) and to the proximity to the volcano edifice, as well as to whether it is on a drainage that emanates from the edifice. The potential volcanic hazards that may be present along the pipeline routes include tephra (ash) fall, pyroclastic flow/surge, blast surge, lava flow, mud flow, debris flow, ground deformations, and volcanic earthquakes.

25.5 Determining hazard likelihood

To be technically complete, seismic risk assessment should consider the integrated likelihood of exceeding a specific acceptance criterion from the total range of earthquake hazards. Such an approach accounts for the cumulative likelihood associated with a large number of frequently occurring earthquakes with a small probability of producing a vulnerability. In practice, the process is often simplified by assuming a single earthquake frequency that is acceptable for a particular level of vulnerability. For example, if an acceptable level of risk is an annual probability that the pipeline will lose pressure integrity less than 0.5%, it is common practice to only address earthquake hazards that have an annual chance of being exceeded of 0.5%. This simplification allows the results of the risk assessment to be viewed in the context of other components such as buildings, dams, and bridges that are typically designed and evaluated for a single level of earthquake hazard.

The severity of earthquake ground shaking is commonly defined using probabilistic methods that include consideration of the variability in the size, recurrence interval, and location of earthquakes within a region. The other induced seismic hazards are typically defined based upon deterministic or empirical methods that may rely upon the probabilistic ground shaking estimate as an input parameter. It needs to be recognized that there is considerable uncertainty in any seismic hazard definition.

The range of uncertainty can be gauged by examining the typical uncertainty in estimating ground shaking. Considering only the uncertainty in attenuation and adopting the range of standard deviations in ground motion

estimates presented in Lee *et al.* (2000), for a predicted peak ground acceleration value of 0.5 g, the actual peak ground acceleration would lie within 0.4 g to 1.0 g, 50% of the time. The actual variation would be larger if variability other than that associated with attenuation were considered. From this simple example, it should be clear that defining levels of ground shaking to the nearest 0.05 g is adequate for most risk assessment applications. Similarly, ground displacement hazards should not be defined with artificial precision.

25.6 Determining severity of hazard

The assessment of site-specific geotechnical hazards forms an important part in overall vulnerability assessment of a pipeline system and facilitates the development of potential mitigation measures. Permanent ground deformations triggered by earthquakes have been recognized as one of the major causes of system damage and associated service disruption to life-line facilities. For example, significantly large permanent lateral deformations are expected to occur even under gently sloping ground conditions in areas of liquefiable soil. Therefore, an estimation of the extent of such ground displacements is important in the assessment of pipeline system vulnerability. In particular, adequate knowledge of site-specific soil and groundwater conditions is critical to the success of the design and installation of pipelines, as well as in predicting its anticipated performance under field conditions.

The methods available for the computation of earthquake-induced permanent lateral ground displacements can be broadly classified into: (a) empirical approaches developed based on measured displacements (e.g., Youd *et al.*, 2002); and (b) mechanistic approaches which rely more on the principles of engineering mechanics (e.g., Byrne *et al.*, 2004). Mechanistic methods mostly involve finite element or finite difference analyses, which are more appropriate for detailed site-specific analyses requiring a greater level of confidence. The estimation of earthquake-induced ground deformations, particularly from the viewpoint of regional assessments, still relies heavily on empirical correlations. It is important to note that both empirical and mechanistic approaches (or any combined approaches thereof) would require the selection of earthquake magnitude and epicentral distance that is consistent with the probabilistic seismic hazard considered in the design.

Except under high ground shaking intensity levels (i.e. in excess of peak ground acceleration of about 0.4 g), propagation of seismic waves through soft soils generally causes the ground motions to amplify as they reach the ground surface. This aspect is important and should be accounted in the liquefaction assessment and estimation of ground deformations in soft/loose soil zones.

25.6.1 Wave propagation strain

Pioneering work by Newmark (1967) still serves as the basis for estimating strains in pipelines arising from wave propagation. The Newmark approach has been incorporated, with minor modifications, into many guideline documents (ASCE, 1984; ALA, 2001a; Honegger and Nyman, 2004). The basic formulation of the Newmark approach for shear waves is provided in Equation 25.1:

$$\varepsilon = \frac{V}{\alpha C} \quad [25.1]$$

where

ε = axial pipe strain,

V = peak horizontal ground velocity,

C = apparent shear wave propagation velocity (inverse of slowness between earthquake source and site),

α = coefficient dependent upon the incidence of the wave with the pipeline (2 for strain in a pipeline from shear waves, 1 for ground strain).

The inverse of the apparent propagation velocity ($1/C$ in Equation 25.1) is termed 'slowness' in seismology and is often noted by the variable ' p '. Tables of slowness are available in regions where detailed earthquake location studies have been undertaken and the value generally ranges from 0.2 s/km to 0.5 s/km. If such tables are not available, it is conservative to adopt the propagation velocity in bedrock at depth for the apparent propagation velocity. More detailed discussion of the phenomenon of wave propagation is provided in Litehiser *et al.* (1987). Litehiser *et al.* (1987) note two important findings with respect to the application of Equation 25.1. There is a potential for amplification of strain associated with surface soil layers compared with rock of approximately 1.5 to 2. There is also the possibility of further increased amplification, on the order of two times as great as the above, when the shear response of the surface soil layers is closer to being linear (i.e., for small ground motions) as opposed to an amplification on order of 1.5 when the soil response is nonlinear (i.e., for large ground motions).

The two findings from Litehiser *et al.* (1987) are consistent with Paolucci and Smerzini (2008) who determined variations of maximum ground strain for weak to moderate ground motions (most of the data from sites with peak horizontal ground velocities far less than 30 cm/s and peak ground accelerations less than 0.4 g) that were two to three times greater than what would be obtained assuming an inverse slowness of 2000 m/s.

Based upon Equation 25.1, it is clear that wave propagation strains are exceedingly small. Considering the largest ground velocities recorded are

less than about 1.5 m/s and the lower bound value for the inverse of the slowness is 2000 m/s, and an amplification factor of 1.5 for nonlinear effects, an upper limit on the strain that can be generated from wave propagation is 0.056%. With the exception of early oxyacetylene welded pipelines and some pre-1945 pipelines fabricated with early arc welding processes (Honegger, 1999), oil and gas pipelines should be capable of tolerating longitudinal strains of 0.5%, the nominal yield strain for API 5L pipe. Assuming a longitudinal strain of 0.18% from internal pressure, the remaining longitudinal strain capacity available for oil and gas pipelines to accommodate wave propagation is nearly six times greater than what is expected to be typically generated from earthquake shaking. For this reason, wave propagation is an insignificant hazard to modern welded steel oil and gas pipelines and only need be considered for older pipelines when the axial strain capacity of the older pipelines is less than about 0.25%.

25.6.2 Site-specific estimation of permanent ground deformations

In general, for a given site, the site-specific ground displacements are estimated based on the following key steps: (a) geotechnical investigation to understand the site-specific soil and groundwater conditions; (b) assessment of site-specific loading parameters (e.g., changes to groundwater level under different weather conditions, and ground shaking intensity due to an earthquake) and ground response for the identified loading levels; (c) assessment of the geotechnical stability of the site; and (d) assessment of ground displacement hazard using empirical and/or mechanistic approaches (e.g., seismic slope stability, liquefaction-induced ground movements, and post-earthquake bearing capacity).

The deformation modulus and shear strength of liquefied soil are the key factors determining the extent of ground displacements. These parameters are often necessary for the assessment of seismic vulnerability and design of mitigation measures. Numerical approaches based on soil mechanics principles provide a means to estimate displacement patterns, but they often suffer from a lack of rigorous calibration with actual earthquake data.

25.6.3 Regional assessments of permanent ground deformations

When it is necessary to assess the seismic vulnerability of a wide-area pipeline networks, regional approaches are required to determine the ground displacement hazards. The approach proposed by Youd and Perkins (1987) provides a general method for mapping the liquefaction susceptibility based

on the geological characteristics of a given area. Youd and Perkins (1987) have defined 'liquefaction susceptibility' as the capacity of the soil to resist liquefaction. They suggested that the age of the deposit, relative density, particle size, and depth to groundwater table are the primary factors that influence the liquefaction susceptibility. This approach provides a convenient and effective method of assessing the liquefaction susceptibility of a large area for a regional zonation study, where general surficial geological data are available but site specific data are limited.

The empirical methods have been mainly developed based on statistical/regression analyses of seismic, topographical, soils, and geological data associated with lateral spreads resulting from major earthquakes. Empirical methods are also unable to estimate ground displacement patterns on the surface and at different depths; therefore, they cannot account for the presence of man-made site features (i.e., the effects arising from zones of ground improvement). While the empirical models provide a convenient method to estimate liquefaction-induced ground displacements, use of the results should be made with a proper understanding of the limitations of the model, as well as the uncertainties in the input parameters and the model. Although such an approach is considered reasonable for regional pipeline study, more detailed site-specific analyses should be considered for locally identified risk areas. Moreover, even with the advantage of being based on an actual database of ground displacements while accounting for many physical parameters governing the 'free-field' ground displacements, empirical methods do not incorporate the deformation modulus and shear strength of liquefied soil in the computations.

25.6.4 Ground displacements due to landslides

In the case of site-specific assessments, the estimation of ground displacements due to earthquake-induced landslides (without liquefaction) is often undertaken using limit equilibrium-based stability analysis followed by Newmark's sliding-block approach (Newmark, 1965). This method can be also applied regionally. In these assessments, again, it is important to establish the material strengths, liquefaction susceptibility, and groundwater conditions.

25.6.5 Vertical ground displacements due to post-earthquake pore water pressure dissipation

Besides the lateral spread displacements occurring during and after an earthquake, saturated loose sands can settle (vertically deform) as the earthquake-induced excess pore pressures dissipate (typically called post-liquefaction settlements). Although not a major concern in pipelines

located in generally flat areas, these settlements need careful attention in some situations. For example, plant structures, compressor stations, etc., located in soft/loose soil zones may require pile-supported foundations. Under such conditions, there is a potential for post-liquefaction settlements leading to major differential settlements between pile-supported structures and pipelines. Thus, provision should be made for adequate flexibility of pipeline to minimize potential pipeline breakage due to differential settlements.

Studies conducted to evaluate volume change after cyclic loading have shown that in general, the factors controlling post-cyclic settlements in sands are the degree of excess pore water pressure generation and the induced cyclic shear strain. Owing to the direct connection with excess pore water pressure development, potential for volumetric strains has been often linked with the field density. Several simplified methods have been proposed to estimate settlements of soils knowing the field penetration resistance (i.e., standard penetration resistance N -value or cone penetration testing resistance) or laboratory cyclic shear resistance and the cyclic stress ratio corresponding to the level of ground shaking being considered (e.g., Tokimatsu and Seed, 1987; Wu, 2002; Wijewickreme and Sanin, 2010).

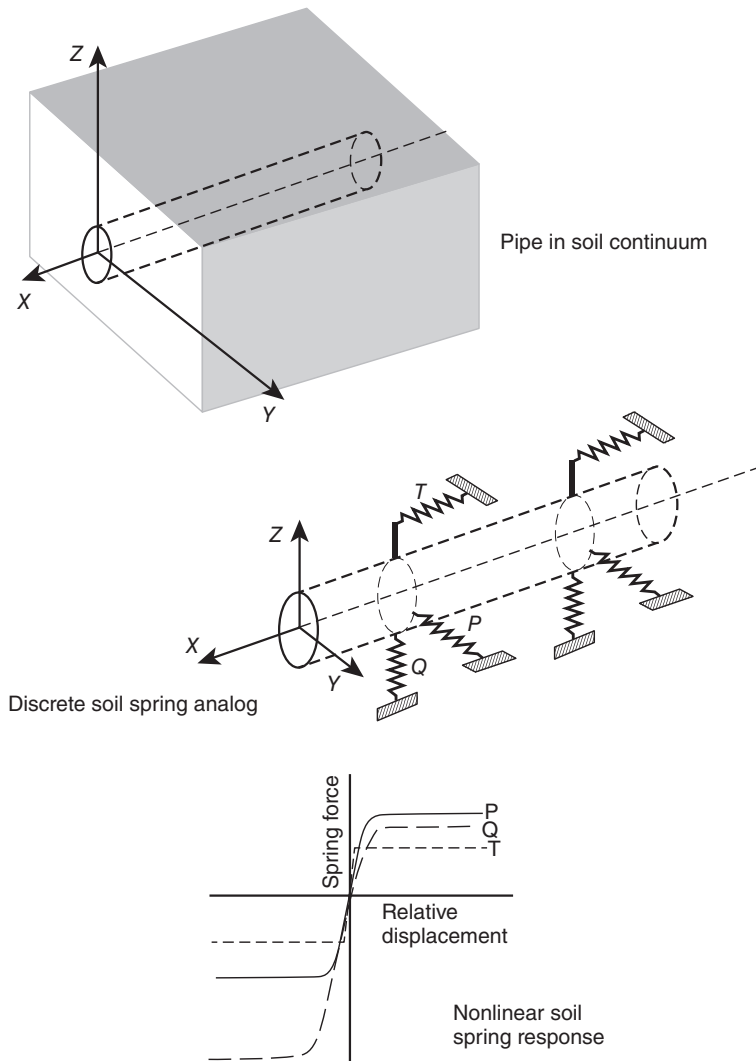
Much larger vertical movements are expected at river crossings, in the vicinity of dykes, ditches, road embankments, etc., due to distortion of the soil mass. Estimation of such vertical deformations would require rigorous site-specific analyses.

25.7 Pipeline response to earthquake hazards

The analytical assessment of buried pipeline response to permanent ground displacement dates back to the mid-1970s. Current recommendations on the methodology for assessing pipelines for ground displacement are contained in PRCI guidelines (Honegger and Nyman, 2004; PRCI, 2009). Non-linear finite element techniques are the only practical means available to analyze all but the most simple of problems. Methods relating generic ground displacement values to rates of pipeline damage, such as those incorporated into HAZUS risk assessment software (FEMA, 2011) and presented in American Lifelines Alliance guidelines (ALA, 2001b), are totally inappropriate for assessing oil and gas pipeline damage.

Finite element approaches provide a means to investigate the effects of changes in backfill characteristics, pipeline material, wall thickness, and pipeline alignment. The mechanics of implementing a finite element analysis have remained largely the same over the last 30 to 40 years. The primary advancement in performing such analyses is the availability of powerful desktop personal computers and compatible nonlinear analysis software that accounts for material yielding and large deformations.

The pipeline is modeled with pipe elements that are essentially beam elements that are capable of accounting for internal pressure effects. Soil restraint is modeled with non-linear spring elements that act independently in the axial, horizontal, and vertical directions relative to the axis of the pipeline as illustrated schematically in Fig. 25.2. Ground displacement induced by earthquakes is modeled as displacements applied to the base of the soil springs. There are no restrictions on the analysis software that can be used as long as it is capable of capturing the nonlinear behaviour of soil



25.2 Representation of pipe-soil interaction using nonlinear spring elements.

springs, user-defined stress–strain curves for the pipe material, and large changes in pipeline geometry. The analysis software should include a pipe element in its element library with the capability to model internal pressure and provide output at various circumferential locations.

The length of the pipeline model should be sufficient to adequately capture the anchoring effects of the soil outside the zone of ground movement. The pipe element length in regions where the pipe strain is expected to exceed the yield strain (typically at abrupt transitions in ground displacement or locations with abrupt changes in soil restraint such as elbows) generally should not exceed one pipe diameter. The one-diameter rule is related to the fact that a gauge length of approximately one pipe diameter was the basis for reporting strains in tests used to establish strain acceptance criteria.

Available methods for modeling soil restraint with soil springs assumes that the spring forces always act in the axial, horizontal, and vertical directions relative to the pipeline. In most analyses, the soil springs are defined in a global coordinate system. As a result of this, in the direction of the soil spring forces do not maintain an axial, horizontal, and vertical orientation relative to the pipeline if the pipeline undergoes large rotations. However, the error introduced by this misalignment is acceptable considering other assumptions and uncertainties inherent in the analysis.

In a case where loading and unloading of the soil springs occur, unloading characteristics of the soil springs need to be modeled. Relative pipe–soil displacements are permanent in a sense that the soil does not ‘spring back’. Therefore, soil spring forces should quickly drop to zero or unload along a path parallel to the initial soil spring stiffness.

There is considerable uncertainty regarding the relationships used to compute soil spring properties. Most of this uncertainty is related to estimates of the soil strength parameters. The uncertainty in estimating soil strength parameters for pipeline analyses in fine-grained soils is further complicated by the fact that pipelines are typically located above the water table and within the desiccation zone of the soil. The strength of partially saturated desiccated soils is not well defined in soil mechanics.

Current analysis techniques assume the equivalent soil springs act independently, a common assumption for analytical representation of pile foundations and similar buried structures. This assumption can introduce errors related to the potential for different soil restraints for oblique soil displacement relative to the pipe (e.g., a combination of horizontal and vertical displacement) and the dependency on soil spring force at a particular point along the pipeline on adjacent relative pipe–soil displacement. The error associated with the assumption of independent soil springs is generally small relative to the overall uncertainty typically associated with defining the soil strength properties in the equivalent spring formulations.

For pipeline load conditions produced by an imposed displacement, relatively large strains can be accepted provided the pipeline is in good condition and the girth welds are capable of developing gross-section yielding of the pipe. For situations where the pipeline is subjected to high longitudinal compression strains from a combination of axial and bending loads, there is a potential for the development of local buckling of the pipe wall.

While continuum analyses hold the promise of eliminating many of the simplistic representations of soil–pipe interaction using pipe elements and soil springs, several significant obstacles remain to be overcome before continuum analysis methods can be considered superior to pipe element and soil spring representations for routine engineering applications:

- It is typically necessary to represent several hundred meters of pipeline in the analytical model which results in an unwieldy large model that may take days to run using normally available computational resources.
- The complexity of the model makes it much more difficult to extract results of interest and incorporate relatively minor changes (e.g., change of soil cover or pipe diameter).
- The ability to model large relative displacements is often not possible without an efficient means to reformulate the model mesh.
- Continuum models are generally not capable of capturing flow and fracture behavior and the development of slip planes in the soil surrounding the pipeline.
- Modeling of the soil in a continuum models requires soil properties not normally available and often requires calibration of the analytical material model.
- The fidelity of results from continuum models is likely to be no greater than what is obtained from simple soil spring models, since available test data that could provide a basis for validation do not include information on the state of stress within the soil and at the pipe–soil interface.

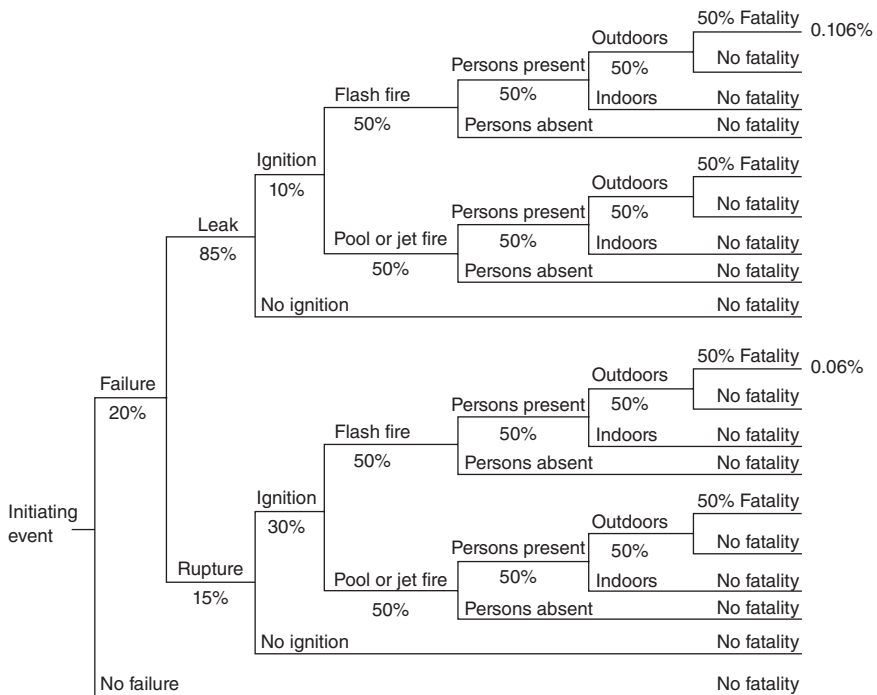
25.8 Consequences of pipeline damage

The protocol developed for the California Department of Education (2007) provides a good overview of a relatively simple consequence assessment process based upon the ALOHA program that is generally available to the public (NOAA, 2011). Consequences of damage to an oil or gas pipeline are typically related to exposure to thermal radiation due to a pool fire, jet fire, or flash fire from a vapor cloud ignition. The consideration of explosion from natural gas release is highly unlikely unless unique conditions exist that would allow accumulation of gas near the ground without ignition, confinement of the gas cloud, and presence of an ignition source in the zone

of confinement. Meteorological conditions (wind direction, wind speed, and atmospheric stability) play an important role in evaluating consequences as they directly impact the direction of gas dispersion, the air mixing that can dilute the natural gas to a point where ignition cannot happen, and the radiated heat distribution patterns from a subsequent fire. Seasonal conditions also affect the potential threat posed by ignition of brush.

Measures of consequences are generally established as a percentage of the lower flammable limit for vapor clouds or the likelihood of exposure to serious thermal radiation. These consequence measures can be linked with a specified likelihood for individual mortality. Flammable vapor clouds are often conservatively considered to pose a risk of ignition and mortality from flash fire if the concentration of flammable vapor exceeds 30–50% of the lower flammability limit. Onset of serious injury from thermal radiation is often considered to occur for exposures exceeding 16000–18000 kW/m² which corresponds to roughly a 1–15% chance of mortality.

A fault tree or event tree approach is commonly used to determine the likelihood of exceeding exposure limits for flash fire or thermal radiation. A simple representative event tree is illustrated in Figure 25.3 for a single



25.3 Example of an event tree assessing likelihood of a fatality from thermal exposure.

initiating event (e.g., slope failure) leading to a fatality. Using an event tree or fault tree approach requires assigning likelihoods to each branch of the tree along the event path of interest, as shown in Fig. 25.3.

25.8.1 Relating computed strain to loss of pressure integrity

For a seismic risk assessment, pipeline failure is related to the exceedance of a specified longitudinal strain limit, typically 2–4% for relatively modern (post-1960) welded steel oil and gas pipelines. One or more failure branches may be necessary to differentiate the consequences from seismic hazards that can produce different probabilities of leak versus rupture. For example, damage from large fault displacement or lateral spread displacement is much more likely to result in rupture than small displacements associated with liquefaction settlement.

25.8.2 Leak versus rupture

In general, opinions on pipeline failure of full bore pipeline rupture given exceedance of an ultimate strain limit range from 10% to 40%. For seismic risk assessment, the potential for full rupture should not be taken below 20% and be biased toward a higher likelihood based upon the degree to which the seismic ground displacements exceed the estimated pipeline displacement capacity.

25.8.3 Ignition likelihood

The likelihood for ignition is largely a function of the built environment near the pipeline. Ignition is more likely for pipelines located within the right-of-way of high-voltage transmission pipelines or urban areas with above-ground electrical lines. Opinions of ignition range from 10% to over 30%, depending upon these factors and the amount of flammable that can potentially be released.

25.8.4 Fire hazard from leak

The exposure to a flash fire from a pipeline leak is dependent upon the size of the leak which is generally assumed to be equivalent to a hole with a diameter of 25–50 mm. In addition to the rate of material release from a leak, the orientation of the leak can significantly alter the exposure for high-pressure pipelines. Caution is warranted when using simple programs such as ALOHA which assume vertical release from pipelines. Proprietary

software capable of more refined modeling of leak conditions may be necessary to capture the effects of impeded flow or influence of jet direction on thermal exposure.

25.9 Mitigation approaches to reduce risk to pipelines

Upon identification of the geotechnical hazards and the resulting vulnerability of a given structure, a combination of structural retrofitting and/or geotechnical remediation (ground improvement) is often considered in the design of mitigation measures. In general, there are four options to improve the performance of a given pipeline against an identified geotechnical hazard: (a) avoid the hazard by relocation; (b) isolate the pipeline from the hazard; (c) accommodate the hazard by strengthening the pipeline or increasing flexibility; and (d) mitigate the hazard using ground improvement. Although avoiding the hazard by relocation is the most effective approach, this option is often not attractive because of prohibitive costs associated with acquisition of pipeline right-of-way for realignment.

The potential for pipeline failure can be reduced by reducing exposure of the pipeline to seismic hazard. Other than rerouting the pipeline to avoid the hazard, there are two methods to reduce pipeline exposure. Horizontal directional drilling techniques can be used to locate the pipeline below the zone of ground displacement. This technique is most commonly used to avoid lateral spread hazards at river crossings. In rare instances, aerial crossings can be used to avoid hazards of limited size. Isolation of pipeline from geotechnical hazard is also considered favorable in certain situations. Use of isolation culverts, or above-ground supports, provide effective means of isolating pipelines from ground movement hazards. The idea herein is to provide a mechanism for the ground to ‘slide past’ or ‘slide below’ the pipeline using a sliding support system. The above-ground isolation structure specifically designed to protect the Trans-Alaska Pipeline crossing of the Denali fault performed successfully during the 2002 M7.9 Denali fault earthquake, confirming the suitability of isolation measures against geotechnical hazards. Moreover, soil restraint acting on the pipelines can be reduced by careful selection of pipeline trench geometry and backfill material, low-friction pipeline coatings, and wrapping pipeline with two layers of geotextile fabric, or placing a portion of the pipeline on the ground surface.

The potential for pipeline failure can also be reduced by increasing the pipeline wall thickness and material strength and also modifying the pipeline alignment to provide a more beneficial angle between the direction of ground displacement and the pipeline axis. The latter option is generally only possible during the design of new pipelines because of extreme difficulties in obtaining new right-of-way for existing pipelines.

Ground improvement is emerging as one of the widely adopted mitigation measures to reduce the impact of earthquake-related ground displacement hazards. In mitigation works, the design philosophy often revolves around implementing ground improvement measures to limit deformations in a given pipeline to acceptable levels (e.g., design to minimize the loss of pressure integrity in pipelines). Observations following major earthquake events have indicated that sites with improved ground had performed well during earthquakes (Mitchell *et al.*, 1995).

When ground improvement is considered to be the desired option, the selection of the most suitable remedial option is governed by many factors including, but not limited to: soil conditions, space restrictions, issues related to the protection of existing structures during ground improvement, operational constraints, environmental regulatory requirements, and land availability. Historically, ground improvement has been used as a means of improving the post-construction bearing capacity and settlement performance of soils under static loading conditions, and a variety of ground improvement techniques have evolved in the past few decades. In addition to resisting static loads, some of the ground improvement measures have been effectively used to retrofit facilities that are located within, or that have foundations supported on liquefiable soils. These measures include dynamic deep compaction, vibro-replacement using stone columns, compaction piling, explosive compaction, and compaction grouting.

The method of vibro-replacement using stone columns is the most preferred technique of ground improvement in sandy soils. The method can be effectively used to densify soils within about 25 m below existing ground level (see Fig. 25.4). The method is attractive because of the potential availability of drainage through stone columns for the dissipation of excess pore water pressures in addition to the densification effect. Compaction grouting is a useful tool not only in fine-grained soils, but also in improving sites that have physical constraints such as low headroom. Deep dynamic compaction is a viable means of improving the settlement characteristics and liquefaction resistance of random fills and alluvial soils that are in a state of loose relative density and difficult for a probe to penetrate through. *In-situ* verification using penetration resistance measurements confirm that this method can be used to a maximum depth of about 10–12 m below existing ground level. Below this depth, the achieved improvement in penetration resistance diminishes considerably.

Detailed site-specific studies are required to quantify potential for pipeline damage, and to determine whether or not practical alternatives exist to reduce the risk. Development of site-specific recommendations requires careful consideration of many factors including site geology, environmental conditions, pipeline response characteristics, and system performance requirements. The ground improvement configurations used in practice are



25.4 Ground densification using vibro-replacement at a river bank in Vancouver, British Columbia, Canada, to reduce the risk of liquefaction-induced lateral ground displacements (Wijewickreme *et al.*, 2005).

clearly dependent on geotechnical risks that are to be mitigated, and these configurations essentially fall into one of the following two categories: (i) in-ground densified barrier(s) aligned perpendicular to the direction of ground movement to reduce liquefaction induced lateral spreading; and (ii) densification of wide-area footprints beneath and/or around foundation footprints to improve bearing capacity and to reduce the impacts from lateral spreading.

Verification testing for quality control forms a key component in undertaking ground improvement works. Evaluation of the treated soil type, method of ground improvement, and site constraints are required in selecting the parameters and testing tools to assess the conformance of ground improvement to specified criteria. In addition, effects from parameters that affect the soil behavior (e.g., aging and pore pressure dissipation) can have significant influence on the observations from verification testing, and they should be carefully evaluated in determining the acceptability of a given ground improvement.

The need to protect adjacent existing pipelines/structures is often a key consideration during ground improvement. Thus systematic monitoring of existing facilities during ground improvement is essential. Furthermore, structural evaluation of the performance of structures based on data from monitoring as well as modification of ground improvement methodology and configurations to meet the constraints are generally required.

Prediction of the anticipated geotechnical hazard and liquefaction-induced permanent ground displacements are critical considerations in the design of remedial measures. These predictions are often undertaken using approaches combining both numerical and empirical methods.

25.10 Challenges and issues

25.10.1 Pipeline strain versus size of rupture

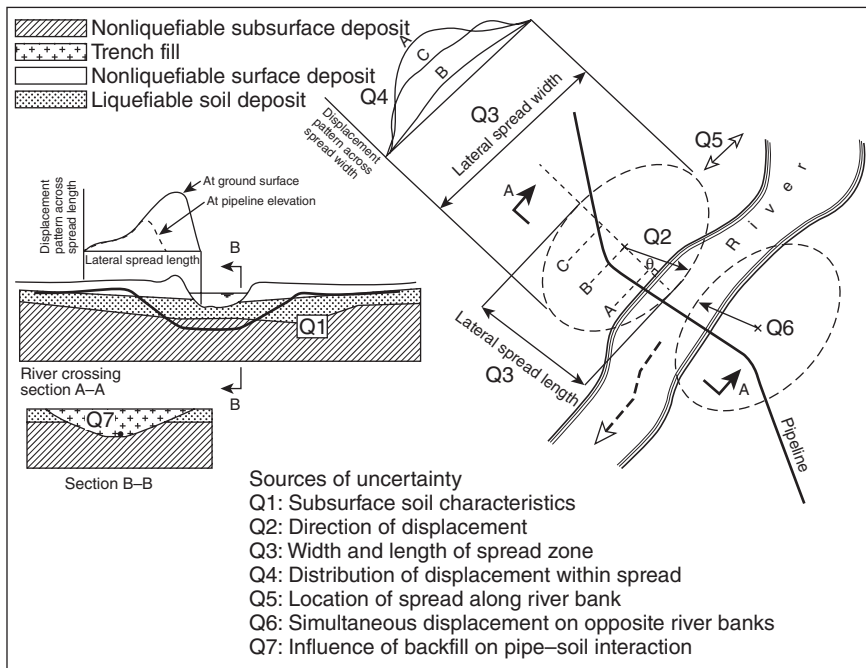
Establishing pipeline strain criteria that relate pipeline strain to a probability for loss of pressure integrity is an ongoing area of active research (PRCI, 2011). For new pipelines, establishing acceptable tension strain limits is often based upon the results of full-scale or wide-plate testing of girth welds with or without machined flaws to represent defects that may be missed by weld inspection procedures. However, the number of tests performed is generally not sufficient to provide a statistical relationship between tension strain and failure probability. There are no generally accepted methods for relating pipeline strain to the severity of the pipeline failure mode. As a result, establishing strain criteria must be based upon expert judgment.

25.10.2 Geotechnical constraints

The available data on soil and groundwater conditions along pipeline alignments are typically limited. As a result, characterization of sites for soil–pipe interaction often becomes a challenging task. In addition, high variability of soil conditions along pipeline alignments combined with complexities associated with the mechanical behavior of soil (e.g., stress and strain level dependence, effects of particle fabric, effects of loading paths.) present additional constraints to the level of accuracy attainable in seismic performance evaluations. With respect to lateral spread displacement hazards, some of the basic uncertainties in defining the hazard are illustrated in Fig. 25.5. Sensitivity of the geotechnical hazard estimates to the above considerations along with the uncertainties arising due to the difficulties in accurately defining earthquake shaking (i.e., ground acceleration time histories) should be kept in mind when interpreting outcomes from hazard assessments.

25.10.3 Condition of older pipelines

A key difficulty in assessing the vulnerability of existing pipelines is the uncertainty in estimating the tension strain capacity of the existing girth welds. Compression strain capacity is governed by buckling of the pipe wall and is not highly dependent upon girth weld quality. For older pipelines,



25.5 Uncertainties associated with quantifying lateral spread displacement hazards.

there will not be sufficient information on the severity or prevalence of girth weld defects that could reduce pipeline tension strain capacity. This difficulty is normally addressed by simply assuming a tension strain capacity based upon assumed flaw severity. Another alternative, if portions of the pipeline are removed to provide some pipe weld information, is to assume the removed pipe is identical to the remaining pipe and assign a strain capacity based upon a fitness for purpose assessment. API 1101 Appendix A (API, 2010) and API 579 (API, 2007) provide methodologies for defining girth weld strain capacity, but these methods can be extremely conservative and require special knowledge and experience to implement correctly.

25.10.4 Acceptable risk

As risk is defined as the product of vulnerability and consequences, a 0.01% (1 in 10000) chance per year of an event causing 1000 deaths is equivalent to a 10% chance per year of an event causing the death of one person. Therefore, risk alone is generally insufficient for making decisions where the general public is involved because it is extremely difficult to educate the public to accept serious consequences, such as death. Conversely, the

public is more accepting of severe consequences for events that are deemed sufficiently rare as to be considered ‘acts of God’. Thus, there is more outrage expressed if a pipeline fails under normal operating pressure than if it fails as a result of a landslide or an extreme flood event, even if the annual likelihood of failure for the three causes is the same.

The difficulty in conveying risk-based decisions to the public at large is often ameliorated by an official governmental regulatory process in which specific risk-based acceptance criteria are developed (e.g., ASCE, 2010), often with the advice and input from experts. That is not to imply that the regulatory requirements are always based on a rational interpretation or assessment of risk. As with any governmental regulatory action, the process can be subverted by individuals and organizations acting in their own best interests. The benefit of governmental regulations establishing risk acceptance criteria is primarily that the regulations can impose uniformity on those performing the risk assessments.

25.10.5 Risk-based code provisions

At this time, there are no code requirements on the level of reliability that is required for the design of oil and gas pipelines. Work sponsored by the Pipeline Research Council International (PRCI) has led to a set of guidelines for implementing a reliability-based design framework with a recommendation with respect to performance goals (Nessim and Zhou, 2005). This work has been largely incorporated into the non-mandatory Annex O of CSA Z662. Those referring to Annex O should be aware that the reliability limits contained therein are only recommendations and not requirements. Considerable caution should be taken in considering adoption of the reliability limits in Annex O for several reasons:

- The Annex O reliability targets were developed using the reliability under normal operating conditions of the population of existing pipelines as a baseline value of acceptable risk. A more appropriate approach would have been to base non-seismic reliability targets on the minimum performance allowed in pipeline codes (ASME, 2007).
- The Annex O reliability targets are based upon pipeline diameter, pressure, and population density and typically require an annual likelihood of pipeline failure in the range of $1/150\,000$ to $1/200\,000$ which is unreasonable compared with other reliability targets where the potential for death and injury is far greater than what exists for oil and gas pipelines.
- Development of the reliability targets did not address the difference in performance expectations for extreme rare events. This is especially relevant to seismic risk assessment. For example, design requirements

for typical buildings in the 2010 version of standard ASCE 7 are based upon a target annual probability of sudden progressive failure of $7/10\,000\,000$ for non-seismic loading and $1/24\,750$ for seismic loading (ASCE, 2010).

As an alternative to the PRCI reliability targets, consideration could be given to adopting an acceptable rate of pipeline failure from seismic loading be no greater than $1/24\,750$, the same as the targeted annual probability of building collapse from earthquake loading adopted in current US building standards (ASCE, 2010).

25.11 Future trends

25.11.1 Numerical modeling

The use of numerical modeling, in general, allows the effects of a wide range of variables to be assessed in a timely and efficient manner. The data generated from physical testing work would be of great applicability to calibrate and verify the numerical models. Despite the difficulties of attempting to use for routine engineering applications, continuum modeling methods have value in increasing the understanding of the general soil–pipe interaction problems, particularly with respect to improved definition of equivalent soil springs. One example in this regard is the need to understand the response of pipelines subject to ground movements that are oblique to the pipeline alignment; clearly, 3-dimensional continuum modeling becomes valuable in the modeling of such problems that cannot be captured in a 2-dimensional analysis. It also appears that discrete element methods seem to have a role to play in modeling complex soil–pipe interaction situations, especially when the particulate nature of soil seems to be the major governing factor of the observed response (e.g., modeling of the effectiveness of using geotextile interfaces, or coarse-grained trench backfills, in reducing soil loads on pipelines).

A vast amount of numerical analysis has also been performed to comprehend the complex interactions between pipe and soil (e.g., Rowe and Davis, 1982; Guo and Stolle, 2005). Work in this area is expected to continue.

25.11.2 Analytical approaches to capture nonlinear pipe material behavior of plastic pipelines

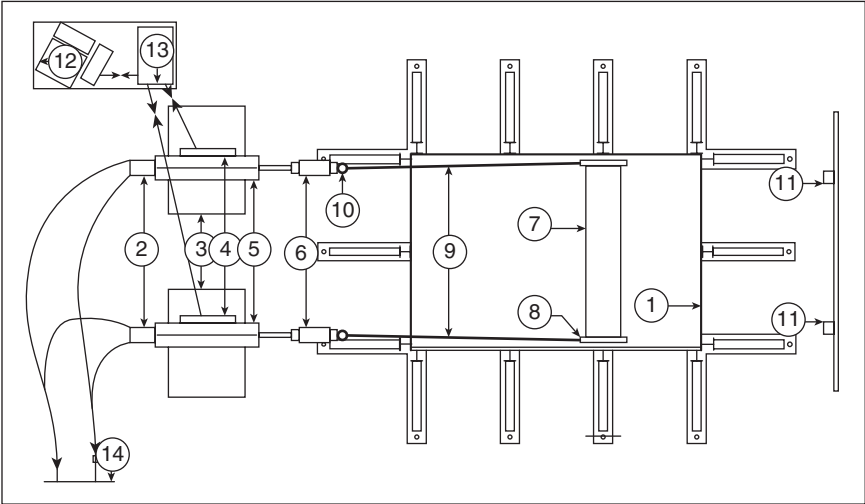
Since its introduction in the late 1960s, polyethylene (PE) pipes (either high-density polyethylene (HDPE) or medium density polyethylene (MDPE)) have become increasingly popular in natural gas distribution

networks due to their lower material, installation and maintenance costs, corrosion resistance, lower friction at the interface, lightweight, and claimed greater capacity to accommodate displacements than its counterpart, steel pipes. The use of MDPE pipes takes advantage of having higher flexibility and fracture toughness while having comparable long-term strength and stiffness to that of HDPE (Stewart *et al.*, 1999).

Over the years, numerous studies have been performed on steel pipes to characterize the behavior when the pipes are subjected to ground movement. However, reported experimental research on the response of buried PE pipe systems subject to ground movement is very limited. Considering the relatively smaller deformation stiffness and time-dependent and non-linear stress–strain response (viscoelastic and creep behavior) of PE pipe material in comparison to steel, there is likelihood for significant limitations to arise when methods developed for steel pipes are used in evaluating the response of PE pipes. Clearly, data from controlled experimental work on pipelines subject to axial movement, particularly at full-scale level, is needed to advance the knowledge of the response of buried PE pipe systems subject to ground movement. With this background, a number of research programs have been already undertaken to investigate the response of PE pipe systems under permanent ground movements and analytical methods have been developed to account for the mobilization of soil loads in buried PE pipes under such ground movements (Weerasekara and Wijewickreme, 2008).

25.11.3 Full-scale model testing

Modeling of chosen full-scale pipeline configurations in the laboratory provides a very attractive way of capturing and understanding the complexities associated with soil–pipe interaction. Due to the large number of variables, full-scale testing provides a meaningful approach to characterize soil-springs for pipe–soil interaction modeling (i.e., provides a meaningful approach to estimate parameters of soil-springs in axial, lateral, and upward directions to model the interaction between pipe and soil). Physical models simulating the field situations also play a key role in calibrating and validating the analytical approaches and numerical models. (Note: tests in smaller scales, however, may be subjected to errors associated with scaling.) Some of the large soil chambers for full-scale testing of pipe–soil interaction problems are available at Cornell University (Trautmann and O'Rourke, 1983), Center for Cold Oceans Resources Engineering (Paulin *et al.*, 1997), Queen's University (Moore and Brachman, 1994), and University of British Columbia (Wijewickreme *et al.*, 2009; see Figs 25.6 and 25.7 for details related to the University of British Columbia (UBC) test facility).



Test system components

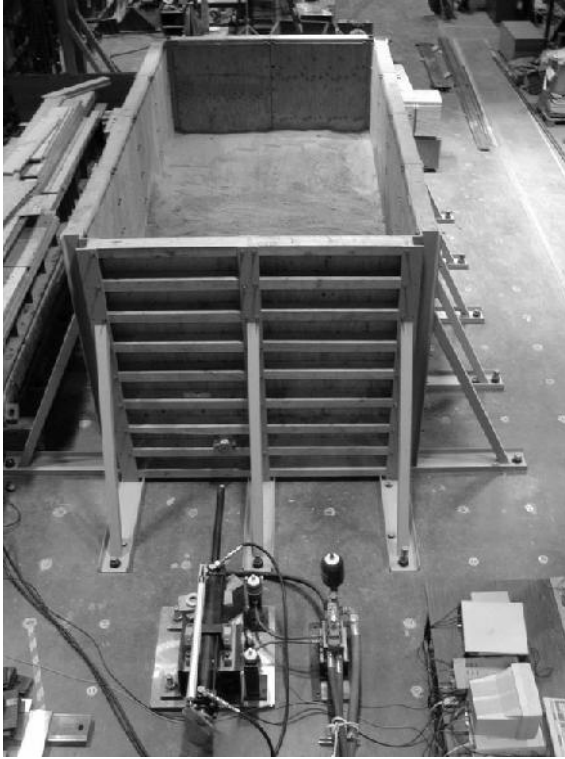
No.	Description	No.	Description
1	Soil test chamber (3.78 m x 2.5 m)	8	3 bolts end clamps
2	Servo controller	9	1 1/8" steel cables
3	Pedestal	10	Shackles
4	Linear voltage displacement transducer	11	String potentiometers
5	Hydraulic actuator	12	Data acquisition system & computer
6	Load cell	13	Control system
7	Steel pipe (2.4 m long)	14	To hydraulic power

25.6 Layout of Physical Modeling Facility at UBC (set-up for lateral soil-pipe restraint testing) and test components.

25.11.4 Field monitoring and testing of pipeline performance

Field monitoring of the performance of pipelines, or soil mass in the vicinity of pipelines, has the potential to play an important role with respect to several key aspects of pipeline geotechnical engineering including, but not limited to the following:

- assessment of potential imminent pipeline concerns;
- identification of pipeline performance in areas of less understood, varying, soil and groundwater conditions;
- development of an understanding of field variations that are not necessarily captured in general design works using existing guidelines;



25.7 View of Physical Modeling Facility at UBC (set-up for axial soil-pipe restraint testing).

- development of real-life data for validation of outcomes from numerical modeling of pipe–soil interaction problems;
- furthering the understanding of observations from physical modeling.

Under certain circumstances, continuous field monitoring of critical pipeline sections becomes valuable. This may become essential in situations where numerical modeling is complicated due to lack of knowledge in representing the slide fronts and due to difficulties in determining the actual pipe–soil interaction taking place because of varying soil conditions. Soil constitutive laws, boundary conditions, and the assumptions in initial soil stress state can significantly affect the results of numerical modeling. However, in remote areas or in zones where the ground movement is spread over a wide region, the continuous monitoring of the pipeline would become costly and unfeasible. An advantage in field monitoring is that they could be conducted over longer periods, without being subjected to space–time constraints that are more common in laboratory environments.

25.12 Conclusions

The preceding discussion provides a general overview of key aspects of quantitative seismic risk assessment for oil or gas pipelines. As is hopefully apparent from this discussion, the process requires numerous assumptions regarding many poorly defined or highly variable input parameters and models. Therefore, specific results can be highly dependent upon the professional qualifications and experience of the personnel making the assumptions and conducting the analyses. This level of uncertainty is considered to be one of the main reasons why explicit risk acceptance criteria have not been directly incorporated regulatory requirements. Those undertaking or contemplating facilitating decisions on oil and gas pipelines based upon the results from a seismic risk assessment should exercise caution in giving undue credence to numerical risk values. Given the current state of knowledge, estimates of risk within at least a factor of two can be considered 'equal'.

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Seismic risk analysis of wind turbine support structures

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Abstract: Probabilistic models are developed to predict deformation, shear, and moment demands on wind turbine support structures subject to seismic excitations, and environmental (wind, wave, and current) and operational loadings. The probabilistic models are formulated starting from existing deterministic models and developing correction terms that capture the inherent bias. The correction terms and a model error are assessed using data obtained from detailed three-dimensional nonlinear finite element analyses of a set of wind turbine systems that consider different design parameters and account for the dynamic soil–structure interaction. The proposed probabilistic seismic demand models provide unbiased predictions of the seismic demand on support structures and properly account for the underlying uncertainties. The developed demand models are used to compute fragility estimates of an example support structure defined as the conditional probability of not meeting specified capacity levels.

Key words: Bayesian inference, offshore wind turbines, demand models, experimental data, fragility, probabilistic models.

26.1 Introduction

Extensive installation of offshore wind farms for electricity production in moderate to high seismic regions in the United States and other countries has raised a new concern about the safety of wind turbine support structures subject to seismic loads (Prowell and Veers, 2009). The demands on the support structure of wind turbines are typically determined using computational models. Computational analyses require accurate modeling of the structural dynamic response of the support structure as well as evaluation and modeling of the environmental loading including wind, and for offshore wind turbines, wave and current loading. Several aeroelastic simulation codes are used in the industry to simulate fatigue, aerodynamics, structural dynamic response, and turbulence. They are mostly used to predict the extreme and fatigue loads of wind turbines. For example, FAST (Jonkman and Buhl, 2005) and ADAMS (Laino and Hansen, 2001) are two

commonly used simulators. The main limitation of these simulators is that they are not capable of modeling the nonlinear foundation behavior and the dynamic soil–structure interaction.

Mono-piles are widely used to resist lateral loads and bending moments resulting from the operation of the turbine as well as environmental loading. Common methods for the analysis of laterally loaded single piles in general are based on Winkler (elastic) foundation models, or continuous models. Winkler foundation models are popular because of their simplicity and reasonable accuracy. For nonlinear analyses, the p - y method, developed first by Matlock (1970), is the most commonly used one. A foundation specifically for wind turbines has been modeled utilizing equivalent springs that uses a design method based on p - y curves (Reese *et al.*, 1975) to take soil–structure interaction into account. However, using a three-dimensional (3D) nonlinear finite element model, Mardfekri *et al.* (2013) showed that, depending on the pile diameter and soil type, using common simple models, such as the p - y method and particularly modeling the pile using one dimensional beam-column elements may result in inaccurate responses. This is true in particular for the pile sizes typical of foundations of offshore wind turbines.

Mardfekri and Gardoni (2012) assessed the deformation, shear, and moment demands on the support structure of offshore wind turbines subject to wind, wave, current, and turbine operational loadings properly accounting for the nonlinear soil–structure interaction. Mardfekri and Gardoni (2012) considered a continuous modeling of the pile and the surrounding soil using a 3D nonlinear finite element analysis in which both the soil and the pile were modeled with 3D finite elements. Because a complete nonlinear 3D finite element analysis can be quite expensive and time consuming, Mardfekri and Gardoni (2012) also proposed simplified probabilistic demand models for the deformation, shear, and moment demands on the support structure of offshore wind turbines subject to wind, wave, current, and turbine operational loadings. The probabilistic models were calibrated using the results from the nonlinear 3D finite element analyses and properly accounted for the inherent uncertainties, including the statistical uncertainty (associated with the finite sample size) and the modeling errors (associate with the selection of the variables in the models and the model forms).

With the extensive installation of wind farms in moderate and high seismic regions, it is important to consider also seismic loading. This chapter develops novel deformation, shear and moment demand models for the support structure of offshore wind turbines subject to seismic loading in addition to wind, wave, current, and turbine operational loadings. The next section discusses the general formulation of probabilistic demand models following Gardoni *et al.* (2002, 2003). Then, we develop the deformation,

shear and moment demand models for the support structure of offshore wind turbines. The approach is consistent with the work by Mardfekri and Gardoni (2012) and emphasis is given to the additional seismic load. Finally, fragility estimates are presented for an example offshore wind turbine support structure for given values of the mean wind speed, significant wave height, and spectral acceleration.

26.2 Probabilistic demand models

A demand model is an analytical expression or procedure that relates the demand on a structural component to the properties of the system considered (such as material properties, structural dimensions, and boundary conditions) and the intensity measures of the system demand(s) or hazard(s). A probabilistic demand model accounts for the relevant aleatory and epistemic uncertainties in predicting the demand quantity of interest. Such uncertainties include the randomness inherent in the properties of the system, model error, and statistical uncertainty. Gardoni *et al.* (2002) provide a complete description of such uncertainties. Ideally a model should incorporate all available sources of information including the rules of physics and mechanics and experimental and field data. To incorporate the rules of physics and mechanics and facilitate the acceptance of the proposed models, following Gardoni *et al.* (2002, 2003), we develop probabilistic demand models by adding a correction term to selected existing deterministic demand models. A wind turbine support structure is subject to multiple demands that could lead to failure due to excessive shear, moment, and deformations. A probabilistic demand model for analysis of a system having K different demands is formulated as

$$D_k(\mathbf{x}, \mathbf{w}, \Theta_k) = d_k(\mathbf{x}, \mathbf{w}) + \gamma_k(\mathbf{x}, \mathbf{w}, \theta_k) + \sigma_k \varepsilon_k \quad k = 1, \dots, K \quad [26.1]$$

where $D_k(\mathbf{x}, \mathbf{w}, \Theta_k)$ = k th probabilistic demand model, $\mathbf{x}$ = vector of material properties, structural dimensions and boundary conditions, $\mathbf{w}$ = vector of external loading, including environmental and seismic loading, $\Theta_k = (\theta_k, \sigma_k)$, in which θ_k = vector of unknown model parameters, $\hat{d}_k(\mathbf{x}, \mathbf{w})$ = selected deterministic demand model, $\gamma_k(\mathbf{x}, \mathbf{w}, \theta_k)$ = correction term for the bias inherent in the deterministic model, $\sigma_k \varepsilon_k$ = model error, ε_k = random variable with zero mean and unit variance, and σ_k = standard deviation of the model error. In formulating the model in Eq. (26.1), we employ a logarithmic transformation of the data to satisfy the homoskedasticity assumption (i.e., σ_k is constant), the normality assumption (ε_k follows the normal distribution), and the additive form used in the equations.

The correction term, $\gamma_k(\mathbf{x}, \mathbf{w}, \theta_k)$, is added to incorporate the missing terms in the deterministic model into the developed demand models and to correct for the potential bias in $\hat{d}_k(\mathbf{x}, \mathbf{w})$. It is written as

$$\gamma_k(\mathbf{x}, \mathbf{w}, \boldsymbol{\theta}_k) = \sum_{i=1}^p \theta_{ki} h_{ki}(\mathbf{x}, \mathbf{w}) \quad [26.2]$$

where $\boldsymbol{\theta}_k = [\theta_{ki}]$, $h_{ki}(\mathbf{x}, \mathbf{w})$ = normalized explanatory functions that might be significant in correcting $\hat{d}_k(\mathbf{x}, \mathbf{w})$, and p = the number of unknown model parameters. The model parameters, $\boldsymbol{\Theta}_k = (\boldsymbol{\theta}_k, \sigma_k)$, can be estimated following a Bayesian approach using field, laboratory, and/or virtual (simulated) data.

26.3 Demand models for the support structure of offshore wind turbines

In this section we develop probabilistic deformation, shear, and moment demand models for horizontal axis offshore wind turbines rated between 0.5 and 5 megawatts (medium to large wind turbines). The wind turbines of interest in this chapter are supported by a tubular steel tower, which is seated on a steel mono-pile foundation at the base and installed in water depths less than 30 meters. We predict deformation, shear, and moment demands on the support structures subject to seismic excitation, in addition to wind, wave, current, and turbine operational loadings.

26.3.1 Deterministic demand model

An ideal deterministic model $\hat{d}_k(\mathbf{x}, \mathbf{w})$ should be simple and yet accurate, and commonly accepted in practice. Because of these reasons, Mardfekri and Gardoni (2012) used FAST (Fatigue, Aerodynamics, Structures, and Turbulence) to compute deterministic predictions of the deformation, shear and moment demands on the support structure of wind turbines subject to wind, wave, current, and turbine operational loadings. FAST is a comprehensive aeroelastic simulator capable of predicting both the extreme and fatigue loads of two- and three-bladed horizontal-axis wind turbines. FAST employs a combined modal and multibody dynamics formulation. The structural model consists of nine rigid bodies (earth, support platform, base plate, nacelle, armature, gears, hub, tail, and structure furling with the rotor) and five flexible bodies (tower, three blades, and a drive shaft) that relates through 24 degrees of freedom (Jonkman and Buhl, 2005). Internally, FAST uses the computer program AeroDyn (Laino and Hansen, 2002) to compute the aerodynamic forces on the rotating blades. In FAST, the support platform is modeled as a rigid body with six degrees of freedom that allows applying external loads other than wind (i.e., wave, current and earthquake) on the support structure.

For a given value of the mean wind speed and turbulence intensity, a time history of wind speed is generated internally to FAST by TurbSim and used

as an input for the dynamic analysis in FAST. TurbSim is a stochastic, full-field, turbulent-wind simulator. It uses a statistical or empirical model (as opposed to a physics-based model) to numerically simulate time series of three-component wind-speed vectors (Jonkman, 2009). TurbSim supports the IEC Kaimal model (Kelley and Jonkman, 2007) that is used in this study. More details on the Kaimal model can be found in Mardfekri and Gardoni (2012).

In addition, FAST supports the JONSWAP/Pierson–Moskowitz spectrum (Dean and Dalrymple, 1991) to model linear irregular waves for given significant wave height and wave peak period. It then uses the Morison's equation to determine the hydrodynamic forces on the tower. Current loading is also incorporated in the Morison's equation. See Mardfekri and Gardoni (2012) for more details on wave and current loading.

The deterministic predictions of the deformation, shear, and moment demands on the support structure subject to seismic loading are developed in this chapter following a consistent approach to the one that Mardfekri and Gardoni (2012) used to compute deterministic predictions for wind, wave, current, and turbine operational loadings, which is described above. For given intensity and duration parameters of the ground motion and frequency content of a filter, a synthetic ground motion is generated following Rezaeian and Der Kiureghian (2010). The generated ground motion is then used as an input for a dynamic analysis also carried out using FAST. Rezaeian and Der Kiureghian (2010) formulated the ground motion process with a stochastic model as

$$\hat{y}(t) = q(t, \alpha) [\Psi(t)^T \mathbf{u}] \quad t_m \leq t < t_{m+1} \quad [26.3]$$

where t stands for the time, $q(t, \alpha)$ = a modulating function that controls the time-varying intensity of the process, $\Psi(t) = [\psi_1(t), \dots, \psi_n(t)]^T$ = a unit vector of the deterministic basis functions that controls the evolving frequency content of the process and $\mathbf{u} = [u_1, \dots, u_j]^T$ = a vector of standard normal random variable that provides the randomness that exists in real ground motions. Following Rezaeian and Der Kiureghian (2010), we select the gamma modulating function formulated as

$$\begin{aligned} q(t, \alpha) &= 0 & \text{if } t \leq T_0 \\ &= \alpha_1 (t - T_0)^{\alpha_2 - 1} \exp[-\alpha_3 (t - T_0)] & \text{if } T_0 \leq t \end{aligned} \quad [26.4]$$

This model has four parameters $\alpha = (\alpha_1, \alpha_2, \alpha_3, T_0)$, where $\alpha_1, \alpha_3 > 0$, $\alpha_2 > 1$, and T_0 denotes the start time of the process. The deterministic basis function is written as a function of the filter parameters, $\lambda(t_j)$, as follows

$$\psi_j[t, \lambda(t_j)] = \frac{h[t - t_j, \lambda(t_j)]}{\sum_{i=1}^n h^2[t - t_i, \lambda(t_i)]} \quad t_n \leq t < t_{n+1}; 1 \leq j \leq n \quad [26.5]$$

where $h[t - \tau, \lambda(\tau)]$ represents the pseudo-acceleration response of a single-degree-of-freedom linear oscillator subject to a unit impulse and is formulated as

$$\begin{aligned} h[t - \tau, \lambda(\tau)] &= \frac{\omega_f(\tau)}{\sqrt{1 - \zeta_f^2(\tau)}} \exp[-\zeta_f(\tau) \omega_f(\tau)(t - \tau)] \\ &\quad \times \sin\left[\omega_f(\tau) \sqrt{1 - \zeta_f^2(\tau)}(t - \tau)\right] \quad \tau \leq t \\ &= 0 \quad \text{otherwise} \end{aligned} \quad [26.6]$$

where τ = the time of the pulse, $\omega_f(\tau)$ = natural frequency, and $\zeta_f(\tau)$ = damping ratio of the filter.

The unknown parameters of the process $\alpha = (\alpha_1, \alpha_2, \alpha_3, T_0)$ and $\lambda(t_j) = [\omega_f(\tau), \zeta_f(\tau)]$ are assessed by matching the properties of generated and reference ground motions. The modulating function parameters $(\alpha_1, \alpha_2, \alpha_3)$ are related to ground motion time history variables $(\bar{I}_a, D_{5-95}, t_{\text{mid}})$, where $\bar{I}_a$ = expected Arias intensity (Arias, 1970) of the acceleration process, D_{5-95} = time interval between the instants at which the 5% and 95% of the expected Arias intensity are reached, and t_{mid} = time at which 45% of the expected Arias intensity is reached. The filter parameters $[\omega_f(\tau), \zeta_f(\tau)]$, which control the evolving predominant frequency and bandwidth of the process, are assessed based on their relations to the rate of zero-level up-crossings and the cumulative number of negative maxima and positive minima of the acceleration process. More details on the assessment of the unknown parameters of this process are presented in Rezaeian and Der Kiureghian (2010).

In case of a seismic event, a base acceleration time history is responsible for the resulting forces in the structure. FAST does not accept an acceleration time history as an input. Therefore we apply a time history of force, $F(t)$, to the platform. Using an artificially large mass for the support platform, the force $F(t) = \mu a(t)$ produces the desirable acceleration $a(t)$ at the base of the turbine support structure, where μ is the total mass of the support platform and the wind turbine.

26.3.2 Model correction

The correction term $\gamma_k(\mathbf{x}, \mathbf{w}, \theta_k)$ is intended to adjust for the bias inherent in the deterministic model. We use the linear form presented in Eq. (26.2) for the correction term, where for each demand of interest k , $\theta_k = [\theta_{ki}]$ and $h_{ki}(\mathbf{x}, \mathbf{w})$, $i = 1, \dots, p$, are, respectively, unknown model parameters and selected explanatory functions. Also, in this chapter, $k = \delta, v$ or m , for the deformation, shear, or moment demand, respectively. Ideally, explanatory functions should be selected from laws of mechanics and structural dynamics. We select $h_{k1}(\mathbf{x}, \mathbf{w}) = 1$ to capture potential constant bias in the model

that is independent of $\mathbf{x}$ and $\mathbf{w}$, and $h_{k2}(\mathbf{x}, \mathbf{w}) = \hat{d}_k(\mathbf{x}, \mathbf{w})$ to capture any possible under- or over-estimation of the deterministic models. To capture possible dependence of residuals on foundation, environment and earthquake parameters, which are not properly included in the deterministic model, additional explanatory functions are also considered. Table 26.1 shows candidate explanatory functions for the demand models. To characterize the influence of wind and wave parameters, h_{k3} – h_{k6} are selected as normalized functions of the mean wind speed, turbulence intensity, significant wave height, and wave peak period, respectively. The explanatory functions h_{k7} – h_{k13} are considered to incorporate the influence of the magnitude and frequency content of the ground motion. In addition, h_{k14} is considered to capture the possible influence of the rotor diameter. Finally we consider h_{k15} – h_{k18} to capture the possible influence of the foundation stiffness, which is not included in the deterministic model. A Bayesian inference is then used to estimate the unknown model parameters θ_k . Owing to the lack of available data needed to conduct the statistical analysis required to estimate the model parameters, we generate a database of virtual experi-

Table 26.1 Explanatory functions for demand models

Explanatory function	Formula	Parameters
h_{k1}	1	$k = \delta, v \text{ or } m$
h_{k2}	$\hat{d}_k$	$\hat{d}_k$ = deterministic deformation, shear or moment demand
h_{k3}	$\ln(W_s \cdot T_n/H_H)$	W_s = mean wind speed; H_H = hub height T_n = natural period of the support structure
h_{k4}	$\ln(IT_w)$	IT_w = wind turbulence intensity
h_{k5}	$\ln(H_s/H_H)$	H_s = significant wave height
h_{k6}	$\ln(T_p/T_n)$	T_p = wave peak period
h_{k7}	$\ln(S_a/g)$	S_a = spectral acceleration; g = ground acceleration
h_{k8}	$\ln(S_d/H_H)$	S_d = spectral displacement
h_{k9}	$\ln(PGA/g)$	PGA = peak ground acceleration
h_{k10}	$\ln(PGV \cdot T_n/H_H)$	PGV = peak ground velocity
h_{k11}	$\ln(PGD/H_H)$	PGD = peak ground displacement
h_{k12}	$\ln[2\pi PGV/(PGA \cdot T_n)]$	
h_{k13}	$\ln[2\pi PGD/(PGV \cdot T_n)]$	
h_{k14}	$\ln(RD/H_H)$	RD = rotor diameter
h_{k15}	$\ln(C_s/C_{smax})$	C_s = shear wave velocity of the soil; $C_{smax} = 194.594 \text{ m/s}$
h_{k16}	$\ln(C_{soil}/E_{soil})$	C_{soil} = soil cohesion
h_{k17}	$\ln[\tan(\phi_{soil})]$	ϕ_{soil} = soil friction angle
h_{k18}	$\ln(K_t/K_f)$	K_t = tower stiffness; K_f = foundation stiffness

ments using accurate nonlinear dynamic analyses of offshore wind turbine support structures as explained next.

26.3.3 Virtual experiment data

A set of representative configurations is selected to generate virtual experiments used to calibrate the proposed probabilistic demand models. The representative configurations are selected by conducting an experimental design to maximize their information content and minimize the computational costs associated with running 3D nonlinear finite element analyses.

Latin hypercube sampling

In the ‘classical’ design of physical experiments, a random variation is accounted for by spreading the sample points out in the design space and by taking multiple (replicated) data points (Simpson *et al.*, 2001). However, when it comes to deterministic computer experiments, there is no random error and no replication is required. According to Simpson *et al.* (2001), the design space is defined as the region bounded by the upper and lower limits of each design (input) variable being studied and the sample points should be chosen to fill the design space for computer experiments such that they spread as far from each other as possible. There are several ‘space filling’ designs in the literature; here the Latin hypercube sampling technique introduced by McKay *et al.* (1979) is used to select representative configurations of the support structure. The Latin hypercube sampling technique maximizes the minimum distance between sample points, while the range of each variable, x_i , is divided into N strata of equal marginal probability $1/N$; therefore, this method ensures that the sampling has a good coverage of the design space. A total of 100 configurations are generated. Variables considered to characterize each wind turbine configuration and their ranges are presented in Table 26.2.

Analytical modeling

Finite element models are developed in ABAQUS (2007) to simulate the dynamic response of the support structure of typical offshore wind turbines, subject to wind, wave, current, and turbine operational loading as well as earthquake. The finite element model of the support structure is constructed such that it accounts for the nonlinearity of the soil behavior and soil–structure interaction.

Representative ground motions for the virtual experiments must be properly selected in order to assess the seismic demand variables of interest and their associated uncertainties. In particular, the selected ground motion

Table 26.2 Geometrical and mechanical properties used in experimental design

Property	Range	Property	Range	
Rotor diameter, RD (m)	40–126	Soil elasticity modulus, E_{soil} (MPa)	13–200	
Tower height, H_1 (m)	40–80	Friction between pile and soil, μ	0.2–0.3	
Tower top diameter, D_1 (m)	1.9–4.0	Soil type	Clay	Sand
Tower base diameter, D_2 (m)	3.0–6.0	Soil cohesion, C_{soil} (kPa)	10–200	0–80
Tower diameter to thickness ratio	100–200	Soil friction angle, ϕ_{soil} ($^\circ$)	10–25	35–45
Water depth, WD (m)	20–30	Mean wind speed, W_s (m/s)	3.0–30	
Marin segment height, H_2 (m)	WD+(2.5–10)	Turbulence intensity, IT_w	0–0.16	
Steel type	S235, S275, S355	Significant wave height, H_s (m)	1.0–10	
Tower vibration period, T_n (s)	0.9–11.9	Significant wave period, T_p (s)	$3.6\sqrt{H_s} - 5.0\sqrt{H_s}$	
Pile penetration, H_3	10–50	Rated wind speed, $W_{s,\text{rated}}$ (m/s)	10.3–11.7	
Pile diameter to thickness ratio	50–100			

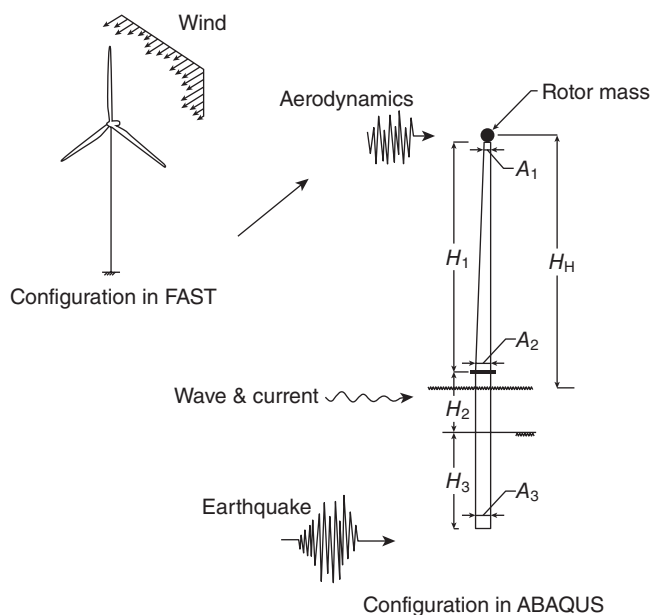
Table 26.3 Bins from which ground motions are selected

Bin no.	Bin characteristics	Magnitude	Distance (km)	No. of records
1	Large magnitude, small distance	$M > 6.5$	$13 < R < 30$	20
2	Large magnitude, large distance	$M > 6.5$	$R > 30$	20
3	Small magnitude, small distance	$M < 6.5$	$13 < R < 30$	20
4	Small magnitude, large distance	$M < 6.5$	$R > 30$	20
5	Near fault	$M > 6.5$	$R < 13$	20

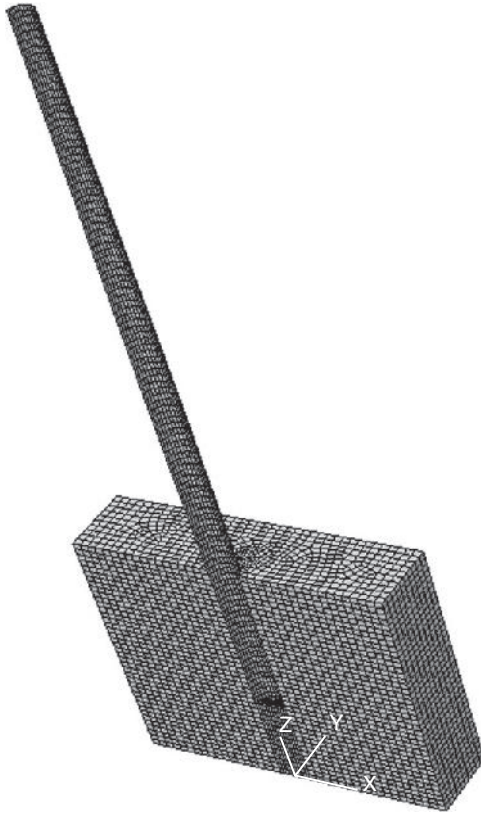
records should capture the characteristics of the possible seismic hazards, including their return periods, intensities, frequency contents, and durations (Krawinkler *et al.*, 2003). We select ground motion records from the PEER (Pacific Earthquake Engineering Research Center) NGA database (1999). Following Shome and Cornell (1999), the selected ground motions are subdivided into five bins based on moment magnitude (M) and the closest distance between the record location and the rupture zone (R). Table 26.3 shows the bins from which the ground motions are selected. Each bin rep-

resents specific combinations of the earthquake characteristics and the collection of all bins captures all possible characteristics. We use 20 representative ground motion records from each bin. It is noteworthy that selected ground motions are recorded at the surface. For the purpose of these dynamic analyses, we transfer the recorded ground motions to the depth of the soil mass base using the continuous solution to the wave equation (Roesset and Whitman, 1969). We idealize the soil profile as a system of homogeneous, visco-elastic sublayers of infinite horizontal extent and the shear waves are considered to propagate vertically. Also, to account for the nonlinearity of the soil, we use an equivalent linear procedure developed by Idriss and Seed (1968).

The aerodynamics of the turbine is simulated by the support of FAST. We use FAST to simulate the wind turbine and produce the time history of the forces at the top of the tower due to the wind and the operation of the turbine. The time history of wind loading used as an input for FAST is generated using TurbSim. The result of this simulation is the operational loading on the tower, which is used in the finite element model of the support structure as an external loading in addition to wave and current loading. Figure 26.1 shows a schematic representation of how FAST and ABQAU are combined to model the dynamic behavior of a wind turbine system.



26.1 Schematic representation of an offshore wind turbine model in FAST and ABAQUS.



26.2 Finite element model of a sample wind turbine support structure in ABAQUS.

Figure 26.2 shows the detailed 3D nonlinear finite element model of a sample wind turbine support structure created in ABAQUS. Considering the symmetry of the super-structure and foundation, only half of the support structure is modeled. The tubular steel tower, marine segment and pile foundation are modeled using linear elastic 3D shell elements. Foundation nonlinearities are considered explicitly in defining soil behavior and soil–pile interactions. The Mohr–Coulomb plasticity model is used to define the nonlinear behavior of the soil.

The nonlinear behavior of the soil–pile contact is modeled using ‘contact pair’ in ABAQUS. Tangential movement between the two parts, pile and surrounding soil, is allowed with a friction coefficient ranging from 0.2 to 0.3. In the radial direction, a ‘no separation’ contact behavior is assumed. The pile outer surface is chosen as the ‘master surface’ and the surface of

the soil mass which is in contact with the pile is considered to be the ‘slave surface’. The ‘small sliding’ tracking approach is employed for the contact of the two bodies assuming that even if the two bodies undergo large motions, there is relatively little sliding of one surface along the other. An elastic-plastic Coulomb model is also used to describe the nonlinear behavior of the soil–pile contact.

A model using the p - y curves is implemented for estimating the pile lateral stiffness, to be then used in correction terms. The p - y method uses an elastic beam column member to model the pile and nonlinear horizontal springs to represent the soil reactions. The p - y curves describe the nonlinear behavior of the soil springs. They were originally proposed by Matlock (1970) for soft clays under the water table and models for hard clays and sands were shortly after introduced by Reese *et al.* (1975). In this work the sand, soft clay, and hard clay models are used. The clay model requires the specification of a parameter ε_{50} that is assumed to be 0.005, as recommended by Reese *et al.* (1975).

Equality and lower bound data

Following Gardoni *et al.* (2002) and Ramamoorthy *et al.* (2006), the data from the virtual experiments are divided into equality and lower bound data. An equality datum is such that the quantity of interest (deformation, shear force, or bending moment) is recorded in the 3D finite element analysis and the recorded value is believed to be accurate. A lower bound datum is such that an accurate record of the quantity of interest is not available and only a value that is a lower bound to the true value is available.

Specifically, when the displacements in the finite element analyses are large, the results might not be accurate and it is found that they are sensitive to how the solution method handles large displacements and second order effects. Therefore, we consider a threshold for drift of 5%, such that if the maximum drift during one time history analysis is less than 5%, then the deformation, shear, and moment data are considered as equality data. If an analysis produces a drift that exceeds 5%, then we consider the 5% as a lower bound datum for the deformation, and the maximum shear and moment that occurred prior to reaching the 5% drift as the lower bound data for the shear and moment, respectively. With this approach, we include the data from analyses that lead to large deformations without letting inaccurate values wrongfully influence the model parameters.

26.3.4 Probabilistic demand models

A virtual experiment database is generated (as described in Section 26.3.3) to estimate the unknown model parameters of the probabilistic models

(formulated in Sections 26.3.1 and 26.3.2) for deformation, shear and moment demands on offshore wind turbine support structures due to seismic loading in addition to environmental (wind, wave, and current) and operational loadings. To develop parsimonious probabilistic demand models (i.e., with only the explanatory functions that are strictly needed) a model selection process is used to identify the important explanatory functions among the candidates presented in Table 26.1.

Model selection

A step-wise addition method is used for model selection. The first step of this process is to choose $\gamma_k(\mathbf{x}, \mathbf{w}, \boldsymbol{\theta}_k) = \theta_{k1}h_{k1}(\mathbf{x}, \mathbf{w})$, where $h_{k1}(\mathbf{x}, \mathbf{w}) = 1$ and estimate the model parameters using the Bayesian approach described next. In the second step, the residuals (i.e., the difference between the virtual data and the median prediction from the probabilistic model) are plotted versus all the remaining explanatory functions. Finally, in the third step, an additional term $\theta_{kj}h_{kj}(\mathbf{x}, \mathbf{w})$ might be added to γ_k if there is evidence of a trend in the plots of the residuals. In particular, the term that corresponds to the explanatory function with the clearest trend (here $h_{kj}(\mathbf{x}, \mathbf{w})$) should be added. Steps 2 and 3 are then repeated until no clear trend remains.

Bayesian updating

The unknown model parameters $\boldsymbol{\theta}_k$ are estimated using a Bayesian approach using the following updating rule (Box and Tiao, 1992):

$$f(\boldsymbol{\theta}_k) = \kappa L(\boldsymbol{\theta}_k) p(\boldsymbol{\theta}_k) \quad [26.7]$$

where $f(\boldsymbol{\theta}_k)$ = the posterior distribution of $\boldsymbol{\theta}_k$ that represents the updated state of knowledge, $L(\boldsymbol{\theta}_k)$ = the likelihood function that represents the objective information on $\boldsymbol{\theta}_k$ that comes from this study's virtual experiments, $p(\boldsymbol{\theta}_k)$ = the prior distribution of $\boldsymbol{\theta}_k$ that reflects the state of knowledge about $\boldsymbol{\theta}_k$ prior to conducting the virtual experiments, and κ = a normalizing factor. The likelihood is a function that is proportional to the conditional probability of observing the results from the virtual experiments for a given value of $\boldsymbol{\theta}_k$. Following Gardoni *et al.* (2002), $L(\boldsymbol{\theta}_k)$ is written as:

$$L(\boldsymbol{\theta}_k) \propto \prod_{\text{equality data}} \left\{ \frac{1}{\sigma_k} \varphi \left[\frac{r_{ik}(\boldsymbol{\theta}_k)}{\sigma_k} \right] \right\} \times \prod_{\text{lower bound data}} \Phi \left[-\frac{r_{ik}(\boldsymbol{\theta}_k)}{\sigma_k} \right] \quad [26.8]$$

where $r_{ik}(\boldsymbol{\theta}_k) = D_{ik} - \hat{d}_k(\mathbf{x}_i, \mathbf{w}_i) - \gamma_k(\mathbf{x}_i, \mathbf{w}_i, \boldsymbol{\theta}_k)$, D_{ik} = observed value for the k th demand for a given $\mathbf{x}_i$ and $\mathbf{W}_i$.

Deformation demand model

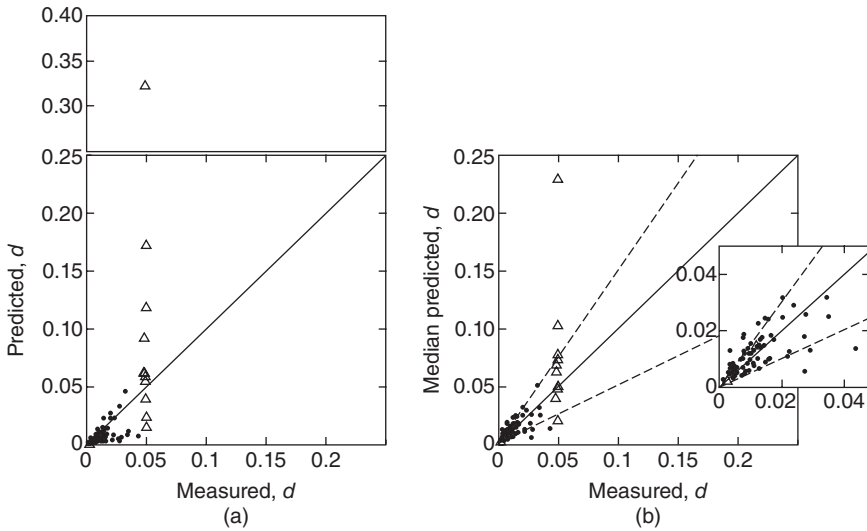
The probabilistic deformation demand model is formulated in terms of the natural logarithm of the drift demand defined as deformation demand at the top of the tower, Δ , normalized by the hub height, H_H . The logarithmic transformation is used to satisfy the homoskedasticity, normality, and additivity assumptions. Upon carrying out the model selection process, final probabilistic deformation demand model is written as:

$$D_\delta(\mathbf{x}, \mathbf{w}, \boldsymbol{\theta}_\delta) = \hat{d}_\delta(\mathbf{x}, \mathbf{w}) + \theta_{\delta 1} + \theta_{\delta 2} \hat{d}_\delta(\mathbf{x}, \mathbf{w}) + \theta_{\delta 3} \ln\left(\frac{S_d}{H_H}\right) + \theta_{\delta 4} \ln\left(\frac{W_s \cdot T_n}{H_H}\right) \\ + \theta_{\delta 5} \ln\left(\frac{K_t}{K_f}\right) + \theta_{\delta 6} \ln\left(\frac{K_t}{K_f}\right)^2 + \sigma_\delta \varepsilon_\delta \quad [26.9]$$

Since no prior information on the unknown parameters $\boldsymbol{\Theta}_\delta = (\boldsymbol{\theta}_\delta, \sigma_\delta)$ is available before conducting the virtual experiments, in the Bayesian approach we used a non-informative prior in the form $p(\boldsymbol{\theta}_\delta, \sigma_\delta) \propto \sigma_\delta^{-1}$ (Box and Tiao, 1992). Table 26.4 gives the posterior statistics of $\boldsymbol{\Theta}_\delta = (\boldsymbol{\theta}_\delta, \sigma_\delta)$. Also, Fig. 26.3 shows a comparison between measured and predicted deformation demands based on deterministic and probabilistic models. The points inscribed in triangles represent the lower bound data, the others are the equality data. The dashed lines in Fig. 26.3(b) delimit the region within one standard deviation of the model. The figure clearly shows an improvement in predicting the demand when using the proposed probabilistic demand model rather than the deterministic model demand. The correction term adjusts the bias inherent in the deterministic model and makes the probabilistic model unbiased.

Table 26.4 Posterior statistics of the parameters in the deformation demand model

Parameter	Mean	Standard deviation	Correlation coefficient						
			$\theta_{\delta 1}$	$\theta_{\delta 2}$	$\theta_{\delta 3}$	$\theta_{\delta 4}$	$\theta_{\delta 5}$	$\theta_{\delta 6}$	σ_δ
$\theta_{\delta 1}$	4.81	2.062	1.0						
$\theta_{\delta 2}$	-0.50	0.104	-0.01	1.0					
$\theta_{\delta 3}$	0.151	0.065	-0.14	-0.44	1.0				
$\theta_{\delta 4}$	0.078	0.111	0.06	-0.66	0.05	1.0			
$\theta_{\delta 5}$	1.94	0.677	0.96	-0.22	-0.15	0.21	1.0		
$\theta_{\delta 6}$	0.153	0.053	0.94	-0.26	-0.13	0.22	0.99	1.0	
σ_δ	0.493	0.043	0.03	0.06	0.06	-0.05	0.02	0.03	1.0



26.3 Comparison between measured and predicted deformation demands based on (a) deterministic and (b) probabilistic models.

Shear demand model

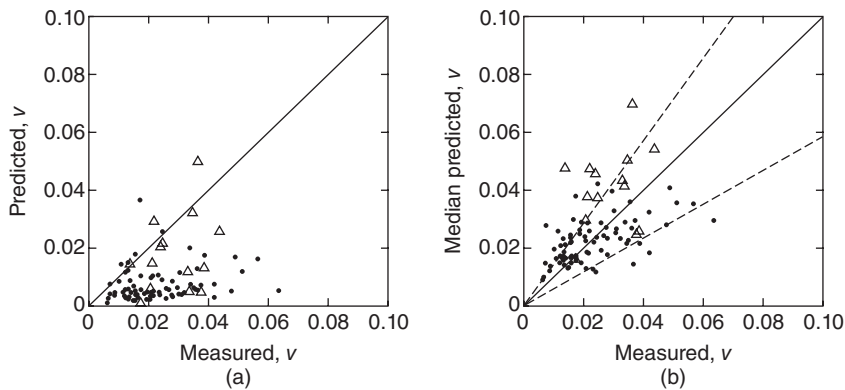
The probabilistic shear demand model is formulated as the natural logarithm of the shear demand at the base of the tower normalized by the mean value of the yield shear force, defined as $\hat{V}_y = \hat{f}_y A (3/4)(R^2 + r^2)/(R^2 + Rr + r^2)$, where $\hat{f}_y$ = expected yield stress of steel, A = tower base cross-section area, and R and r = outer and inner diameter of the tower section, respectively. The model selection results in the following probabilistic shear demand model:

$$D_v(\mathbf{x}, \mathbf{w}, \boldsymbol{\theta}_v) = \hat{d}_v(\mathbf{x}, \mathbf{w}) + \theta_{v1} + \theta_{v2} \hat{d}_v(\mathbf{x}, \mathbf{w}) + \theta_{v3} \ln\left(\frac{K_t}{K_f}\right) + \theta_{v4} \ln\left(\frac{S_a}{g}\right) + \theta_{v5} \ln\left(\frac{W_s \cdot T_n}{H_H}\right) + \theta_{v6} \ln\left(\frac{PGV \cdot T_n}{H_H}\right) + \sigma_v \varepsilon_v \quad [26.10]$$

As in developing the deformation demand model, due to the lack of prior information, we use a non-informative prior in assessing the posterior statistics of the model parameters, $\boldsymbol{\Theta}_v = (\boldsymbol{\theta}_v, \sigma_v)$, which are presented in Table 26.5. Figure 26.4 shows a comparison between measured and predicted shear demands. Comments analogous to those for the deformation demand model (Fig. 26.3) can be made also for the shear demand model based on Fig. 26.4.

Table 26.5 Posterior statistics of the parameters in the shear demand model

Parameter	Mean	Standard deviation	Correlation coefficient						
			θ_{v1}	θ_{v2}	θ_{v3}	θ_{v4}	θ_{v5}	θ_{v6}	σ_v
θ_{v1}	-4.57	0.508	1.0						
θ_{v2}	-0.849	0.097	0.60	1.0					
θ_{v3}	-0.300	0.056	0.66	0.03	1.0				
θ_{v4}	-0.007	0.062	-0.14	-0.27	-0.36	1.0			
θ_{v5}	-0.066	0.080	-0.15	-0.39	0.06	0.41	1.0		
θ_{v6}	0.137	0.064	0.02	-0.38	0.18	-0.44	-0.26	1.0	
σ_v	0.417	0.036	0.06	0.02	-0.06	0.09	0.02	0.05	1.0



26.4 Comparison between measured and predicted shear demands based on (a) deterministic and (b) probabilistic models.

Moment demand model

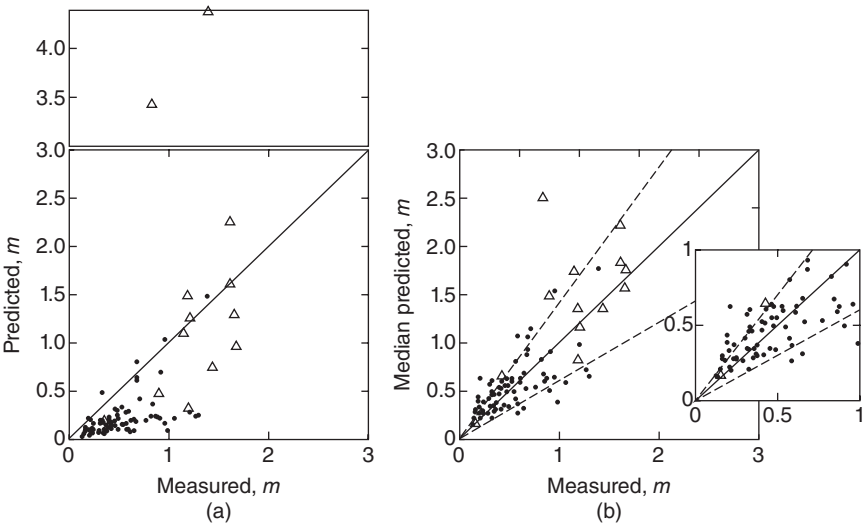
The probabilistic moment demand model is formulated as the natural logarithm of the moment demand at the tower base normalized by $\hat{M}_y = \hat{f}_y S$, where S = elastic section modulus at tower base. The model selection leads to the following model form:

$$\begin{aligned}
 D_m(\mathbf{x}, \mathbf{w}, \boldsymbol{\theta}_m) = & \hat{d}_m(\mathbf{x}, \mathbf{w}) + \theta_{m1} + \theta_{m2} \hat{d}_m(\mathbf{x}, \mathbf{w}) + \theta_{m3} \ln\left(\frac{S_a}{H_H}\right) + \theta_{m4} \ln\left(\frac{K_t}{K_f}\right) \\
 & + \theta_{m5} \ln\left(\frac{PGV \cdot T_n}{H_H}\right) + \sigma_m \epsilon_m
 \end{aligned}
 \quad [26.11]$$

Again, due to the lack of prior information, a non-informative prior is selected for $\Theta_m = (\theta_m, \sigma_m)$. Table 26.6 gives the posterior statistics of model parameters, and Fig. 26.5 shows a comparison between the predicted moment demands versus measured demand based on the deterministic and the probabilistic models. Comments analogous to those made based on Figs 26.3 and 26.4 are also applicable to the results shown in Fig. 26.5.

Table 26.6 Posterior statistics of the parameters in the moment demand model

Parameter	Mean	Standard deviation	Correlation coefficient					
			θ_{m1}	θ_{m2}	θ_{m3}	θ_{m4}	θ_{m5}	σ_m
θ_{m1}	-0.838	0.411	1.0					
θ_{m2}	-0.654	0.069	0.27	1.0				
θ_{m3}	-0.121	0.131	0.15	0.11	1.0			
θ_{m4}	-0.223	0.052	0.89	0.36	0.02	1.0		
θ_{m5}	0.313	0.141	-0.04	-0.33	-0.93	-0.07	1.0	
σ_m	0.397	0.034	-0.02	0.03	-0.01	-0.05	0.01	1.0



26.5 Comparison between measured and predicted moment demands based on (a) deterministic and (b) probabilistic models.

26.4 Example of fragility estimates for an offshore wind turbine support structure

We now use the developed demand models to investigate the fragility of an example offshore wind turbine support structure. For this purpose we consider the configuration of a typical 5 MW offshore wind turbine supported by a mono-pile installed in a 20 m water depth. The structure of interest is the NREL offshore 5-MW baseline wind turbine and its specifications is documented by Jonkman *et al.* (2009). Important properties of considered wind turbine are presented in Table 26.7.

Fragility is defined as the conditional probability of failure for a given value of the vector $\mathbf{w}$, which includes the mean wind speed, significant wave height, and spectral acceleration. Following Gardoni *et al.* (2002), a predictive estimate of the fragility can be formulated as:

$$\tilde{F}(\mathbf{w}) = P \left[\bigcup_k \{g_{sk}(\mathbf{x}, \mathbf{w}, \Theta_k) \leq 0\} | \mathbf{w} \right] \quad [26.12]$$

where $g_{sk}(\mathbf{x}, \mathbf{w}, \Theta_k)$ is the k th limit state function defined as

$$g_{sk}(\mathbf{x}, \mathbf{w}, \Theta_k) = C_{sk}(\mathbf{w}) - D_k(\mathbf{x}, \mathbf{w}, \Theta_k) \quad [26.13]$$

in which $C_{sk}(\mathbf{x})$ represent the capacity corresponding to $D_k(\mathbf{x}, \mathbf{w}, \Theta_k)$ and $s = y$ or u stand for yield and ultimate limit states, respectively.

In this study, we consider two modes of failure: shear and bending. Table 26.8 illustrates the proposed damage states and the corresponding limit states for a wind turbine subject to day-to-day wind, wave and current loads and seismic loading. Shear capacity is defined as $V_s = f_s A (3/4)(R^2 + r^2)/(R^2 + Rr + r^2)$, in which A , R and r stand for area, outer and inner diameter of the cross-section, respectively and $f_s = f_y$ or f_u , for yield and ultimate stresses. The yield and ultimate stresses, f_y and f_u , are considered to be lognormal

Table 26.7 Properties of the NREL offshore 5 MW baseline wind turbine

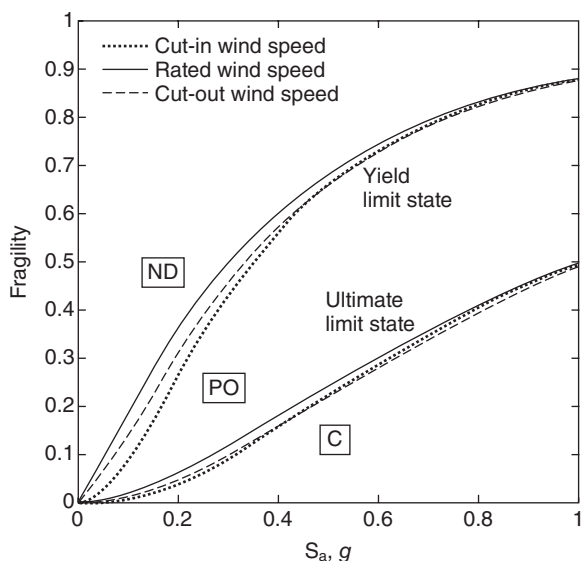
Property	Value	Unit
Rotor diameter	126	m
Hub height	90	m
Cut-in, rated, cut-out wind speed	3, 11.4, 25	m/s
Rotor mass	110 000	kg
Nacelle mass	240 000	kg
Tower mass	347 460	kg
Tower top diameter and wall thickness	3.87, 0.019	m
Tower base diameter and wall thickness	6.00, 0.027	m

Table 26.8 Proposed damage states

Damage state	Description	Performance level
No significant damage (ND)	No structural damage	Tower base shear or moment exceeds yield limits
Permanently out-of-service (PO)	Support structure yields Permanent excessive deformations will make the turbine out-of-service	Tower base shear or moment exceeds ultimate limits
Complete (C)	Support structure is unable to carry additional loads	

random variables with a mean of 300 and 410 MPa (for a structural steel of grade S235 according to EN 10025 standard; CEN, 2004), respectively and a coefficient of variation of 10%. In addition, $M_y = f_y S$ is used to calculate the yield bending moment capacity in which S = elastic section modulus. Finally, we calculate the ultimate bending moment capacity using a moment–curvature diagram constructed for the tubular cross-section of tower, based on the stress–strain curve for structural steel of grade S235. Therefore, for the structure of interest, ultimate bending moment is the maximum moment in the moment–curvature diagram with a mean of 390.6 MN m and a standard deviation of 39.57 MN m. Monte Carlo simulations are used to estimate the fragility for each failure mode and to assess the system fragility of the wind turbine support structure, where all the model parameters and error terms in the developed demand models are considered as random variables.

Figure 26.6 shows the predictive fragility estimates for the example off-shore wind turbine plotted as a function of the spectral acceleration, S_a , in units of g , at the natural period of the support structure ($T_n = 2.5$ s) within its linear elastic range, for both yield and ultimate limit states. The dotted, solid and dashed lines in the figure show the fragility due to the ultimate bending failure mode for cut-in, rated and cut-out wind speeds, respectively. Cut-in and cut-out wind speeds are the limits of the range of wind speeds in which a turbine is operating and producing power. The rated wind speed is the wind speed at which a control system is activated to limit the aerodynamic forces on the blades of the wind turbine and keep the power generated constant by changing the blade pitch angle. As shown in the figure, the fragility at the rated wind speed is higher than the fragilities at the other two wind speeds due to the higher wind speed than the cut-in



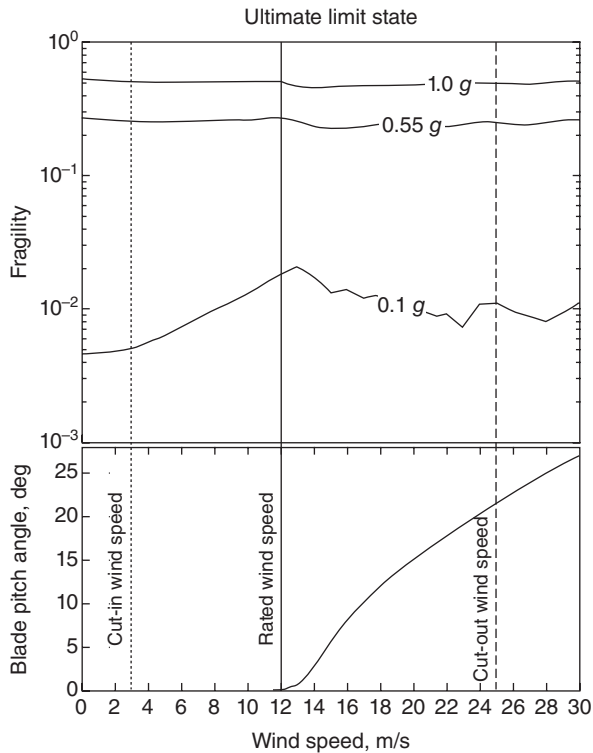
26.6 Fragility estimates for an offshore wind turbine as a function of spectral acceleration.

wind speed and operational loading than at the cut-out wind speed. However, the contribution of the wind loading is not significant compared with the seismic excitation even for small earthquakes. In addition, the fragility in shear failure mode is found to be negligible compared to the bending failure mode, as expected for slender elements like wind turbines towers, and is not shown in the figure because it is approximately equal to zero.

Predictive fragility estimates due to ultimate limit state are also plotted as a function of the wind speed, for different values of spectral accelerations (Fig. 26.7). The reduction in the probability of failure after the rated wind speed is due to the activation of the turbine control system at the rated wind speed that controls the power production and forces in the blades by changing the blade pitch angle, as shown in the bottom plot. Figure 26.7 again shows that the changes in wind speed do not affect noticeably the probability of failure, especially for large earthquakes.

26.5 Conclusions

This chapter developed probabilistic models for deformation, shear and moment demands on the support structure of offshore wind turbines subject to seismic, environmental, and operational loads. Employing an experimental design, 100 wind turbine configurations were generated to produce a



26.7 Fragility estimates for an offshore wind turbine as a function of wind speed at different spectral accelerations.

virtual experiment database used to calibrate the probabilistic models. Detailed finite element analyses were conducted on the generated configurations accounting for the nonlinearities of the soil behavior and soil-structure interaction. A step-wise selection process was used to develop appropriate model forms and a Bayesian approach was used to assess unknown model parameters. The developed probabilistic models account for the relevant aleatory and epistemic uncertainties in predicting the demand quantities of interest.

As an illustration, the developed demand models were used to estimate the fragility of a typical offshore wind turbine support structure. The fragility estimates show that bending failure controls the failure of the support structure, while the shear failure mode is negligible. Also the fragility estimates show that wind speeds within the operational range do not noticeably affect the probability of failure in case of a seismic excitation, especially for large earthquakes.

26.6 Future trends

Once more experimental or analytical data are available for the demands on offshore wind turbines, the models developed in this chapter can be updated using a Bayesian updating rule. The methodology developed in this chapter can also be applied to onshore wind turbines as well as both offshore and onshore wind turbines supported by different types of foundations.

26.7 Acknowledgments

The authors thank Dr Jose Roesset of Texas A&M University for many discussions regarding dynamic soil–structure interaction methods. We thank Dr Jason Jonkman of National Renewable Energy Laboratory for discussions regarding the wind turbine simulation program FAST. We acknowledge the Texas A&M Supercomputing Facility (<http://sc.tamu.edu/>) for providing computing resources useful in conducting the research reported in this chapter.

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Seismic risk and possible maximum loss (PML) analysis of reinforced concrete structures

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Abstract: Seismic risk analysis facilitates the quantitative evaluation of seismic damage and loss, and is an indispensable technique for disaster prevention planning and risk-based asset management. This chapter presents an analytical procedure for assessing seismic risk indices, such as the normal expected loss (NEL) and possible maximum loss (PML). First, a seismic risk analysis procedure for deriving seismic risk curves and risk indices is introduced. The method employs a multi-event model for earthquake scenarios and fragility functions for reinforced concrete (RC) structures based on nonlinear push-over analysis. Then, several case studies for reinforced concrete infrastructure, such as railway viaducts and buildings in Japan, are presented. The importance and usefulness of seismic risk curves and PML analysis are highlighted from the viewpoints of catastrophic earthquake risk mitigation and informed decision-making for asset management.

Key words: seismic risk, normal expected loss (NEL), probable maximum loss (PML), seismic risk curve, seismic event risk curve, reinforced concrete structure.

27.1 Introduction

Complex urban infrastructure and structures in Japan are exposed to significant risk due to exposure to recurrent earthquakes. For example, recent major events include the 1995 Kobe earthquake, the 2001 Geiyo earthquake, the 2003 Tokachi-Oki earthquake, the 2004 Chuetsu earthquake, and most notably the 2011 Tohoku earthquake. In particular, the casualty losses in the 1995 Kobe and 2011 Tohoku earthquakes exceeded 6500 and 20500, respectively, while the economic losses in these two events were tremendous, exceeding 100 billion and 210 billion US dollars, respectively. Therefore, quantitative assessment of catastrophic earthquake risk is essential to mitigate seismic damage and loss in future earthquakes. The quantitative assessment entails comprehensive understanding of earthquake source information (e.g. rupture process, propagation, and site effects) as well as vulnerability of infrastructure (e.g. structural damage mechanism, seismic loss generation, and societal/economical impact). Such assessment is often

conducted through probabilistic seismic risk analysis (PSRA) (e.g. Chang *et al.* 2000; Porter *et al.* 2001; Cornell *et al.* 2002; FEMA 2003; Goulet *et al.* 2007; Yoshikawa *et al.* 2007, 2009).

The PSRA quantitatively evaluates, through probabilistic calculus, the potential damage and loss that a certain group of structures in a region is likely to experience due to various seismic events. Although details of the existing methods are different, the fundamental framework is broadly similar for these methods, and is based on the total probability theorem. It combines seismic hazard (e.g. chance of experiencing strong ground motion excitation) with structural vulnerability (e.g. chance of reaching a specific damage/loss level for a given seismic intensity level). For instance, Chang *et al.* (2000) and Yoshikawa *et al.* (2007, 2009) considered scenario-based representation of seismic hazard, while Cornell *et al.* (2002) took advantage of the seismic hazard curve that is often available from a reliable third body, such as US Geological Survey (USGS). For vulnerability analysis, simpler nonlinear static methods via pushover analysis can be used (Yoshikawa *et al.* 2007, 2009). Alternatively, more rigorous methods, such as incremental dynamic analysis (Vamvatsikos and Cornell 2004), can be adopted in the assessment.

Mathematically, seismic risk due to a scenario earthquake can be defined as the product of the seismic hazard function and the vulnerability or fragility of structures. By taking into account possible scenarios with occurrence probabilities, overall seismic risk can be expressed as a seismic risk curve, which is a plot of seismic risk measure (e.g. structural drift response or specific damage level) as a function of occurrence probability. Moreover, from a seismic risk curve, various risk indices, such as normal expected loss (NEL) and probable maximum loss (PML), can be derived. These indices are useful to summarize succinctly the important features of the seismic risk, expressed in monetary terms, and facilitate the risk-based decision making by various stakeholders (e.g. owner, investor, and risk consultant). The adoption of such seismic risk indices has been greatly promoted in Japan (e.g. Kanda and Okamura 2006); for instance, PML values are included in financial reports of building/property assets, providing valuable information for investors and asset managers who are not familiar with engineering-based indices (e.g. design base shear coefficient).

This chapter presents seismic risk analysis and risk quantification, which are essential in the decision-making process of catastrophe risk management. The seismic risk obtained from both earthquake hazard curve and vulnerability characteristics of structures is discussed. In particular, focus is given to an analytical procedure for developing seismic risk curve, seismic event risk curve, and scalar risk measures in monetary terms, such as PML and NEL. Then, the proposed method is demonstrated through four case studies for reinforced concrete structures in Japan, such as railway viaducts

and office buildings. The aim of the case studies is to highlight the usefulness of seismic risk curves and PML analysis from the viewpoints of catastrophic earthquake risk mitigation and risk-based decision-making for asset management. Finally, conclusions and future trends of the seismic risk analysis are highlighted.

27.2 Analytical procedure for assessing seismic risk

This section summarizes theoretical background and mathematical formulation of seismic risk analysis. More details of the method can be found in Yoshikawa (2008), Nakamura and Ugata (2008), and Yoshikawa *et al.* (2012).

27.2.1 Seismic risk curve based on analytical method

A seismic risk curve $G_C(c)$ is calculated based on two random variables, earthquake hazard information and seismic vulnerability of a structure. It quantifies an adopted seismic risk measure as a function of annual occurrence probability. In other words, the $G_C(c)$ is equivalent to the probability distribution of the seismic risk measure by taking all relevant uncertainties in seismic hazard and structural vulnerability into account. Using the seismic risk curve, other useful quantities and curves, such as expected seismic loss, NEL, and PML, can be calculated. For the seismic risk measure, damage-based as well as loss-based indicators can be used (e.g. peak drift of a structural system or component, and total seismic loss for a building). A formulation to compute the $G_C(c)$ from seismic hazard curve ($P_A(\alpha)$) and seismic loss function ($f_C(c|\alpha)$) is discussed below.

The seismic loss function $f_C(c|\alpha)$ is characterized by seismic fragility curve and seismic damage function (Mander 1999; Shinozuka *et al.* 2000; Yoshikawa 2008). (Note: detailed descriptions of the fragility curve and damage function are beyond the scope of this chapter). Typically, the fragility curves are defined as the conditional probability function for the specified damage states, e.g. minor, major, extensive, and complete damage states in the HAZUS-Earthquake (FEMA/NIBS 2003), whereas the damage functions relates the attained damage states and (monetary) consequences. These curves/functions can be derived from statistical analysis of empirical data (e.g. observations of damage states and ground shaking from past earthquakes; see Shinozuka *et al.* 2000) and from analytical methods, such as nonlinear static pushover analysis and incremental dynamic analysis (Vamvatsikos and Cornell 2004).

For a seismic hazard curve $P_A(\alpha)$, α is the seismic hazard measure (i.e. seismic intensity measure); and its derivative) denoted as $p_A(\alpha)$, $p_A(\alpha) = -dP_A(\alpha)/d\alpha$, is gradient of the curve at a particular ground motion

level. Typically, it can be the peak ground acceleration (PGA) or peak ground velocity (PGV) at bedrock. The seismic hazard curves at specific locations are derived from probabilistic seismic hazard analysis (Cornell 1968; McGuire 2004; see Chapter 1), and may be available from the responsible governmental body/institution, such as the Japan Meteorological Agency and the USGS. By introducing the seismic loss function $f_c(c|\alpha)$ (precisely, this is the conditional probability density function of earthquake loss c for given ground motion level α), the seismic risk curve $G_C(c)$ can be formulated by the convolution of the two random variables (Cornell *et al.* 2002):

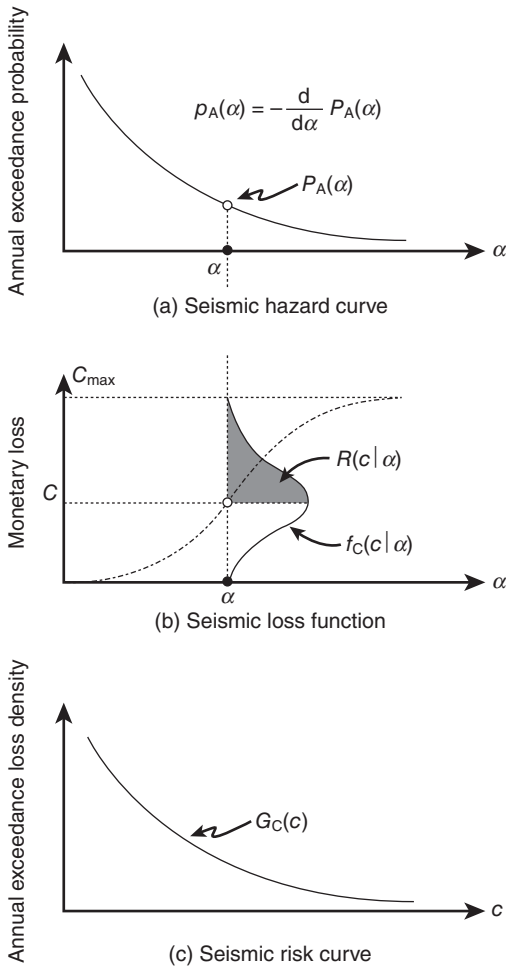
$$G_C(c) = \int_0^{\infty} p_A(\alpha) \left[\int_c^{c_{\max}} f_c(\zeta|\alpha) d\zeta \right] d\alpha \quad [27.1]$$

The $G_C(c)$ represents the probability of exceedance of seismic loss c , noting that the seismic intensity parameter α is integrated out. This means that, in the assessment, uncertainty of various possible ground motion levels is taken into account. For risk-based earthquake risk management, it is useful to express the seismic risk measure c in monetary values, such as structural loss (direct loss), operating loss (indirect loss), or outage days (suspension period), facilitating the informed decision making and risk communication by different stakeholders. The analysis procedure of Equation 27.1 is illustrated in Fig. 27.1. It is noted that other scalar risk measures, such as expected seismic loss, standard deviation of seismic loss, and fractiles of seismic loss (corresponding to specific selected probability levels), can be calculated based on $G_C(c)$.

27.2.2 Seismic event risk curve based on multi-event model

There are alternative formulations/methods to derive seismic event risk curves. In this subsection, a method that adopts the so-called multi-event model is introduced (Chang *et al.* 2000; Yoshikawa *et al.* 2009). The advantage of this method, in comparison with the above analytical method based on the seismic hazard curve, is the explicit consideration of earthquake scenarios that contribute to seismic hazard at a site. The main idea is to retain earthquake scenario information during the seismic risk assessment; this approach is particularly useful clearly when dominant earthquake sources are identified (e.g. major faults and subduction zones in Japan and California), and facilitates the transformation between multi-event-based results and specific-scenario-based results.

The first step in the multi-event model is to develop an inventory of possible seismic events at the proximity of a building site. This can entail, for



27.1 Analytical procedure for seismic risk curve.

example, consideration of all earthquakes that can happen within a radius of 300 km, evaluated by considering different earthquake sources/types (i.e. plate-boundary/in-slab subduction earthquakes, crustal earthquakes from clearly identified faults, and background earthquakes from unidentified faults). For each earthquake scenario, the probability of occurrence is assigned based on existing databases of major earthquakes (location, magnitude, depth, faulting mechanism, etc.). In assessing seismic hazard of the earthquake scenarios, regional ground motion prediction equations (e.g. Kataoka *et al.* 2006), along with appropriate site amplification factors can be employed. To derive the event-based seismic hazard curve, the identified

major scenarios are sorted in a descending order in terms of expected seismic intensity measure. Specifically, the plotting position for the earthquake scenarios for the event-based seismic hazard curve Q is given by:

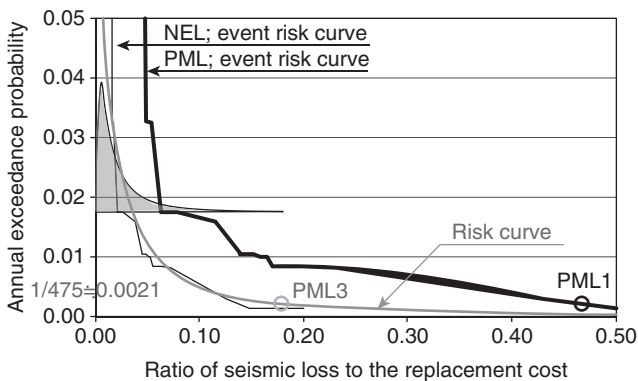
$$Q_j = q_j \text{ for } j = 1$$

$$Q_j = 1 - (1 - q_j)(1 - Q_{j-1}) = 1 - \prod_{j=1}^m (1 - q_j) \text{ for } j > 1 \quad [27.2]$$

where q_j is the assigned probability of occurrence for the j th scenario ($j = 1$ corresponds to the largest event in terms of seismic intensity measure); usually, q_j is expressed as the annual occurrence probability. Then, the conditional probability distribution of the seismic risk measure c is assessed for each scenario. This conditional probability distribution of c incorporates the uncertainty associated with the considered scenario, such as uncertainty of source magnitude and location, ground motion, site amplification, and seismic damage/loss. Finally, the event-based seismic risk curve is converted into an integrated seismic risk curve as follows:

$$G_C(c) = 1 - \prod_{j=1}^m \left[1 - q_j \int_c^{c_{\max}} f_C(\zeta|j) d\zeta \right] \quad [27.3]$$

where m is the number of scenarios considered in the assessment, $f_C(c|j)$ is the conditional seismic loss density function expressed in terms of earthquake scenario, rather than ground motion intensity level as in Equation 27.1 (note: the unconditional density function $f_C(c_j) = q_j \times f_C(c|j)$). This equation represents the integrated form of seismic risk curve resulting from all of the scenario earthquakes. The event-based seismic risk curve is illustrated in Fig. 27.2. Generally speaking, event-based seismic risk curves are jagged in appearance, because of discrete values of the assigned occurrence probabilities to individual scenario events.



27.2 Seismic event risk curves and integrated seismic risk curves.

Moreover, the use of the multi-event approach facilitates the development of other related seismic risk curves and quantities. Two seismic risk indices/quantities, NEL and PML, can be derived for each scenario as follows:

$$NEL_j = \int_0^{c_{\max}} c \cdot f_C(c|j) dc \quad [27.4]$$

$$0.9 = \int_0^{PML_j} f_C(c|j) dc \quad [27.5]$$

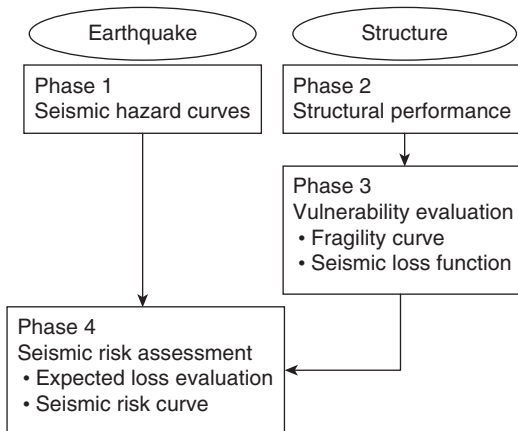
NEL_j is the expected seismic loss for the j th scenario, whereas the PML_j is defined as the 0.9-fractile value for the j th scenario (i.e. PML_j will be exceeded with 10% chance given the occurrence of the scenario). By repeating the evaluations of NEL and PML for all major scenarios, seismic risk curves based on NEL/PML can be developed. An illustration of the NEL and PML seismic risk curves is also given in Fig. 27.2; the PML curve is plotted on the right-hand side of the NEL curve, indicating greater seismic loss level. The use of these two risk curves may be beneficial as they capture different aspects of the seismic loss distribution (i.e. average versus right upper tail), especially for risk-based quantitative seismic risk management of buildings and infrastructure. (Note: the risk characteristics for rare events are of particular concern for risk undertakers, such as insurers and reinsurers.) In Japan, the return period level of 475 years (i.e. 90% non-exceedance probability in 50 years) is a popular probability level that is often considered for seismic risk assessment (see Fig. 27.2).

27.3 Case studies of seismic risk analysis for reinforced concrete structures

In this section, four case studies for quantitative seismic risks analysis for reinforced concrete structures in Japan are discussed. The aim of this section is to introduce practical applications of the above-mentioned analytical procedures and to demonstrate how such assessments can facilitate informed decision-making related to earthquake risk mitigation.

27.3.1 Railway viaduct

A railway system is an important infrastructure for urban intercity traffic, and provides daily mass transportation in the metropolitan area. In the metropolitan city of Tokyo, there are numerous railway viaducts due to limited space for transportation and complex configurations of major highways. Typical railway viaducts in Japan are reinforced concrete frame structures. Two case studies that consider old and current versions for Design



27.3 Seismic risk analysis procedure for railway viaduct (Yoshikawa *et al.*, 2007).

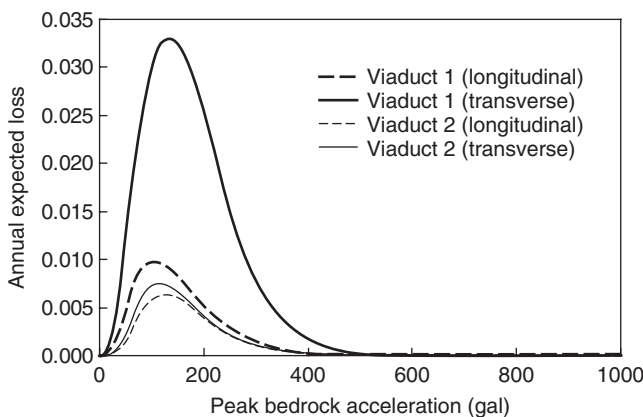
Provisions for Railway Structures (Seismic Design) by Railway Technical Research Institute (RTRI) are considered. The current version has been revised based on the careful review after the Kobe Earthquake in 1995. The objective of this investigation is to quantify the seismic risk of railway viaducts with different seismic capacities, and to assess the benefit gained from improved seismic design provisions for such structures.

The numerical procedure of seismic risk analysis is shown in Fig. 27.3, which consists of four phases (Yoshikawa *et al.*, 2007): (1) seismic hazard analysis at a construction site, (2) evaluation of structural performance of a viaduct by pushover analysis, (3) vulnerability and loss evaluation of the structure, and (4) seismic risk assessment by taking into account seismic hazard and vulnerability characteristics. The final output from the analysis is the seismic risk curve and other seismic loss quantities.

The first case study is focused on seismic risk analysis of reinforced concrete viaducts. In this case study, two viaducts are considered: viaduct 1 was designed by the old provisions (RTRI, 1992), while viaduct 2 is based on the current design code (RTRI, 2004). The structures are supposed to be located in Tokyo, Shibuya ward, and then seismic hazard analysis is carried out. The foundations of the viaduct 1 are spread-footings isolated each other and have no piles, while the columns of the viaduct 2 are connected rigidly with underground beams and have piles from the column base straight down into the ground.

Probabilistic seismic risk analysis of two railway viaducts was carried out. For nonlinear pushover analysis (as part of vulnerability assessment), the viaducts were modeled as two-dimensional frames in transverse and longi-

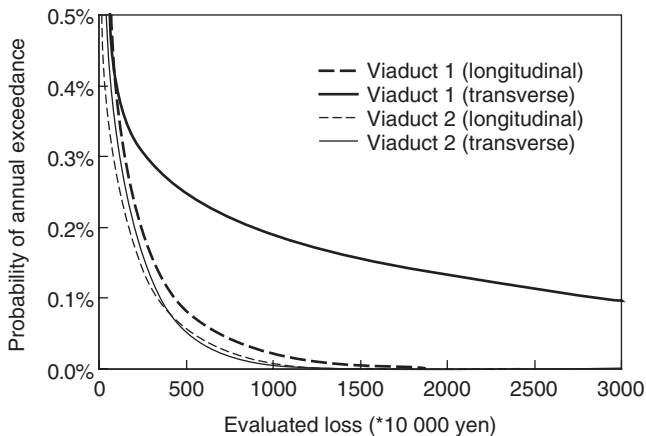
tudinal direction, respectively. The structural elements such as columns and beams were modeled as a beam element with nonlinear components at plastic hinge region. The foundation springs for the spread-footings and the piles were appropriately modeled with lateral, vertical and rotational stiffness calculated based on each design code. Figure 27.4 compares the seismic loss functions of two viaducts along transverse and longitudinal seismic excitations, expressed in terms of annual expected seismic loss. It is noted that the seismic loss functions for viaduct 1 (old code) are greater than those for viaduct 2 (new code); in particular, the difference is large for transverse direction. The main reason for this is the low lateral capacity in the transverse direction of the viaducts designed according to the old code. To evaluate the seismic risk analysis results quantitatively, the annual expected seismic losses for two viaducts are listed in Table 27.1. It is indicated that the annual expected loss of viaduct 2 significantly decreased to 19% in the transverse direction and to 63% in the longitudinal direction as compared to viaduct 1. The inspection of a scalar risk quantity greatly facilitates the comparison.



27.4 Seismic loss function of railway viaducts.

Table 27.1 Annual expected loss *ED* for two types of viaduct

Type	Annual expected seismic loss (yen)	
	Transverse	Longitudinal
1992 code: Viaduct 1	64 500	16 600
2004 code: Viaduct 2	12 100	10 400
Ratio of 2004 value to 1992 value	18.8%	62.7%



27.5 Seismic risk curves of railway viaducts.

Finally, seismic risk curves for two viaducts (in two directions) are computed using Equation [27.3], and are shown in Fig. 27.5. Similar conclusions can be obtained: the seismic loss for viaduct 1 in the transverse direction is remarkably higher for rare situations than other cases; for instance, at the annual probability of 0.002 (i.e. 500-year return period level), the loss for the viaduct 1 in the transverse direction is approximately five times greater than other cases. This example demonstrates that the use of seismic risk analysis can provide an effective means to compare the seismic risk at stake for structures with different seismic designs, and can facilitate the decision-making regarding the replacement of old-type viaducts to new-type viaducts (note: the benefit for such decisions is quantitatively evaluated.)

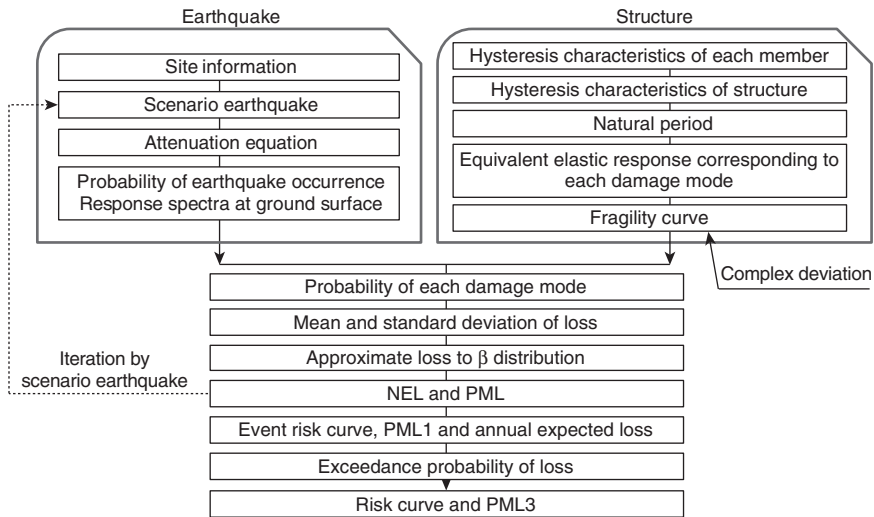
27.3.2 Reinforced concrete piers of a highway bridge

The next case study deals with reinforced concrete piers of highway bridges (Yoshikawa *et al.*, 2011). The focus of this investigation is to examine the effect of structural properties on the seismic risk curves and seismic loss quantities, such as PML. In total, six reinforced concrete piers are considered; the yield and ultimate displacement capacities of the six piers are summarized in Table 27.2. The six concrete piers have different seismic capacities, some are below the required seismic load level, while others exceed such a load level. The construction site is set to a location in Shinjuku-ward, Tokyo. In the context of Specifications for Highway Bridges (Part 5), the design acceleration spectrum is defined as three classes by the soil conditions: Class I (rock or shallow deposits), Class II (diluvium or alluvium), and Class III

Table 27.2 Structural specifications of six reinforced concrete piers

Soil condition	Design earthquake*	P019	P043	P072	P097	P141	P184
Class I	Demand (kN)	3301	4518	4978	5461	6077	6765
	Capacity (kN)	2300	4104	6557	8185	10911	13593
	Capacity/demand	1557	3779	6579	9539	14836	20725
Class II		0.47	0.84	1.32	1.75	2.44	3.06
	Type 1						
	Type 2	0.68	0.92	1.00	1.17	1.36	1.52
	Demand (kN)	4550	5486	6045	6631	7379	8215
	Capacity (kN)	4481	7439	9188	10277	11959	13333
	Capacity/demand	1557	3779	6579	9539	14836	20725
	Type 1	0.34	0.69	1.09	1.44	2.01	2.52
	Type 2	0.35	0.51	0.72	0.93	1.24	1.55

* Specifications for highway bridges.



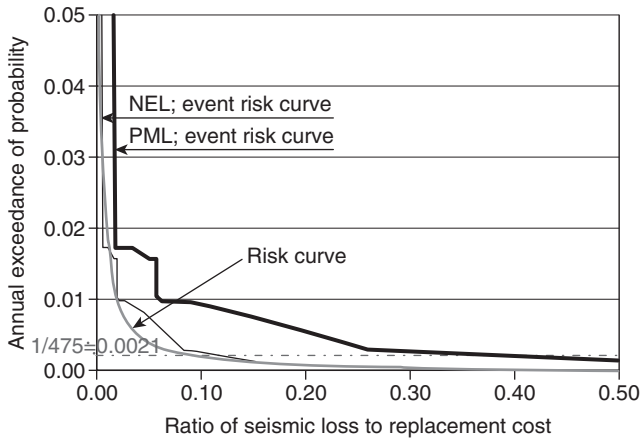
27.6 Flow diagram of seismic risk assessment for reinforced concrete piers.

(soft alluvium). Class I is, for example, is the hardest soil among other soil classes so that its natural period is less than 0.2 s, whereas other soil classes are associated with longer natural period values. Figure 27.6 displays the flow diagram of seismic risk analysis. The inelastic seismic demand estimation (i.e. fragility analysis) is conducted via equivalent linearization method.

The seismic hazard information is characterized by the multi-event model (see Section 27.2.2). For given scenarios, ground motion parameters were estimated based on the equation by Kataoka *et al.* (2006); the parameter in this case study was the acceleration response spectrum for natural period corresponding to 5% damping ratio at engineering bedrock surface. The logarithmic standard deviation of this prediction equation is 0.45 in the natural logarithmic scale.

The fragility information of the piers was categorized into four damage levels; little damage ($0 < \delta_r < \delta_y$), slight damage ($\delta_y < \delta_r < 3\delta_y$), intermediate damage ($3\delta_y < \delta_r < 5\delta_y$), and heavy damage ($5\delta_y < \delta_r$), where δ_r is the peak displacement response, δ_y is the yielding displacement of a pier, and δ_u is the ultimate displacement, respectively. This modeling of fragility characteristics by means of the displacements beyond the yield point has been facilitated based on the experimental results as well as engineering judgment (Yoshikawa *et al.* 2011).

Then, for each case (six piers at two ground conditions), the seismic risk curves were calculated following the procedure described in Section 27.2.2.



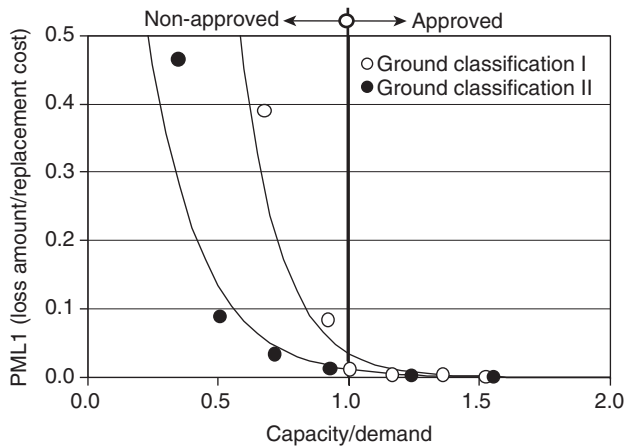
27.7 Seismic risk curves for NEL and PML.

Figure 27.7 shows the results for P019-Class I; in the figure, the probability level corresponding to 475-year return period is highlighted. The PML_{475} (i.e. PML at the return period of 475 years) is adopted as the seismic risk measure in this case study.

Finally, to assess the seismic performance of different bridge piers in relation to PML, the capacity and demand of the piers are estimated. The horizontal maximum strength P_u of the piers (referred to as capacity) and the required horizontal seismic load $k_{hc}W$ (referred as demand) are appraised according to the Japanese specifications for highway bridges (JRA, 2002). The capacity level of each pier varies depending on the designed load-carrying strength P_u , while the demand (actually seismic load) is almost the same among six piers. Figure 27.8 shows a relationship between PML_{475} (as a fraction of to the replacement cost of the piers) and capacity/demand ratios for two ground conditions. The capacity/demand ratio indicates that:

$$\begin{aligned} \frac{\text{Capacity}}{\text{Demand}} &> 1 \text{ code approved} \\ \frac{\text{Capacity}}{\text{Demand}} &< 1 \text{ non-approved} \end{aligned} \quad [27.6]$$

Figure 27.8 demonstrates that the risk values are high for piers with low-strength (P019, P043), whereas those for P141 and P184, having a capacity/demand ratio greater than 1.0, are sufficiently low. It is important to note that the PML values can be used as a surrogate measure for seismic design level. Because the PML values are expressed in monetary terms, various non-expert stakeholders can comprehend the extent of the seismic risk



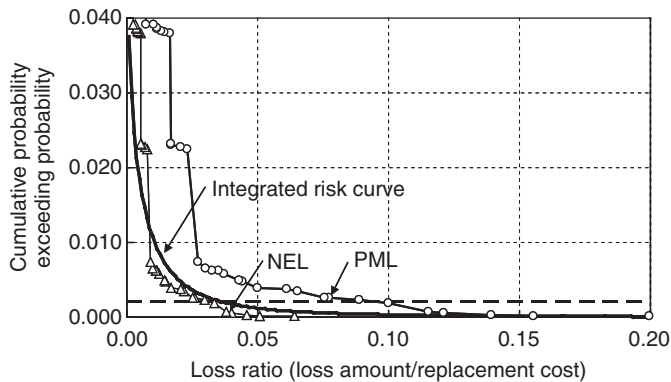
27.8 Relationship between PML_{475} and seismic capacity-demand index for six RC Piers.

better than the structural capacity/demand values. This is the practical advantage of using the seismic risk measures like PML.

27.3.3 Reinforced concrete buildings in various cities in Japan

The third case study is focused on the development of seismic event risk curves and its integrated risk curves of a seven-story office building that is located at various cities with different seismic intensities and characteristics in Japan (Yoshikawa *et al.*, 2009). Based on the present seismic design provision, the building type is office and is classified as a steel-framed reinforced concrete building. Its overall capacity can be specified as $Q_u/Q_{un}=1.05$, where Q_u is denoted as loading-carrying capacity, while Q_{un} expresses required strength that is regarded as demand. Since this building is designed to be $Q_u/Q_{un}=1.05$ (for all six cities considered in this study), it suggests that this building is safe but has only a small margin of safety.

First, by using Equations [27.4] and [27.5], respectively, the seismic event risk curves were evaluated for NEL_j and PML_j of buildings located in six major cities including Tokyo in Japan. Results for City D are shown in Fig. 27.9. The considered earthquake scenarios for City D are listed in Table 27.3. (Note: seismic characteristics of the earthquake scenarios differ, depending on the cities; a list of scenario events with their occurrence probabilities was prepared for each city.) The PML curve is greater than the NEL curve by about a factor of 2–3, noting that their difference is indicative of uncertainty in seismic loss estimation for given scenarios (i.e. right tail



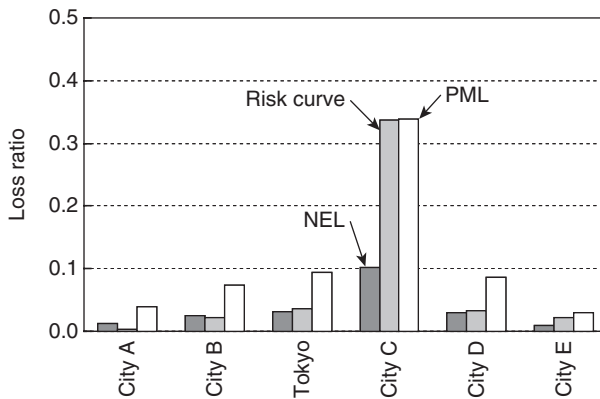
27.9 Comparison of seismic risk curves and integrated risk curve for City D.

Table 27.3 Summary of the major earthquake scenarios for City D

Hypocenter	Mag.	Annual prob. of occurrence	Cumulative annual prob.	PGA (cm/s ²)	NEL	PML
1 (139.70, 35.70)	6.5	0.000107	0.000107	344.1	0.1038	0.3447
2 (139.50, 35.70)	6.5	0.000107	0.000214	313.4	0.0814	0.2600
3 (139.70, 35.50)	6.5	0.000107	0.000322	300.7	0.0729	0.2298
4 (139.70, 35.70)	6.0	0.000309	0.000631	285.3	0.0633	0.1966
5 (139.50, 39.50)	6.5	0.000107	0.000738	280.0	0.0601	0.1859
6 1703, 1923 Kanto	8.0	0.001114	0.001852	270.9	0.0549	0.1686
7 Kanto north-western fault	8.0	0.000000	0.001852	264.2	0.0512	0.1566
8 (139.50, 35.70)	6.0	0.000309	0.002161	253.5	0.0457	0.1388
9 (139.70, 35.50)	6.0	0.000310	0.002470	240.7	0.0395	0.1193
10 (139.70, 35.70)	5.5	0.000891	0.003359	225.6	0.0329	0.0988
11 (139.50, 35.50)	6.0	0.000310	0.003668	220.1	0.0307	0.0921
12 Tachikwa fault zone	7.4	0.000428	0.004094	220.0	0.0307	0.0919

sensitivity). Therefore, from catastrophic risk management viewpoints, consideration of both curves may provide additional insight on the underlying seismic loss process.

The same analysis was repeated for six cities (including Tokyo); values of the three risk measures, PML, NEL, and integrated seismic risk curve, for the six cities are compared in Figure 27.10. Note that these risk measures are calculated for the return period of 475 years (as a typical risk level). From Fig. 27.10 it can be observed that for all cities, except for City C, the

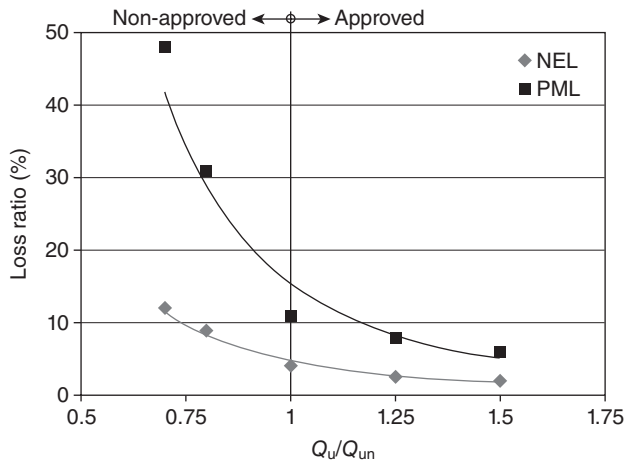


27.10 Comparison of various risk indices based on recurrence time of 475 years.

estimated seismic risk level by the integrated risk curves are smaller than PML_{475} . This is because that City C is located in the region where the seismic hazard is very high. It is noted that values of NEL_{475} is approximately a third of PML_{475} . It is denoted that since the load-carrying capacity ratio Q_u/Q_{un} of buildings and soil conditions at six cities are the same in order to simplify this case study, loss ratios of all buildings such as NEL and PML are only dependent on the seismic intensity and characteristics of the city locations. Therefore, one clearly understands that City C exhibits extremely high risk because it is located near the source of strong earthquakes (Tokai region in the central Japan) with high probability. Such comparative analysis results provide valuable information to both expert engineers/risk analysts and non-expert investors and insurers. For example, results shown in Fig. 27.10 render a clear message to business managers about the extent of seismic risk at stake in monetary terms.

27.3.4 Comparative risk assessment of reinforced concrete buildings with different capacities

The final case study is related to seismic risk assessment of five-story reinforced concrete buildings with different structural capacity. The aim of this investigation is to evaluate the influence of the structural performance of the buildings on the risk assessment. Such analysis results are useful as they clearly show the trade-off between seismic code compliance level and the potential earthquake risk. Moreover, the seismic code compliance level has a direct influence on initial construction cost (different sectional sizes), and thus is related to cost-benefit analysis or optimum seismic design of buildings (Rosenblueth 1976; Rackwitz 2000). For such purposes, ratios of



27.11 Relationship between loss ratio and Q_u/Q_{un} .

the load-carrying capacity Q_u over the required load-carrying demand Q_{un} was varied from 0.7 to 1.5, by altering the sectional amount of reinforcement of beams and columns. (Note that the higher ratio of Q_u/Q_{un} can impart better seismic performance to a structure.) The adopted seismic risk analysis procedure is similar to the previous case study discussed in Section 27.3.3.

Probabilistic seismic risk analysis was conducted for five seismic design levels (i.e. different values of Q_u/Q_{un}). The calculated values of NEL_{475} and PML_{475} are shown in terms of Q_u/Q_{un} in Fig. 27.11 (note: NEL_{475} and PML_{475} are expressed as a fraction of the replacement cost of the building). These results indicate that even though the loss ratio increases with the increasing ratio of Q_u/Q_{un} , it becomes constant as the ratio of Q_u/Q_{un} becomes to be greater than 1.0, which implies that the marginal gain due to increased section strength becomes less as the seismic design level increases. It is also found that the values of PML_{475} are more sensitive to Q_u/Q_{un} in comparison with those of NEL_{475} , indicating that very low seismic design levels may induce a disproportionate increase of the seismic loss level for rare situations. Such a tendency should be taken into account when decision making regarding the selection of a suitable seismic design level is conducted.

27.4 Conclusions and future trends

The features in this chapter and future trends are summarized as follows:

- Analytical formulations for the expected seismic risk, the seismic event risk curve, the seismic risk curve, and risk indices such as NEL and PML,

are presented. The proposed procedures and engineering representations are essential in seismic risk management, leading to decision making in the framework of catastrophe disaster management/strategy.

- Four case studies for typical reinforced concrete structures in Japan are carried out and comparatively discussed. The examples presented include railway viaducts, bridge piers, and mid-rise office buildings. The case studies are focused on the practical applications of the developed seismic risk analysis procedures to facilitate the informed decision making related to seismic risk mitigation. The use of quantitative risk measures, such as NEL and PML, together detailed seismic risk curves (to give more detailed and accurate descriptions of seismic risk at stake) provide both expert engineers/risk analysts and non-expert risk managers (e.g. insurers and investors) with valuable information on the seismic risk characteristics (e.g. frequency, extent, and uncertainty).
- The author believes that the analytical processes introduced here can encourage wider and more comprehensive development of risk-based management for earthquake-induced disasters. Moreover, the risk-based analysis may lead to more advanced seismic design frameworks in tandem with performance-based design methodologies.
- The next phase of the present research is risk analysis for a complex facility dealt with as a multiple serial model or a parallel/serial mixed model which consists of different types of components/structures. For this system risk analysis, it is expected that overall risk assessment for the railroad system and highway network can be carried out, evaluating all damages/losses and functionalities of the entire facility.

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Seismic risk management of insurance losses using extreme value theory and copula

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Abstract: Extremely large seismic loss due to a catastrophic earthquake is a serious concern for risk management of multiple buildings and infrastructure. In particular, accurate modelling of the upper right tail of seismic loss distribution is important for insurers and reinsurers. This study discusses a novel statistical approach for characterising the probability distribution of seismic loss of an insurance portfolio based on extreme value theory and copula. Its effectiveness is verified through a numerical example involving seismic loss estimation of multiple wood-frame houses in Vancouver, Canada. The proposed approach demonstrates good modelling capability with respect to the earthquake-engineering-based model, and is useful for more complex risk management problems.

Key words: catastrophic earthquake risk, earthquake insurance, extreme value theory, copula, seismic loss dependence.

28.1 Introduction

A catastrophic earthquake and its induced event, such as tsunami and fire, affect multiple structures and infrastructure simultaneously, causing catastrophic seismic loss to urban cities and coastal towns/communities. Incurred seismic damage includes loss of life and limb, direct financial loss to building properties and lifeline facilities, and indirect loss due to the ripple effects across regional and national economies. The 11 March 2011 M_w 9.0 earthquake in Tohoku, Japan, was one of such devastating and tragic events among recent earthquake disasters around the world (e.g. 2004 Sumatra, 2008 Sichuan, and 2010 Haiti earthquakes). Thus vulnerability assessment to extremely large earthquakes is a global and urgent problem in active seismic regions. In particular, catastrophic earthquakes impose tremendous financial stress on insurers and reinsurers underwriting earthquake insurance policies (Kleindorfer and Kunreuther, 1999; Grace *et al.*, 2003). Although earthquake insurance loss is not the major part of natural disaster loss claims, the potential earthquake-related loss can be extremely high. For instance, two major earthquakes in 2010, M_w 8.8 Chile and M_w 7.0 Darfield (New Zealand) earthquakes, caused insurance claims of about 8.0 and 4.4

billion US dollars, respectively (Swiss Re, 2011), which rank in the top 20 costliest natural catastrophes since 1970 (in terms of insurance loss). Moreover, two most recent devastating events in Christchurch (New Zealand) and Tohoku (Japan) are expected to reach insurance claims of 12 and 20–30 billion US dollars, respectively. (Note that the insured loss is only a fraction of the total economic seismic loss.)

To assess potential impact of future destructive earthquakes accurately and to implement risk mitigation measures effectively, there is a need for quantitative seismic loss estimation and decision-making tools. The tools should take into account key uncertainty and dependence in earthquake occurrence, ground motion intensity, and nonlinear structural behaviour. For such purposes, many methods and tools have been developed (FEMA/NIBS, 2003; Goulet *et al.*, 2007; Ellingwood *et al.*, 2008; Yucemen *et al.*, 2008; Goda *et al.*, 2011). For instance, the HAZUS-Earthquake, developed by FEMA and NIBS (2003), is capable of assessing regional seismic loss to a number of buildings and infrastructure due to a scenario earthquake, and serves as a critical risk management tool for both pre-disaster planning and post-disaster relief activities. However, it cannot evaluate the probability distribution of aggregate seismic loss; such information is valuable for evaluating/comparing viable mitigation options and for analysing insurance/reinsurance portfolio under earthquake risk (Kleindorfer *et al.*, 2005; Bazzurro and Park, 2007). In fact, there is an urgent research need to further develop more rigorous tools/methods that can be used for complex risk management problems. One such problem is insurance/reinsurance portfolio analysis of properties protected by earthquake insurance (Dong and Grossi, 2005; Kleindorfer *et al.*, 2005; Bazzurro and Park, 2007; Goda and Yoshikawa, 2012).

An important aspect in assessing seismic loss of a portfolio of buildings and infrastructure is the adequate consideration of uncertainty and dependence of physical effects on multiple structures due to ground shaking. The probabilistic approach laid out by the Pacific Earthquake Engineering Research (PEER) Center is a rational and comprehensive framework (Goulet *et al.*, 2007), and serves as a good initial model for general purposes. Furthermore, consideration of spatiotemporally correlated seismic excitations, when analysing multiple structures, must be emphasised (Goda *et al.*, 2011). Recently, Bazzurro and Park (2007) investigated the effects of portfolio relocation and aggregation (i.e. modelling many buildings originally located in a wide area as a single macro-structure at a single site) on the accuracy of seismic loss estimation for multiple properties, and found that such artificial manipulation, which is routinely done in insurance portfolio analysis, could bias the assessment. Aiming at avoiding such inaccuracy in portfolio relocation and aggregation, Goda and Ren (2010) investigated a copula-based approach to combine seismic losses from two building

portfolios probabilistically, by taking seismic loss dependence of the portfolios into account. They showed that seismic losses from geographically close portfolios can be nonlinearly correlated in the upper tail. A failure to account for this correlation might result in biased seismic loss estimation of the combined portfolio. It was also highlighted that a single parametric model may not be sufficient to approximate the entire marginal probability distribution of seismic loss. In particular, a model that fits the major part of small loss data well may not be suitable for describing the upper tail of large loss data, which is of greater importance for insurers and reinsurers. A recent study by Goda and Yoshikawa (2012) demonstrated that physical effects of spatially correlated ground motions affect the insurer's ruin probability; adequate portfolio management strategies must be sought to meet regulatory requirements for stable operation, depending on reserve fund, characteristics of business related to non-catastrophic risks, and insurance arrangements.

In this chapter, a statistical approach based on extreme value theory and copula is proposed to model earthquake risk exposures of an insurance portfolio. Extreme value theory provides a rational theoretical framework for analysing large seismic loss data. The issue addressed herein is related to portfolio aggregation of two separate insurance portfolios in a seismic region. Special attention is given to the characterisation of the upper right tail of the combined seismic loss distribution. The approach is based on the generalised Pareto (GP) model to approximate seismic loss data that exceed some large threshold value for marginal distribution modelling, and employs the copula technique to combine two random loss datasets. The GP model for describing the upper tail of data has well-established statistical foundation based on Pickands–Balkema–de Haan theorem (Coles, 2001). Moreover, the copula technique effectively separates dependence modelling from marginal distribution modelling based on Sklar theorem (McNeil *et al.*, 2005). The above method is especially useful for assessing earthquake risk exposure for insurers and reinsurers who underwrite high loss coverage. To demonstrate the method, 2400 hypothetical wood-frame houses located in Vancouver, BC, which reflect realistic features of existing wood-frame houses in Vancouver, are considered. The 2400 houses are grouped into six portfolios (each consisting of 400 houses); the six portfolios are positioned at different locations with various inter-portfolio distances to investigate the effects of geographical separation distance on portfolio aggregation. An earthquake-engineering-based seismic loss model developed by Goda *et al.* (2011) is employed to generate seismic loss samples for the 2400 wood-frame houses; the seismic loss data are used to develop a statistical model based on the GP distribution and copula function. It is worth mentioning that the above set-up is motivated by two facts: (i) the use of numerical computer models for determining insurance coverage rates and assessing

catastrophe risk exposure is popular and inevitable in practice owing to the lack of empirical loss/claim data; and (ii) information regarding seismic loss dependence for rare events is critically missing. In Section 28.2, brief summaries of the peak-over-threshold (POT) method and the copula method are given. Subsequently, salient features of the earthquake-engineering-based seismic loss model are discussed in Section 28.3. In Section 28.4, the developed statistical models are applied to calculate several risk measures for a reinsurance portfolio, such as value-at-risk (VaR) and tail value-at-risk (TVaR). The results are compared with the estimates based on the earthquake-engineering-based model to investigate the accuracy of the proposed approach.

28.2 Statistical modelling of extreme data

28.2.1 Extreme value theory and marginal distribution modelling

The occurrence of extremely large losses is of particular concern for insurers and reinsurers who undertake catastrophe earthquake risk. Insurers and reinsurers need to deal with very rare events, estimate their potential loss, and design a portfolio to meet regulatory as well as business requirements. Therefore, accurate assessment of risk exposure due to extreme events is the primary focus for portfolio analysts and managers. Moreover, it is desirable to have a tool that enables quick assessment for sensitivity analysis.

Extreme value theory offers a rational theoretical framework for analysing large seismic loss data. A standard method in extreme value theory is the block maxima model, which focuses on the maximum data during a specified period only (e.g. annual maximum flood volume). Based on Fisher–Tippett–Gnedenko theorem, such data can be modelled as the generalised extreme value (GEV) distribution. The distribution function of the GEV model is given by:

$$F(x) = \begin{cases} \exp\left(-\left[1 + \xi(x - \mu)/\beta^{-1/\xi}\right]\right) & \xi \neq 0 \\ \exp(-\exp[-(x - \mu)/\beta]) & \xi = 0 \end{cases}, \quad [28.1]$$

where ξ , μ , and β are the GEV model parameters. The shape parameter ξ identifies a suitable type of the three limiting extreme value distributions: $\xi > 0$, $\xi = 0$, and $\xi < 0$ indicate the Frechet distribution, Gumbel distribution, and Weibull distribution, respectively. Given data, the model parameters can be estimated based on the maximum likelihood method.

Another useful class of extreme value models is the POT model; it concentrates on data exceeding a high threshold value and thus uses data more efficiently than the block maxima model. Under fairly general conditions that are applicable to the majority of common probability distributions,

Pickands–Balkema–de Haan theorem states that the limiting distribution of POT data Y that exceeds a sufficiently high threshold μ , converges to the GP distribution (note: $Y = X - \mu > 0$). The distribution function of the GP model is given by:

$$G(y) = P(Y \leq y | Y > 0) = \begin{cases} 1 - (1 + \xi y / \beta)^{-1/\xi} & \xi \neq 0 \\ 1 - \exp(-y/\beta) & \xi = 0 \end{cases}, \quad [28.2]$$

where ξ is the shape parameter and determines the upper tail behaviour of the data, and β is the scale parameter. Note that Equation [28.2] is given in terms of the shifted variable Y , rather than the original variable X . Given data Y , it is straightforward to estimate the parameters of the GP model based on the maximum likelihood method. The conditional representation of the POT data shown in Equation [28.2] can also be expressed in terms of the original data X as follows:

$$P(X > x) = P(X > \mu) P(X > x | X > \mu) = P(X > \mu) [1 + \xi(x - \mu)/\beta]^{-1/\xi}, \quad [28.3]$$

where $P(X > \mu)$ is the probability of exceedance of μ , and this value can be estimated from the data directly. Furthermore, the fractile value x_p that corresponds to the exceedance probability p can be obtained as:

$$x_p = \mu + \frac{\beta}{\xi} [(P(X > \mu)/p)^{\xi} - 1]. \quad [28.4]$$

The confidence interval of the fractile value x_p can also be calculated relatively easily (Coles, 2001; McNeil *et al.*, 2005).

A key step in modelling POT data using the GP model is to find a reasonable value of μ . A trade-off exists for the estimation of μ , because a μ value that is too low results in poor approximation of data using the GP model, whereas one that is too high reduces the number of data above the threshold level. Several diagnostic methods are available, such as the empirical mean excess plot and the shape parameter plot (Cebrian *et al.*, 2003; McNeil *et al.*, 2005). The mean excess plot examines a trend of $E[X - \mu | X > \mu]$ as a function of μ : a linear relationship is indicative of a sufficiently high threshold level. The shape parameter plot uses the stability of ξ for upper tail data. These two methods will be employed in Section 28.4. In summary, with given data at hand, an adequate threshold parameter μ needs to be selected based on diagnostic methods, and then, the other parameters $P(X > \mu)$, ξ and β are obtained.

The extreme value distributions, such as GEV and GP models, have statistical foundation in dealing with extreme data. Nevertheless, when actual data are considered, it is recommended to explore a wider class of probability distributions, particularly with heavy right tails, because other popular distributions, such as the lognormal and gamma models, and more elabo-

rated distributions, such as the four-parameter generalised beta of the second kind distribution (Kleiber and Kotz, 2003), may be more suitable for the specific case (e.g. when the threshold value μ is constrained to be a relatively low value). Such exploratory investigations are warranted for the copula approach, as described in the following, because it separates marginal distribution modelling and dependence modelling. For this, diagnostic tools and measures, such as quantile–quantile (Q-Q) plot and Akaike Information Criterion (AIC), can be adopted. The real benefit of these analyses is to gain confidence in the choice of marginal distribution types for random data.

28.2.2 Dependence modelling

A copula technique offers a flexible way of describing nonlinear dependence among multivariate data in isolation from their marginal probability distributions, and serves as a powerful tool for modelling as well as simulating nonlinearly interrelated multivariate data. The copula technique has been extensively used for financial and actuarial applications (Klugman and Parsa, 1999; McNeil *et al.*, 2005). Recently, the copula theory has been applied to the fields of natural sciences and engineering, such as hydrology and marine science (Genest and Favre, 2007; Salvadori *et al.*, 2007).

The mathematical basis of the copula technique is founded on Sklar theorem. For bivariate data X_1 and X_2 , the joint probability distribution function $H(x_1, x_2)$ is given by:

$$H(x_1, x_2) = P(X_1 \leq x_1, X_2 \leq x_2) = C[F_1(x_1), F_2(x_2)] = C(u_1, u_2), \quad [28.5]$$

where C is the copula function; $F_1(x_1)$ and $F_2(x_2)$ are the marginal distribution functions of X_1 and X_2 ; and u_1 and u_2 (which equal $F_1(x_1)$ and $F_2(x_2)$, respectively) are the standard uniform random variables. The dependence between X_1 and X_2 is characterised by the copula function C in terms of the transformed standard uniform variables u_1 and u_2 . A significant implication of this theorem is that marginal distribution modelling and dependence modelling can be carried out separately.

The raw bivariate data can be transformed into empirical copula data $C^{\text{Emp}}(u_1, u_2)$ (with standard uniform margins):

$$C^{\text{Emp}}(u_1, u_2) = \frac{1}{m} \sum_{i=1}^m \mathbf{1} \left[\frac{\text{rank}(X_{1,i})}{m+1} \leq u_1, \frac{\text{rank}(X_{2,i})}{m+1} \leq u_2 \right], \quad [28.6]$$

where $\mathbf{1}(\bullet)$ is the indicator function, which equals 1 if the argument is true and 0 otherwise, and $\text{rank}(X_{1,i})$ and $\text{rank}(X_{2,i})$ are the rank of $X_{1,i}$ and $X_{2,i}$ (i th sample in X_1 and X_2), respectively. Then, a suitable copula function can be fitted to $C^{\text{Emp}}(u_1, u_2)$ based on the maximum likelihood method (McNeil *et al.*, 2005). The copula fitting is repeated to find a suitable copula function

by trying several candidates. For quantitative model selection purpose, the AIC is useful; a model with the smallest AIC is the preferred copula function. The commonly used copula functions include the elliptical copulas (e.g. normal and t copulas) and Archimedean copulas (McNeil *et al.*, 2005). For modelling seismic loss data, an investigation by Goda and Ren (2010) showed that the Gumbel and heavy right tail (HRT) copulas are adequate, because of significant upper tail dependence.

The bivariate normal copula and t copula are defined by:

$$C_{\rho}^N(u_1, u_2) = \Phi_{\rho}[\Phi^{-1}(u_1), \Phi^{-1}(u_2)], \quad [28.7]$$

and

$$C_{v,\rho}^t(u_1, u_2) = t_{v,\rho}[t_v^{-1}(u_1), t_v^{-1}(u_2)], \quad [28.8]$$

where $\Phi_{\rho}(\bullet)$ is the bivariate normal distribution function with zero mean and the linear correlation coefficient ρ ; $\Phi^{-1}(\bullet)$ is the inverse standard normal distribution; $t_{v,\rho}(\bullet)$ is the bivariate t distribution function with zero mean, degree-of-freedom parameter v , and ρ ; and $t_v^{-1}(\bullet)$ is the inverse standard t distribution. Both normal and t copulas are symmetrical, and the normal copula is a limiting case of the t copula, when v becomes infinity.

On the other hand, the Gumbel and HRT copulas are given by:

$$C^{\text{Gu}}(u_1, u_2) = \exp\left\{-\left[(-\ln u_1)^{\theta} + (-\ln u_2)^{\theta}\right]^{1/\theta}\right\}, \quad [28.9]$$

and

$$C^{\text{HRT}}(u_1, u_2) = u_1 + u_2 - 1 + \left[(1 - u_1)^{-\theta} + (1 - u_2)^{-\theta} - 1\right]^{-1/\theta}, \quad [28.10]$$

where θ is the copula model parameter. It is noteworthy that the Gumbel copula has been studied extensively in bivariate extreme value theory, and is known as the logistic dependence model (Coles, 2001). The logistic model parameter α is related to θ in Equation [28.9] as $\alpha = 1/\theta$; $\alpha = 0$ indicates independence, whereas $\alpha = 1$ indicates perfect dependence.

Regarding the estimation of the dependence parameter, one more remark is in order. In bivariate extreme value theory, estimation of both marginal GP parameters and dependence parameter is carried out based on the maximum likelihood method by treating data pairs as censored samples (Coles, 2001; McNeil *et al.*, 2005). This is a sensible approach by utilising all available information of POT data (not only data pairs with joint exceedance but also those with exceedance of one of the thresholds only). Another more simplified approach is to focus only on data pairs that jointly exceed respective threshold values (Dupuis and Jones, 2006). Both approaches are valid, but emphasise different aspects of the bivariate POT data; the former provides an overall picture of data pairs that exceed threshold values at

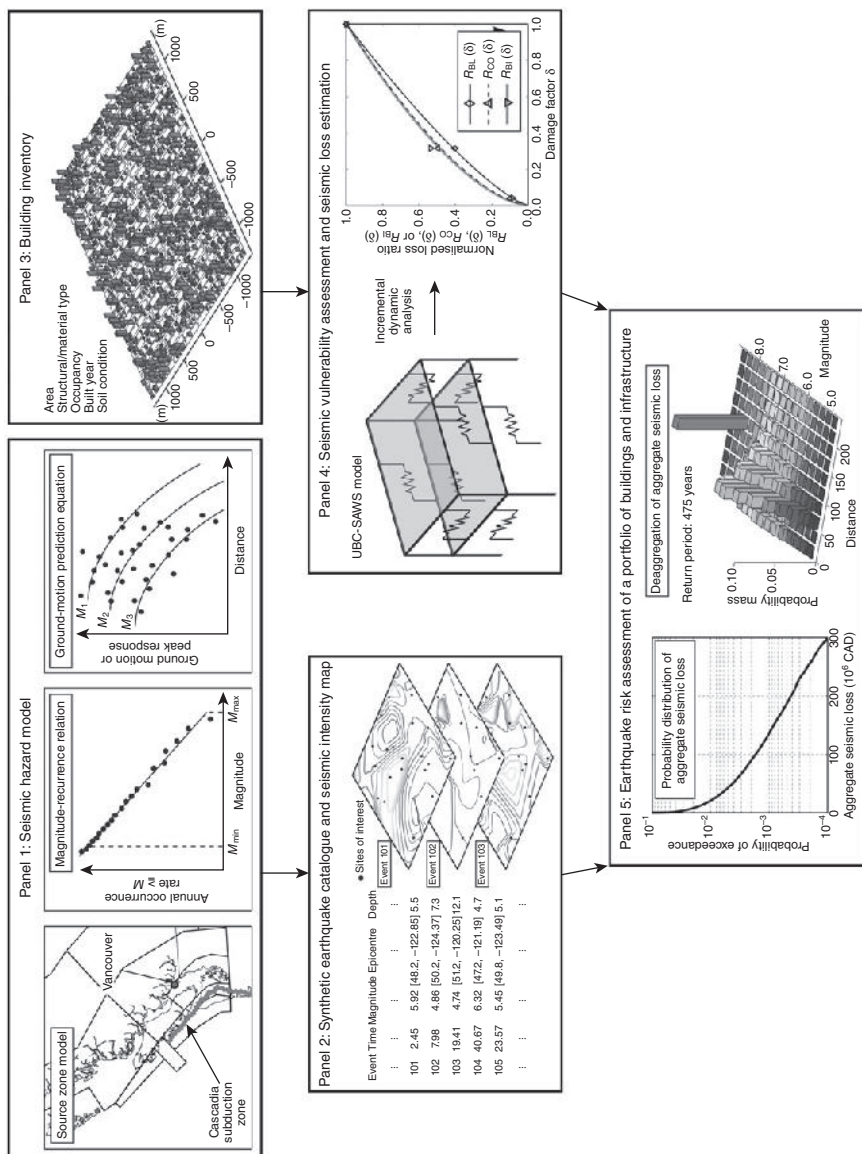
least for one variable, whereas the latter gives a more localised description of data pairs that are both extreme.

28.3 Insurer's earthquake risk exposure modelling

In this section, a regional seismic loss model for conventional wood-frame houses in south-western British Columbia, which was developed by Goda *et al.* (2011), is focused on. The model is fully probabilistic and evaluated using Monte Carlo simulation; the analysis framework is illustrated in Fig. 28.1. In the following, key features of the model components are presented (i.e. seismic hazard analysis and generation of spatially correlated seismic intensities in Section 28.3.1, and seismic vulnerability and loss assessment of wood-frame houses in Section 28.3.2). Interested readers are recommended to refer to Goda *et al.* (2011) for more detailed descriptions.

28.3.1 Seismic hazard analysis and generation of spatially correlated seismic intensities

Probabilistic seismic hazard analysis offers a rational framework to describe elastic seismic demand in terms of peak ground motions and response spectra, and addresses key uncertainties and dependence in earthquake occurrence (both temporal and spatial), earthquake magnitude, rupture characteristics, and ground motion intensities (see Chapter 1). The typical outputs are given by seismic hazard curves and seismic hazard deaggregation. For Canada, details of the national seismic hazard maps, which form the basis of seismic design provisions of the current National Building Code of Canada (NBCC2005), were given by Adams and Halchuk (2003). In this study, an updated seismic hazard model for western Canada (Goda *et al.*, 2010; Atkinson and Goda, 2011) is used to reflect newly available seismological information and models. The key improvements include: (i) conversion of different magnitude scales into a uniform moment magnitude scale in the earthquake catalogue; (ii) re-evaluations of magnitude–recurrence relations for different earthquake sources based on a longer and homogeneous catalogue up to the end of 2008; (iii) use of newer ground motion prediction equations with proper distance measure conversions (note that this enables more flexible adjustment for different site conditions using the average shear wave velocity in the uppermost 30 m V_{s30}); and (iv) consideration of probabilistic mega-thrust Cascadia scenario events (that can be as large as M_w 9.0 earthquake, similar to the 2011 Tohoku earthquake in size). Overall, adoption of the updated seismic hazard model instead of the NBCC2005 model decreases uniform hazard ordinates at short vibration periods (0.1–0.5 s) by 10–30% for the 2% probability of exceedance in 50



28.1 Seismic loss estimation framework.

years (i.e. seismic design level for the majority of structures in Canada), whereas uniform hazard ordinates at long vibration periods (2.0–3.0 s) are similar for the two models. (These changes are specific to soft-to-firm soil sites in Vancouver at the 2% probability of exceedance in 50 years; see Goda *et al.*, 2010, and Atkinson and Goda, 2011, for details.)

The seismic damage assessment of multiple buildings requires the estimation of simultaneous seismic effects at multiple building locations. This is beyond the capability of conventional ground motion prediction equations, because they have been developed for a single site. To enable the application of existing ground motion models to multiple sites, their correlation structures need to be characterised (Goda and Hong, 2008). Consider spectral accelerations at two sites i and j , $SA_{ik}(T_i)$ and $SA_{jk}(T_j)$, due to the k th seismic event in a synthetic earthquake catalogue; the two sites are separated by a distance Δ_{ij} (km), and T_i and T_j represent vibration periods of linear elastic single-degree-of-freedom oscillators at sites i and j , respectively. Using a suitable ground motion prediction model, $SA_{ik}(T_i)$ and $SA_{jk}(T_j)$ are characterised by:

$$\log_{10} SA_{ik}(T_i) = f(M_{w,k}, R_{ik}, \lambda_{ik}, T_i) + \eta_k(T_i) + \varepsilon_{ik}(T_i), \quad [28.11a]$$

$$\log_{10} SA_{jk}(T_j) = f(M_{w,k}, R_{jk}, \lambda_{jk}, T_j) + \eta_k(T_j) + \varepsilon_{jk}(T_j), \quad [28.11b]$$

and

$$\rho_T(\Delta_{ij}, T_i, T_j) = \rho_\eta(T_i, T_j) \frac{\sigma_\eta(T_i)\sigma_\eta(T_j)}{\sigma_T(T_i)\sigma_T(T_j)} + \rho_\varepsilon(\Delta_{ij}, T_i, T_j) \frac{\sigma_\varepsilon(T_i)\sigma_\varepsilon(T_j)}{\sigma_T(T_i)\sigma_T(T_j)}, \quad [28.11c]$$

where $f(M_w, R, \lambda, T)$ is the median prediction equation as functions of the moment magnitude M_w , distance R (typically, closest distance from station to rupture plane), and additional explanatory variables λ ; $\eta(T)$ is the inter-event residual with zero mean and standard deviation $\sigma_\eta(T)$; and $\varepsilon(T)$ is the intra-event residual with zero mean and standard deviation $\sigma_\varepsilon(T)$. $\eta(T)$ and $\varepsilon(T)$ are assumed to be independent and normally distributed, and the total standard deviation $\sigma_T(T)$ therefore equals $[\sigma_\eta^2(T) + \sigma_\varepsilon^2(T)]^{0.5}$. In Equation [28.11c], $\rho_T(\Delta_{ij}, T_i, T_j)$ is the correlation coefficient between $\eta_k(T_i) + \varepsilon_{ik}(T_i)$ and $\eta_k(T_j) + \varepsilon_{jk}(T_j)$; $\rho_\eta(T_i, T_j)$ is the inter-event correlation coefficient between $\eta_k(T_i)$ and $\eta_k(T_j)$; and $\rho_\varepsilon(\Delta_{ij}, T_i, T_j)$ is the intra-event correlation coefficient between $\varepsilon_{ik}(T_i)$ and $\varepsilon_{jk}(T_j)$. The inter-event residual is calculated as the overall deviation of spectral accelerations at multiple recording stations from a median prediction equation for a given seismic event, whereas the intra-event residual is evaluated as the deviation of spectral acceleration at a particular site from the event-based spectral acceleration. These correlations can be evaluated by statistical analysis of regression residuals (Goda and Hong, 2008; Goda and Atkinson, 2009, 2010).

Two spatial correlation models of seismic effects (i.e. $\rho_\eta(T_i, T_j)$ and $\rho_\epsilon(\Delta_{ij}, T_i, T_j)$ in Equation [28.11c]) are adopted in this study: the GH08 model (Goda and Hong, 2008) for shallow crustal earthquakes in North America, and the GA09 model (Goda and Atkinson, 2009, 2010) for Japanese earthquakes. The GH08 model may be suitable for shallow crustal earthquakes in western Canada, as it is based on statistical analysis of strong ground motion records from the PEER-NGA database (<http://peer.berkeley.edu/nga/index.html>), while the GA09 model may be applicable to in-slab and interface earthquakes in the Cascadia subduction zone, because many GMPEs for subduction earthquakes have been developed by including ground motion records from the K-NET and KiK-net databases (<http://www.kyoshin.bosai.go.jp/>). The details of the correlation models can be found in Goda and Hong (2008) and Goda and Atkinson (2009, 2010). Generally, the GA09 model predicts higher correlation than the GH08 model for the same separation distance, and the decay of the correlation coefficient as a function of separation distance is more gradual for the GA09 model than the GH08 model.

The above-mentioned analysis is carried out based on Monte Carlo simulation (see Chapter 1). Initially, earthquake occurrence models, seismic source zones, and magnitude-recurrence relations are used to produce a synthetic earthquake catalogue (see Panel 1 in Fig. 28.1). Then, by using suitable ground motion models and spatial correlation models of peak ground motions and response spectra, simultaneous seismic effects at different locations are generated for each seismic event (see Panel 2 in Fig. 28.1). These two steps generate probabilistic estimates of elastic seismic demands at multiple sites for many seismic events over a certain period (i.e. multiple seismic intensity maps). This information is used as input in seismic vulnerability analysis.

28.3.2 Seismic vulnerability and loss assessment of wood-frame houses

Structural vulnerability models quantify the extent of structural damage for a given seismic excitation, and have significant impact on seismic loss estimation. The first task is to collect comprehensive building inventory information, such as location, structural and material type, age, story number, occupancy type, floor area, value, and local soil condition (Panel 3 in Fig. 28.1). The building inventory database is the key input in seismic loss estimation, and determines overall accuracy of the assessment. Based on the collected building information, a suitable structural model and analysis method need to be selected by achieving a reasonable balance between accuracy and computational efficiency. Damage severity assessment is then carried out by directly comparing sustained inelastic seismic demand with

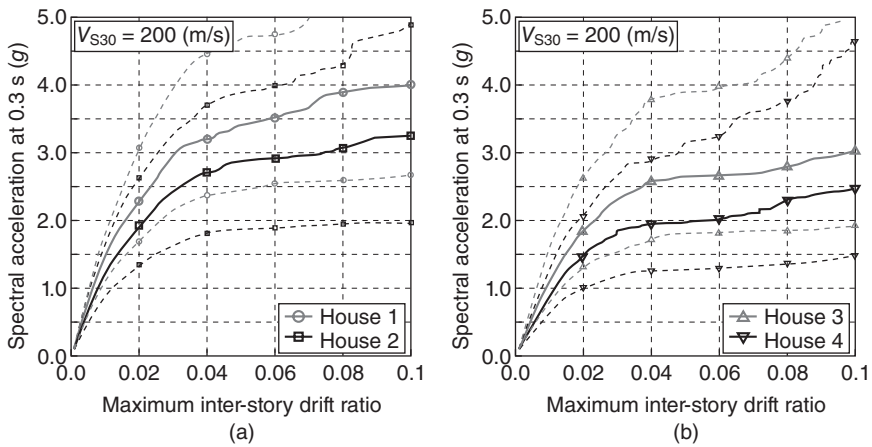
structural capacity (Panel 4 in Fig. 28.1). For an individual structure, the assessed damage severity is described by the damage factor δ , which is defined by:

$$\delta = \max \left[\min \left(\frac{D_D - D_Y}{D_C - D_Y}, 1 \right), 0 \right], \quad [28.12]$$

where D_D is the seismic demand variable, and D_Y and D_C are the yield and collapse capacity variables of a structure, respectively; these variables are expressed in terms of maximum inter-story drift ratio and are treated as random variables. The damage factor δ corresponds to no seismic damage, partial seismic damage, and complete seismic damage for $\delta = 0$, $0 < \delta < 1$, and $\delta = 1$, respectively.

The maximum inter-story drift ratio of conventional wood-frame houses in Vancouver (i.e. D_D) is estimated by using probabilistic models developed by Goda and Atkinson (2011). The structural models of the wood-frame houses (UBC-SAWS models) are based on SAWS (Seismic Analysis of Wood-frame Structures) by Folz and Filiatrault (2003) and were calibrated by researchers at University of British Columbia (UBC) using quasi-static and dynamic test results for typical wood-frame houses in Vancouver (Ventura *et al.*, 2002; White and Ventura, 2006). There are four UBC-SAWS models with different shear-wall characteristics: (i) House 1 has blocked plywood/oriented strand board shear-walls with exterior stucco cladding and gypsum wallboard interior finish; (ii) House 2 has blocked plywood/oriented strand board shear-walls with gypsum wallboard interior finish; (iii) House 3 has unblocked plywood/oriented strand board shear-walls with gypsum wallboard interior finish; and (iv) House 4 has horizontal boards with gypsum wallboard interior finish. These four models can be assigned to individual houses by utilising information on the year of construction. In this study, a house is classified as House 1, House 2, House 3, and House 4, if the year of construction is after 1996, from 1986 to 1995, from 1976 to 1985, and before 1975, respectively (Ventura *et al.*, 2005; note this classification is approximate, as discussed in Goda *et al.*, 2011).

The prediction models of the maximum inter-story drift ratio were developed by conducting incremental dynamic analysis (Goda and Atkinson, 2011); an estimate of the maximum inter-story drift ratio can be generated for a given value of spectral acceleration at 0.3 s. Note that the vibration period of 0.3 s is in the middle of the fundamental vibration periods of Houses 1 to 4 and spectral acceleration at 0.3 s was adopted as a seismic intensity measure in incremental dynamic analysis. The distinct features of the models include careful record selection and scaling based on the conditional mean spectrum (CMS) (Baker, 2011) by considering complex seismic hazard contributions from different earthquake sources in south-western British Columbia (i.e. crustal events, in-slab events, and interface



28.2 Incremental dynamic analysis curves of the four UBC-SAWS models: (a) Houses 1 and 2, and (b) Houses 3 and 4.

events). This method takes into account inter-period correlation of spectral acceleration ordinates for different earthquake types individually; such consideration is of importance when record scaling is involved for nonlinear dynamic analysis (Luco and Bazzurro, 2007; PEER, 2009). To show the impact of different shear-wall types, incremental dynamic analysis curves for Houses 1 to 4 are compared in Fig. 28.2 (solid lines are median curves, while broken lines are 16th/84th percentile curves). The seismic performance of the four UBC-SAWS models, as expected, can be ranked as House 1 > House 2 > House 3 > House 4. The results, as shown in Fig. 28.2, are then used to develop the probability distribution of the maximum inter-story drift ratio given spectral acceleration at 0.3 s by interpolating the empirical probability distribution function.

For seismic capacity modelling, reasonable estimates of D_Y and D_C in terms of maximum inter-story drift ratio can be found in FEMA/NIBS (2003) and Onur *et al.* (2006). Both D_Y and D_C are considered to be log-normally distributed. It is noteworthy that there exists significant uncertainty in the estimation of the collapse/ultimate capacity of wood-frame structures. In shake-table experiments, structures tested by White and Ventura (2006) did not collapse after experiencing the maximum inter-story drift ratio of 0.08–0.09, while Isoda *et al.* (2007) reported that structural collapse generally occurs after experiencing the maximum inter-story drift ratio of 0.15–0.20. Another relevant issue is that severely damaged but not-collapsed structures may be demolished in the rehabilitation process, because of infeasible repair operations from technical and economic viewpoints. By taking these situations into account, for Houses 1 to 4, a few cases

of the mean yield and collapse maximum inter-story drift ratios are considered to capture their epistemic uncertainty (see Table 28.1). The mean values are consistent with FEMA/NIBS (2003). The coefficients of variation for D_Y and D_C are set to 0.15 and 0.3, respectively.

For loss estimation, δ is transformed to seismic loss quantities through damage-loss functions. For convenience, damage costs are categorised into three types: repair/replacement costs related to damaged building parts $L_{RR}(\delta)$ (RR type), costs for lost contents $L_{CO}(\delta)$ (CO type), and costs related to business interruption $L_{BI}(\delta)$ (BI type). They are approximated by power functions:

$$L_{RR}(\delta) = C_{RR}\delta^{\beta_{RR}}, L_{CO}(\delta) = C_{CO}\delta^{\beta_{CO}}, \text{ and } L_{BI}(\delta) = C_{BI}\delta^{\beta_{BI}}, \quad [28.13]$$

where C_{RR} , C_{CO} , and C_{BI} are the replacement costs, and β_{RR} , β_{CO} , and β_{BI} are the exponent parameters for damage cost types RR, CO, and BI, respectively. To develop damage-loss functions that are applicable to wood-frame houses in Vancouver, the use type defined by the City of Richmond is related to the HAZUS-Earthquake occupancy type (FEMA/NIBS, 2003; Ventura *et al.*, 2005), and the exponent parameters of the damage-loss functions are determined by least squares curve fitting. The parameters for four prevalent occupancy types in Vancouver are summarised in Table 28.2. The unit replacement costs are considered to be lognormally distributed with

Table 28.1 Mean yield and collapse maximum inter-story drift capacity ratios

UBC-SAWS model	Yield capacity D_Y : drift ratio (weight)	Collapse capacity D_C : drift ratio (weight)
House 1	[0.004(0.25), 0.006(0.5), 0.008(0.25)]	[0.075(0.25), 0.1(0.5), 0.125(0.25)]
House 2	[0.004(0.5), 0.006(0.5)]	[0.05(0.25), 0.075(0.5), 0.1(0.25)]
House 3	[0.004(0.5), 0.006(0.5)]	[0.05(0.25), 0.075(0.5), 0.1(0.25)]
House 4	[0.002(0.5), 0.004(0.5)]	[0.05(0.5), 0.075(0.5)]

Table 28.2 Model parameters of damage-loss functions for different occupancy types

Occupancy	Replacement cost (CAD/ft ²): [C_{RR} , C_{CO} , C_{BI}]	Exponent parameter: [β_{RR} , β_{CO} , β_{BI}]
Single-family dwelling	[87.2, 21.8, 19.8]	[0.754, 0.681, 0.568]
Multi-family dwelling	[111.0, 27.8, 26.2]	[0.810, 0.681, 0.617]
Store/commercial service	[47.6, 26.4, 23.8]	[0.808, 0.681, 0.426]
Restaurant	[149.4, 74.7, 228.1]	[0.851, 0.681, 0.541]

the coefficient of variation of 0.3. The (mean) unit replacement costs are estimated based on FEMA and NIBS (2003), and do not necessarily reflect market/taxation values.

Finally, seismic loss estimation can be conducted based on Monte Carlo simulation to obtain samples of annual aggregate seismic loss L_A for all buildings and for all seismic events included in the synthetic earthquake catalogue. Specifically, L_A is calculated by:

$$L_A = \sum_{k=1}^{n_A} \sum_{i=1}^m L_{EQ(i)}(\delta_{ik}) = \sum_{k=1}^{n_A} \sum_{i=1}^m [L_{RR}(\delta_{ik}) + L_{CO}(\delta_{ik}) + L_{BI}(\delta_{ik})], \quad [28.14]$$

where n_A is the number of earthquakes in a year; m is the number of buildings in the portfolio; and $L_{EQ(i)}(\delta_{ik})$ is the seismic loss to the i th property in the portfolio due to the k th event. For simplicity, seismic damage is considered to be repaired immediately after a seismic event. The generated seismic loss samples can be used to construct the probability distribution of L_A (i.e. seismic loss curve) and to identify significant scenario events through deaggregation analysis (Panel 5 in Fig. 28.1). Moreover, by applying an insurance policy, seismic loss samples incurred by an insurer can be obtained as follows:

$$L_{A,I} = \sum_{k=1}^{n_A} \sum_{i=1}^m I_P[L_{EQ(i)}(\delta_{ik})] = \sum_{k=1}^{n_A} \sum_{i=1}^m I_P\{L_{RR}(\delta_{ik}) + L_{CO}(\delta_{ik}) + L_{BI}(\delta_{ik})\}, \quad [28.15]$$

where $I_P(\bullet)$ is the insurance pay-out function and is given by:

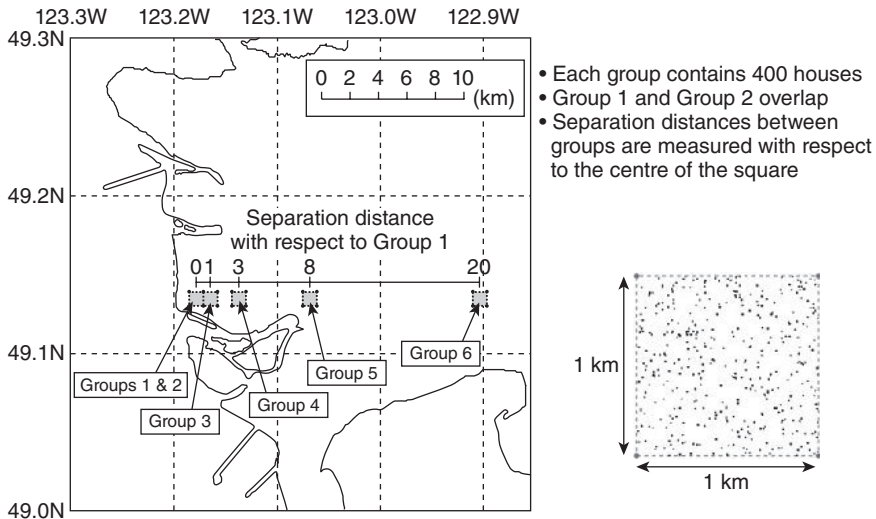
$$I_P(L_{EQ(i)}) = \begin{cases} 0 & L_{EQ(i)} \leq D \\ \gamma(L_{EQ(i)} - D) & D < L_{EQ(i)} < C, \\ \gamma(C - D) & L_{EQ(i)} \geq C \end{cases}, \quad [28.16]$$

in which D , C and γ are the deductible, cap, and coinsurance factor, respectively. The parameters can be adjusted to design various kinds of insurance pay-out structures to transfer undesirable portions of the probability distribution of seismic loss per property to the insurer. Using Equation [28.15], the seismic risk exposure curve for an insurer can be developed.

28.4 Earthquake insurance portfolio analysis

28.4.1 Building portfolios of wood-frame houses in Vancouver

To generate seismic loss samples from the earthquake-engineering-based model described in Section 28.3, a hypothetical but realistic portfolio of conventional wood-frame houses in south-western British Columbia is constructed. The portfolio includes 2400 wood-frame houses and is divided into



28.3 Locations of building groups in Vancouver.

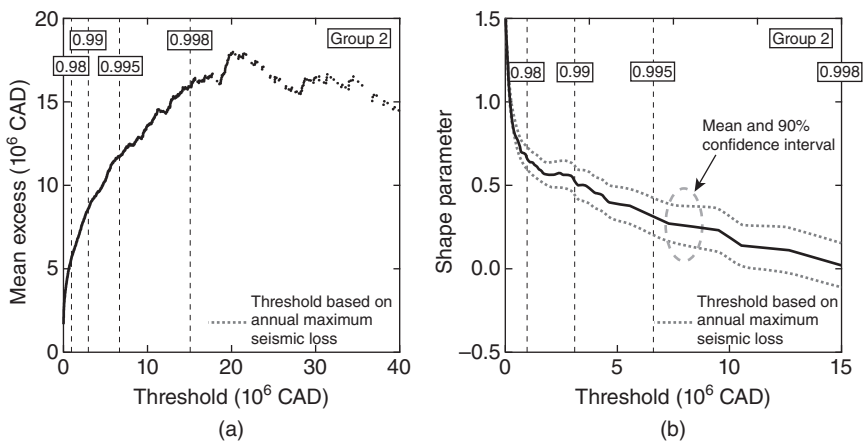
six groups. Each group contains 400 houses, which are randomly and uniformly distributed within a 1 km by 1 km square area. To investigate the effects of separation distance between two groups on seismic loss dependence, groups 2–6 are located at separation distances of 0, 1, 3, 8, and 20 km east to group 1 (see Fig. 28.3). The portfolio is based on an existing building inventory of 1415 wood-frame houses located in the City of Richmond, which was used by Goda *et al.* (2011). The occupancy is mostly residential for single-family dwelling, and the original inventory database does not include large-size wood-frame structures with square footage greater than 3500 ft². The structural characteristics of individual 2400 houses (i.e. building area, height, occupancy, and construction year) are randomly selected from those of the 1415 existing houses in Richmond. Therefore, the generated portfolio of the 2400 houses resembles regional characteristics of wood-frame houses in Richmond. The local soil condition in the City of Richmond is close to the NEHRP site class D/E boundary, with V_{S30} ranging from 150 to 250 m/s (Hunter *et al.*, 1998). V_{S30} is modelled as a lognormal variate; for all locations, the average V_{S30} of 200 m/s and the coefficient of variation of 0.075 are adopted.

28.4.2 Statistical modelling of earthquake risk exposure

The objective of this section is to illustrate how the proposed statistical modelling approach based on the GP model and copula can be applied to

seismic loss samples generated from the earthquake-engineering-based model. The input seismic loss samples are generated for a period of 100 000 years. Both marginal distribution fitting and copula function fitting for several pairs of groups have been carried out. For illustration, seismic loss data L_A for Groups 2 and 3 are focused on. (The insurance policy, as in Equation [28.16], is not applied for the base case.) For marginal distribution modelling, results for Group 2 (which will be discussed in the following) are generally applicable to other groups, because the building portfolio is statistically equivalent and seismic hazard characteristics do not vary significantly for the six groups.

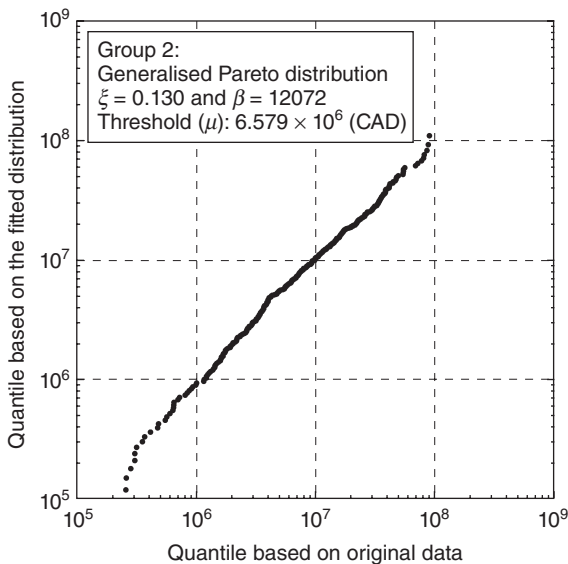
The first step is to determine a suitable value of the threshold μ above which seismic loss data can be approximated by the GP model. Two diagnostic tools, mean excess plot and shape parameter plot, are employed, and the results for Group 2 are presented in Fig. 28.4. Figure 28.4a shows that the trend of mean excess plot is increasing and a linear straight line can be fitted for seismic loss data exceeding a threshold of about 4–5 million CAD. Figure 28.4b indicates that the shape parameter ξ is gradually decreasing as the threshold level increases. For very large threshold values, the mean excess plot becomes flat (Fig. 28.4a), while the shape parameter approaches zero (Fig. 28.4b). These results indicate that the degree of heavy right tail becomes less extensive. (Note that the reliability of such diagnostic analyses decreases with the threshold μ , because the number of used data points decreases rapidly.) On the other hand, it should also be reminded that these diagnostic tools provide loose guidance on the adequate level of μ only (Cebrian *et al.*, 2003; McNeil *et al.*, 2005); the final decision should be made



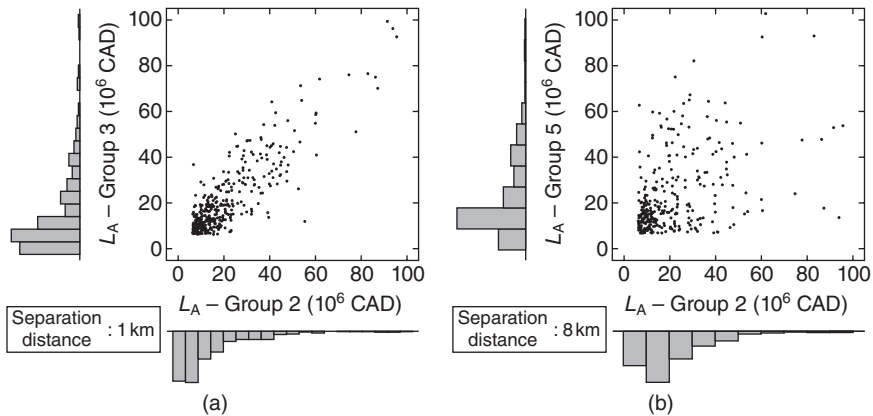
28.4 (a) Mean excess plot for Group 2, and (b) shape parameter plot for Group 2.

by inspecting the fitness of the POT data to the GP model. Another reason for decreasing right tail may be attributed to the limitation of the adopted earthquake-engineering-based model, which does not incorporate, for example, the demand surge effects on repair costs and the indirect ripple effects across local and regional economy. Thus the total economic seismic loss is likely to be underestimated, when very severe seismic damage occurs to a large number of buildings and infrastructure in a region. Based on preliminary investigations, a threshold level corresponding to 0.995 annual probability of non-exceedance (i.e. return period of 200 years) is chosen for the seismic loss data for Group 2 (i.e. 6.579 million CAD).

To show the marginal distribution fitting of the GP model to the POT data for Group 2, a Q–Q plot is presented in Fig. 28.5, and the estimated parameter values are indicated in the figure. The Q–Q plot of the POT data shows that the marginal distribution fitting based on the GP model for Group 2 is adequate. This is an improvement, as good fitting is achieved using a simpler model with statistical foundation rooted in extreme value theory, in comparison with the previous investigation by Goda and Ren (2010), where difficulty was encountered in finding a suitable parametric distribution function for an entire data range. (Note that it was found during preliminary investigations that the gamma distribution fits the POT data equally well.) The Q–Q plot shows that the chosen threshold value, related to Fig. 28.4, was indeed adequate.



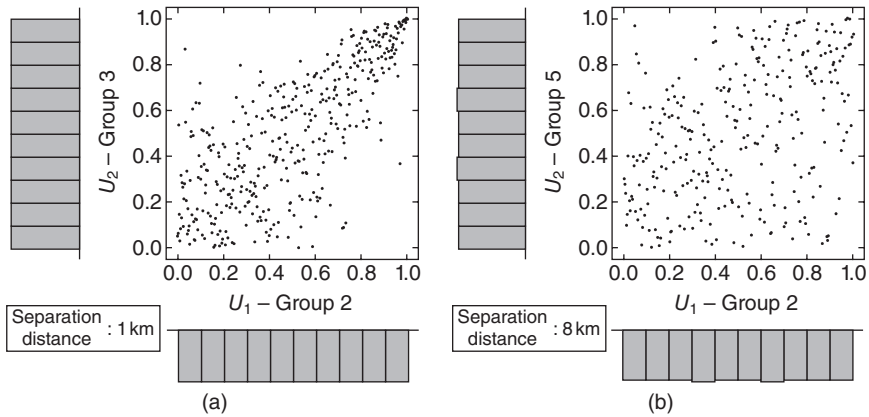
28.5 Quantile–quantile plot of seismic loss data for Group 2.



28.6 Scatter plot and marginal distribution plots of joint POT data: (a) Groups 2 and 3 (1 km separation), and (b) Groups 2 and 5 (8 km separation).

The second step of the method is to investigate dependence characteristics of the joint POT data. In this study, POT data points that exceed the respective threshold values simultaneously are considered; this is the same approach adopted by Dupuis and Jones (2006). The focus is deemed to be reasonable, because the developed statistical model will be used to evaluate risk measures related to very rare events, e.g. calculation of risk exposure of a reinsurance policy where the retention level is generally much higher than the adopted threshold level of the GP model. To inspect characteristics of the seismic loss data for Groups 2 and 3 as well as Groups 2 and 5, scatter/ plots for the two cases are presented in Fig. 28.6. The average separation distance between Groups 2 and 3 is 1 km, while that between Groups 2 and 5 is 8 km; therefore, based on physical features of ground motions, the correlation of seismic loss data for Groups 2 and 3 is greater than that for Groups 2 and 5. Inspection of Fig. 28.6 indicates that a tendency of upper tail dependence for Groups 2 and 3 is evident (i.e. occurrence of a large loss for one group is accompanied by a counterpart for the other group), while the extent of upper tail dependence for Groups 2 and 5 is insignificant. To examine the tail dependence characteristics evidenced in Fig. 28.6, the POT data are transformed to the copula samples (i.e. $C^{\text{Emp}}(u_1, u_2)$ as in Equation [28.6]), and scatter plots in terms of uniform marginals are shown in Fig. 28.7. In Fig. 28.7, it is clearly seen that the right upper tail dependence exists for Groups 2 and 3, while such dependence is not obvious for Groups 2 and 5.

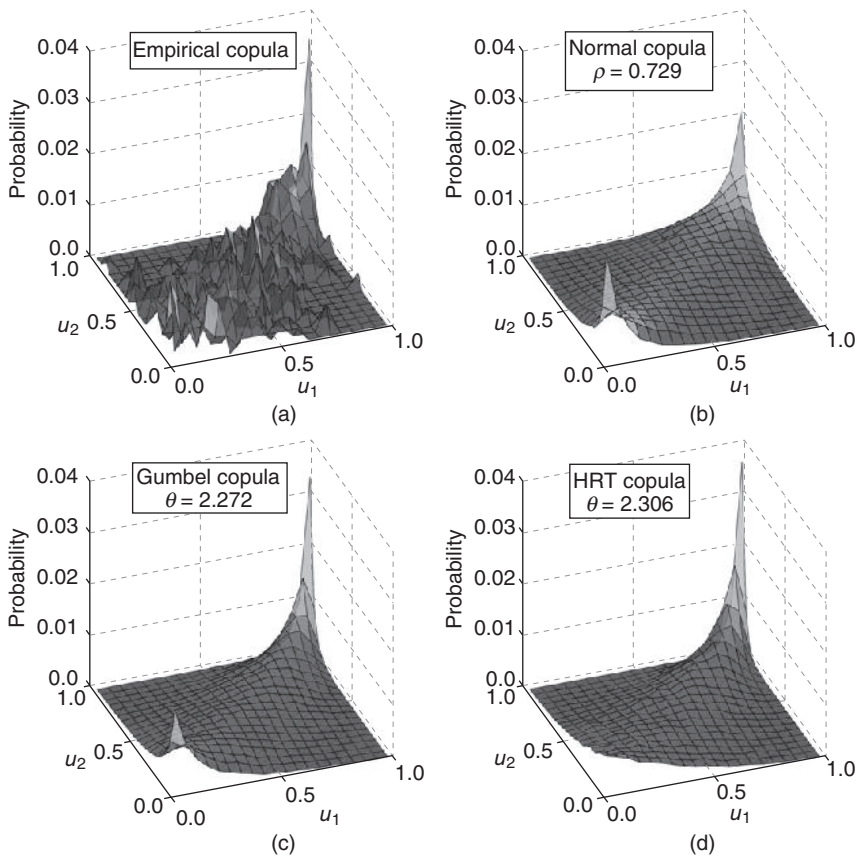
To complete the analysis, copula fitting is carried out by fitting the four parametric copula models mentioned in Section 28.2.2, i.e. normal, t , Gumbel, and HRT copulas, to the transformed copula samples (i.e. pairs of



28.7 Scatter/plot and marginal distribution plots of transformed copula samples: (a) Groups 2 and 3 (1 km separation), and (b) Groups 2 and 5 (8 km separation).

u_1 and u_2 shown in Fig. 28.7). The model fitness evaluated based on the AIC value indicates that the HRT copula is superior to other copulas consistently (but the Gumbel copula achieved a similar AIC value). This is in agreement with the finding by Goda and Ren (2010). To visually inspect the copula fitting, bivariate copula density plots of the empirical copula samples and the three fitted normal, Gumbel, and HRT copulas for the joint POT data for Groups 2 and 3 (i.e. Fig. 28.7a) are compared in Fig. 28.8. (A plot for the t copula is not shown because it is very similar to the one for the normal copula.) It is observed that the empirical copula has strong upper tail dependence; the normal copula has the symmetrical moderate upper and lower tail dependence; the Gumbel copula has strong upper tail dependence and weak lower tail dependence; and the HRT copula has strong upper tail dependence only. These plots confirm the adequacy of the Gumbel and HRT copulas for characterising seismic loss dependence for Groups 2 and 3, which is highly correlated in the upper tail. The above steps complete the statistical modelling of joint seismic loss data based on extreme value theory and copula. The fitted model can be used to simulate seismic loss samples.

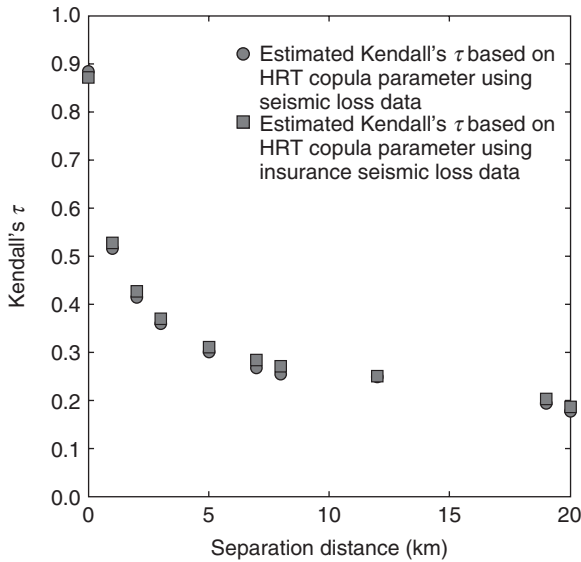
Moreover, to investigate seismic loss dependence in terms of separation distance, the preceding analysis is repeated for other pairs of groups with different separation distances (see Fig. 28.3). As a measure of seismic loss dependence, the Kendall's τ coefficient is useful, because it is invariant under strictly increasing transformations of random variables (unlike the Pearson's linear correlation coefficient) and can be calculated from the (estimated) HRT copula parameter as $\tau = \theta/(\theta + 2)$ (see Equation [28.10]). Figure 28.9 shows the estimated Kendall's τ coefficient as a function of



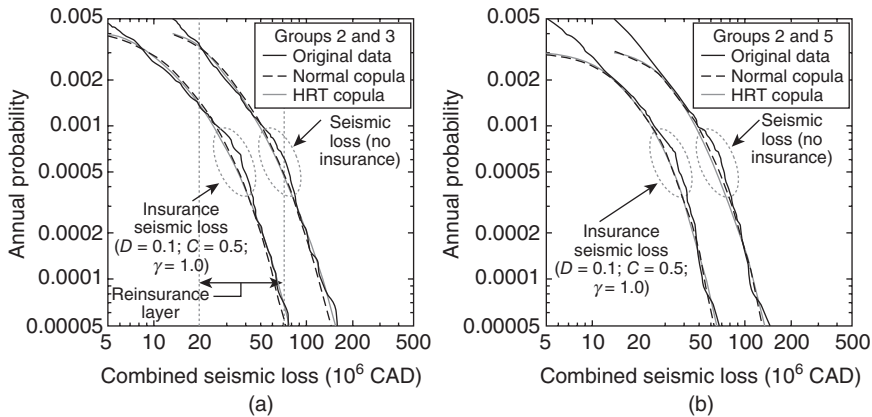
28.8 Comparison of copula density plots using joint POT data for Groups 2 and 3: (a) empirical copula, (b) normal copula, (c) Gumbel copula, and (d) HRT copula.

separation distance (i.e. different pairs of groups); it becomes very high for short separation distance (e.g. $\tau = 0.9$ when two portfolios overlap), and decreases gradually as the separation distance increases. In addition, Fig. 28.9 includes results for insurance seismic loss data (i.e. samples are obtained by applying Equation [28.16] with $D = 0.1$, $C = 0.5$, and $\gamma = 1.0$). The same tendency is seen for original seismic loss data and insurance seismic loss data.

Finally, to verify the capability and accuracy of the proposed statistical method, seismic loss samples for Groups 2 and 3 as well as Groups 2 and 5 are generated by using the fitted GP marginal distributions and the copula functions. Then, the combined seismic loss samples are computed by adding the samples for two groups (i.e. portfolio aggregation), and are used for



28.9 Estimated Kendall's τ coefficient of seismic loss data as a function of separation distance.



28.10 Comparison of the target original seismic loss samples (from the earthquake-engineering-based model) with simulated seismic loss samples from the fitted statistical models: (a) Groups 2 and 3, and (b) Groups 2 and 5.

verification. The simulated seismic loss samples from the statistical models are compared with original samples from the earthquake-engineering-based model (i.e. input data to the statistical modelling). The results are shown in Fig. 28.10; the combined seismic loss curves based on the normal and HRT copulas are included. Inspection of Fig. 28.10 indicates that the

simulated samples from the fitted statistical models (both normal and HRT copulas) match fairly well with the input samples; qualitatively, fitting of the HRT copula is slightly superior to the normal copula (however, visually, for the considered case, both models perform very well). Note that significant deviation around the annual probability level of 0.003 is due to the adopted threshold value for determining the POT data. Therefore, for right tail data, accuracy of the proposed model using the GP distribution and copula in approximating the results from the earthquake-engineering-based model is judged to be good. To further demonstrate the applicability of the proposed method, the same exercise (i.e. marginal and copula fitting and simulation from the fitted statistical model) is carried out by using insurance seismic loss samples by applying Equation [28.16] with $D = 0.1$, $C = 0.5$, and $\gamma = 1.0$. The results are also included in Fig. 28.10; the proposed statistical method performs well for the insurance input data. Generally, the above conclusions are applicable to other cases (e.g. various combinations of groups with different separation distances as well as different insurance policy arrangements). It is noteworthy that the simulation from the fitted statistical model is very quick (less than one minute), in comparison with the earthquake-engineering-based model (several days). Therefore, once the accuracy of the method is verified, the proposed method can serve as a viable alternative to conduct various sensitivity analyses.

28.4.3 Assessment of reinsurance risk exposure

The developed statistical models of bivariate insurance seismic loss data can be useful for evaluating risk measures related to reinsurance. For illustration, consider that a reinsurer covers insurer's seismic loss layer between 20 and 70 million CAD from the combined portfolio of Groups 2 and 3. The insurance arrangement is set to: $D = 0.1$, $C = 0.5$, and $\gamma = 1.0$ (i.e. insurance seismic loss case in Fig. 28.10a). The annual probabilities of exceedance for the loss threshold values are 0.00135 and 0.00007, respectively (Fig. 28.10a). The annual expected payment for the reinsurance layer is calculated as 15.474, 12.854, and 13.367 million CAD for the original data, normal copula case, and HRT copula case, respectively. Similarly, the standard deviation of the annual reinsurance payment is calculated as 11.360, 11.251, and 11.743 million CAD for the original data, normal copula case, and HRT copula case, respectively. These quantities are the fundamental input in determining reinsurance premium. Overall, the calculated statistics for the normal and HRT copulas are comparable to those for the original data, and the statistical seismic loss models can approximate the original data well.

VaR and TVaR are popular and practical risk measures for insurance portfolio management. VaR is the fractile corresponding to small probability of exceedance, whereas TVaR is the conditional expected value above

Table 28.3 Ratio of risk measures VaR and TVaR based on the statistical models with respect to those based on the earthquake-engineering-based model for insurance seismic loss samples for Groups 2 and 3

Risk measure	Probability level	GP model and normal copula	GP model and HRT copula
Ratio of VaR	0.0010	0.9485	0.9288
	0.0005	0.8907	0.8965
	0.0002	0.9503	0.9845
	0.0001	0.9425	0.9916
Ratio of TVaR	0.0010	0.9676	1.0075
	0.0005	1.0468	1.0953
	0.0002	1.0402	1.0856
	0.0001	1.0770	1.1442

the selected fractile. To investigate the effects of using the fitted seismic loss models instead of the earthquake-engineering-based model quantitatively, ratios of VaR and TVaR based on the fitted models to those based on the earthquake-engineering-based model are calculated at four exceedance probability levels between 0.0001 and 0.001, and the results are summarised in Table 28.3. Biases of the estimated VaR for the normal and HRT copula models are typically less than 10% error with respect to the earthquake-engineering-based model (deemed as reasonable). Inspection of the ratios for TVaR suggests that the fitted models have heavier right tail than the earthquake-engineering-based model (this is more applicable to the HRT copula case). However, this observation needs to be interpreted carefully, because the lighter right tail of the earthquake-engineering-based model (more precisely, gradual decrease of heavy right tail behavior) may be caused by the limitation of the adopted seismic loss model, such as ignorance of the compounding indirect seismic loss (which tends to amplify seismic loss significantly).

Overall, it is concluded that the developed statistical seismic loss models achieve satisfactory approximation to the earthquake-engineering-based model, and can be used for complex insurance portfolio analysis. The use of the developed statistical models is advantageous, because the computational burden is only a small fraction of the earthquake-engineering-based model.

28.5 Conclusions and future trends

Extreme value theory for peak-over-threshold data offers a solid theoretical basis to characterise seismic losses caused by catastrophic earthquakes. The close attention on large seismic loss data is particularly important for

risk managers of insurance and reinsurance portfolios. Furthermore, it is desirable to be equipped with techniques for portfolio aggregation, where the consideration of adequate dependence among different parts of a portfolio is of high importance. To deal with such problems, a statistical approach based on the marginal generalised Pareto models together with a versatile copula-based dependence structure was investigated. Using a numerical example, effectiveness of the proposed method was demonstrated in jointly approximating very large seismic losses from two different building portfolios. In particular, the proposed approach is useful for assessing risk exposure of a reinsurance policy with a high retention level. The fast computation with reasonable accuracy is an attractive feature of the method.

In the future, applications of multivariate modelling based on the copula technique will be more popular, because of its capability/flexibility by dealing with marginal models and dependence model separately. Such applications should not be limited to earthquake engineering and should be transferred to any fields/problems that involve with multivariate random data. In line with the present example (i.e. modelling of seismic loss data from different insurance portfolios), extension of the current approach to more than two portfolios is an important step. Another technical challenge is the applications to numerous buildings and infrastructure facilities/components (e.g. more than 50000 structures simultaneously) – this is not easy because the simulation of seismic intensities at a large number of locations requires a huge covariance matrix and takes significant time and computer memory. Some approximation framework/procedure needs to be adopted (e.g. hierarchical structure of buildings and system components and adequate aggregation method). Lastly, a significant challenge in seismic loss estimation methodology is the accurate assessment of indirect seismic damage costs, such as demand surge effects on building repairs/reconstruction, prolonged and inter-related business interruption within a supply chain, and stagnant local economy, slowing down recovery from disasters. These will affect the probabilistic characteristics of aggregate seismic loss for rare cases – in such cases, capability of capturing accurate joint right tail behaviour would be even more important.

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Probabilistic assessment of earthquake insurance rates for buildings

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Abstract: A simple probabilistic model is presented to obtain a realistic estimate of earthquake insurance rates for buildings in Turkey. The model integrates information on seismic hazard and information on expected earthquake damage on engineering facilities in a systematic way, yielding estimates of earthquake insurance rates. Earthquake insurance rates are computed for reinforced concrete buildings constructed in different seismic zones of Turkey with the purpose of checking the validity of the currently charged earthquake insurance rates. The actual implementation of earthquake insurance is also discussed in this chapter. In this respect, an overview of the compulsory earthquake insurance scheme is presented, and the organizational structure and operational details of Turkish Catastrophic Insurance Pool (TCIP) are described.

Key words: earthquake insurance, damage probability matrix, insurance premium, seismic hazard, TCIP.

29.1 Introduction

In most countries, earthquake insurance programs are implemented with the purpose of accumulating funds to cover post-disaster expenditure for future earthquakes. The practice and regulations of earthquake insurance vary from country to country. Below, brief descriptions for some countries, where seismic hazard is significant, are presented. The impact of the two major earthquakes in 1999 in the Marmara region caused the government of Turkey to establish the Turkish Catastrophe Insurance Pool (TCIP) in September 2000 and earthquake insurance was made compulsory for dwellings. The insurance system has five tariff zones and also charges different premium rates depending on the construction type (steel, reinforced concrete, masonry, and others). The rates charged by the insurance companies, as specified by TCIP, range from 5.50‰ to 0.44‰ of insured property. The scheme has a deductible of 2% applied on the insured value of the property.

The obligatory earthquake insurance system established in Turkey is highly rated by national and international insurance authorities and

currently holds the second largest number of policies in the world. The penetration is about 26% as of September 2012. The market penetration, also known as the take-up-rate, is defined as the percentage of the insured versus the maximum number of possible insured (Scawthorn *et al.*, 2003). More detailed information on TCIP and the compulsory earthquake scheme is provided in Section 29.4.

In Japan, earthquake insurance has been available since the 1950s (Kamei, 1976). Earthquake insurance also covers damage from volcano eruptions and tsunamis. The buildings are classified according to their earthquake resistance capacities and in this way construction with poor quality and design is discouraged. The earthquake insurance for homes is an endorsement to fire coverage, and reinsurance is provided by the government (Brillinger, 1992; Scawthorn *et al.*, 2003). According to the 2010 data provided by the Japanese Earthquake Reinsurance Co. Ltd (JER), the penetration has exceeded 25% (Monti, 2011). In California, a state-run earthquake insurance company, called as the California Earthquake Authority (CEA), was formed in 1994 to provide coverage for homeowners. The all-residential CEA program was formed in order to overcome some of the difficulties encountered by the insurance companies after the 1994 Northridge earthquake (Scawthorn *et al.*, 2003).

In New Zealand, the earthquake insurance is a part of fire insurance. The Earthquake Commission (EQC), which was formed in 1993, is New Zealand's primary provider for seismic disaster insurance to residential property owners. For homes, there is a limit of 100 000 NZ dollars of coverage. A standard rate of 0.5‰ has been charged since 1945 (Scawthorn *et al.*, 2003). The New Zealand reinsurance program is one of the largest catastrophe coverages in the world.

As a part of a comprehensive disaster prevention and risk management program, the Ministry of Finance of the Government of the Chinese Taipei introduced the Residential Earthquake Insurance Pool (TREIP). Prior to the establishment of TREIP, earthquake insurance was provided as an endorsement to a long-term residential fire policy. Since April 2002, new residential fire policies have been issued on an annual (rather than long-term) basis, and have been charged automatically to cover earthquake risk. As of August 2010, the take-up-rate was approximately 28% of the estimated eight million households in Chinese Taipei (Monti, 2011).

For more information on different practices in various countries, the reader is referred to *Natural Catastrophes Insurance Cover: A Diversity of Systems* (Insurance Compensation Consortium of Spain, 2008), where a comprehensive review of different systems employed all over the world for the insurance cover of natural disasters was presented. A number of publications by the Organization for Economic Co-operation and Development (OECD) (2005, 2008, 2010) also provide information on different aspects

of earthquake insurance as well as on the financial management of large-scale catastrophes.

Although the concepts of earthquake risk were developed as early as the 1930s (Freeman, 1932), studies on the assessment of earthquake insurance premiums, based on statistical methods and utilizing earthquake engineering concepts, are limited in number. Gurpinar *et al.* (1978) and Gurpinar and Yucemen (1980) considered the problems related to obligatory earthquake insurance implementation in Turkey, and a pilot application was carried out for a city in Turkey. Yucemen (2005) developed a statistical model for the estimation of earthquake insurance rates, whereas Yucemen (2005), Deniz and Yucemen (2007, 2009), and Yucemen (2007) applied this model for the assessment of earthquake insurance rates for different seismic zones in Turkey. Smolka and Berz (1989, 1991) developed a methodology for obtaining insurance premiums consistent with seismic risk and potential loss due to large earthquakes. Other major studies conducted in the field of earthquake insurance include Straub (1973), Vere Jones (1973), Lockett (1980), Brillinger (1992), Kunreuther (1996), Walker (2000), Amendola *et al.* (2000), Scawthorn *et al.* (2003), Yucemen *et al.* (2008), and Goda and Yoshikawa (2011).

This chapter aims to present a simple probabilistic model for the assessment of earthquake insurance rates by considering damage statistics and potential seismic hazard. The proposed model is used for the assessment of earthquake insurance rates for reinforced concrete buildings constructed in different seismic zones of Turkey. The earthquake insurance premiums are computed by considering the risk of shaking-related damage only; secondary losses resulting from fire induced by an earthquake are omitted. Within the context of this chapter, it is also aimed to give an idea about the actual implementation of earthquake insurance in Turkey. Accordingly, in Section 29.4, an overview of the compulsory earthquake insurance scheme is given and background information on TCIP is presented together with the objectives foreseen in establishing the TCIP and its organizational structure.

29.2 Probabilistic model for the assessment of earthquake insurance rates

Earthquake insurance rates should be calculated based on the frequency and the severity of earthquakes. This corresponds to a conditional probability of damage given a range of earthquake hazard levels. The frequency of earthquakes at a site will be the same for all structures. However, the severity of damage depends on the structural system type, age, configuration, and other features. Hence, severity of damage to different facility classes should be considered separately. Accordingly the assessment of earthquake

insurance premiums requires two types of investigation, namely: seismic hazard analysis and estimation of potential earthquake damage to structures. In the following subsections, the models used for these two types of investigation are briefly discussed and a simple probabilistic model developed for the assessment of earthquake insurance rates is presented. The model is based on the integration of earthquake hazard with structural damage through matrix multiplication where the elements of seismic hazard exceedance values are multiplied by the appropriate elements of the damage probability matrices.

29.2.1 Seismic hazard analysis

In the probabilistic term, seismic hazard can be defined as the probability of exceeding different levels of a selected earthquake severity parameter (peak ground motion parameters, intensity, etc.) at a given place and within a given period of time due to expected seismic activity in the region. The future earthquake threat at a site is generally quantified by carrying out either a deterministic or probabilistic seismic hazard analysis. Considering aleatory uncertainties related to earthquake occurrences with respect to time, space, magnitude, and additional epistemic uncertainties, probabilistic models appear to be more appropriate than deterministic ones. Besides the popularity of the probabilistic approach, recently more attention has been paid to the assessment of seismic hazard due to active faults. Accumulation of more data and information on the main characteristics of faults has also contributed to this. Accordingly, in the evaluation of the seismic hazard component of the earthquake insurance, more attention should be given to the potential threat posed by faults, and the stochastic models consistent with the known characteristics of faults should be utilized.

Many models have been developed for probabilistic seismic hazard analysis (PSHA; see also Chapter 1). Most of the earlier models of seismic hazard assessment were based on the assumption that earthquake occurrences are independent events in space and time, utilizing the Poisson model (Cornell, 1968) or the extreme value statistics (Milne and Davenport, 1969). Later studies considered the temporal or spatial dependence of earthquakes. Some models consistent with the 'elastic rebound theory' took into account the temporal dependence of earthquakes based on the processes with Markovian characteristics. Attempts were also made to model the spatial dependence of earthquakes. In recent studies, the occurrence of earthquakes is treated as a space–time process and modeled as a random field (e.g. Yucemen, 1993; Akkaya and Yucemen, 2002).

The probabilistic formulation adopted in this study is based on the standard PSHA model. The PSHA method that will be employed for the assess-

ment of the earthquake insurance rates consists of the following basic steps (Yucemen and Ozturk, 2008):

1. A seismic database is compiled using catalogs which list information on past earthquakes that have occurred in the region of interest. The raw data in the earthquake catalogs have to be processed considering the following: magnitudes reported in different scales have to be converted to a common scale, preferably to the moment magnitude (M_w) scale. Earthquake clusters are identified and dependent events (fore- and after-shocks) are eliminated by defining appropriate space and time windows (Deniz, 2006). Finally, an analysis of catalog completeness is performed to obtain complete rates (i.e. complete number of events over a particular time period).
2. The region is divided into discrete seismic sources in the form of lines and areas. This involves preparation of a fault map, identification of active faults and assessment of the parameters of active faults. The basic information required for an active fault is as follows: actual location, coordinates of starting and ending points, activity rate, type, orientation (strike, dip amount, and direction), length, segmentation, age, total horizontal/vertical slip amount, annual slip rate, the magnitude and return period of the characteristic earthquake, and date of the last characteristic earthquake.
3. A seismotectonic map showing the distribution of earthquake epicenters and their relationships with active faults should be plotted. Such a map also displays distribution of past earthquake data to the different seismic sources consistent with their epicentral locations. Based on the data, recurrence relationships that provide information on the relative frequency of occurrence of earthquakes with different magnitudes are obtained and the seismicity parameters are assessed for each seismic source. Areal background seismic sources are identified/developed to include the contribution of earthquakes that could not be associated with any one of the faults. For this purpose, a spatially smoothed seismicity model (Frankel, 1995) and background areal seismic source with uniform seismicity can be used.
4. Selection of the appropriate probabilistic and stochastic models for the earthquake magnitude distribution and the earthquake occurrence in the time domain. In the PSHA, earthquake occurrences are assumed to exhibit a Poisson process and magnitudes to be distributed exponentially (i.e. Gutenberg–Richter law). These two assumptions are still widely employed in current seismic hazard studies. However, the renewal process (Ellsworth *et al.*, 1999) and the characteristic earthquake model (Schwartz and Coppersmith, 1984) can be considered as alternatives to the Poisson process and the Gutenberg–Richter relation-

ship, respectively. Moreover, stochastic models corresponding to different combinations of these alternatives should be developed. In the estimation of the parameters a and b of Gutenberg–Richter’s magnitude–frequency relationship, the modified maximum likelihood (MML) method should be preferred. The MML estimators are more precise and robust, compared to the least squares (LS) and maximum likelihood (ML) estimators, especially when the error distribution is non-normal (Yucemen and Akkaya, 2012).

5. An adequate set of intensity attenuation equations is developed and/or selected.
6. Preparation of a computational algorithm which aggregates the seismic threat nucleating from different sources, yielding the probability distribution for the earthquake intensity at a specified location. Generally, a series of computer programs are utilized to perform the necessary computations.
7. To account for alternative assumptions and data/parametric/model uncertainties, different sources of uncertainties (aleatory and epistemic) are characterized and incorporated either directly in the hazard calculations or by conducting sensitivity studies through logic tree or statistical methods.
8. Seismic hazard curves are developed by computing exceedance probabilities for different earthquake intensity levels.

29.2.2 Estimation of potential earthquake damage

The other component of the probabilistic model involves the assessment of seismic vulnerability of buildings. Damage is commonly described by a loss ratio that varies with the strength of shaking and type of structure (Whitman, 1973; Blong, 2003a; Askan and Yucemen, 2010). Due to uncertainties involved, damage that may occur during future earthquakes should be treated in a probabilistic manner.

The most reliable data source for earthquake damage estimation is the observed damage data, provided that personal bias in the damage evaluation is eliminated. However, when there is lack of adequate data, missing or unreliable parts of the available data can be complemented based on analytical methods and/or expert opinion.

The expected earthquake damage can be presented in various ways. In the current study, the damage probability matrix (DPM) proposed by Whitman (1973) is adopted. This is based on the facts that many past studies used this method consistently (e.g. Steinbrugge, 1982; Yucemen, 2005). Note that the damage probability matrix approach facilitates the conversion of the observed damage data into a probabilistic format. The general form of a DPM is shown in Table 29.1.

Table 29.1 Damage probability matrix

Damage state (DS)	Central damage ratio (CDR) %	Intensity				
		V	VI	VII	VIII	IX
None	0	Damage state probabilities $P(\text{DS}, I)$				
Light	5					
Moderate	30					
Heavy	70					
Collapse	100					

Source: Adapted from Yucemen (2005).

A DPM describes what will happen to buildings, designed according to some particular set of requirements, during earthquakes of various intensities (Whitman, 1973; ATC, 1985). An element of this matrix $P_k(\text{DS}, I)$ gives the probability that a particular damage state (DS) occurs when the structure of k -type is subjected to an earthquake of intensity, I . The identification of damage states is achieved in two steps (Yucemen, 2005).

1. The qualitative description of the degree of structural and non-structural damage by words. Generally, five levels of damage states are specified (Deniz and Yucemen, 2009): no damage (N), light damage (L), moderate damage (M), heavy damage (H), and collapse (C) states. This categorization of damage states is also used in this study.
2. The quantification of the damage described by words in terms of the damage ratio (DR), which is defined as the ratio of the repair cost due to earthquake damage to the replacement cost of the building (excluding the value of the land on which the building is constructed).

Specifying a range of DRs instead of a single DR value for each DS makes more sense from an engineering perspective to capture a range of uncertainties associated with the assignment of DRs. However, for computational convenience, a single DR (usually the average of the corresponding range), called the central damage ratio (CDR), is assigned to each DS to represent the best estimate DR of buildings in that DS (Blong, 2003b; Gulpinar and Yucemen, 1980). Based on interviews with experts who are in charge of damage evaluation and based on similar studies, the damage ratios corresponding to the five damage states were estimated by Gulpinar *et al.* (1978). Based on these values the CDRs are computed and shown in Table 29.1. In the present study, only DPMs for conventional reinforced concrete frame buildings are considered, and the corresponding matrices are constructed from observational and estimated data available in Turkey.

Using the available post-earthquake damage data, each element of a DPM can be obtained from

$$P_k(\text{DS}, I) = \frac{N_k(\text{DS}, I)}{N_k(I)} \quad [29.1]$$

as the probability that damage state DS is observed in k -type buildings when exposed to an earthquake of intensity I . In this equation $N_k(\text{DS}, I)$ stands for the number of k -type buildings in damage state DS, whereas $N_k(I)$ stands for the total number of k -type buildings under earthquake intensity I . The sum of the probabilities in each column of a DPM equals 1.0. The information contained in the damage probability matrix and in the damage ratios can be combined as the mean damage ratio, $\text{MDR}_k(I)$, which is expressed as follows:

$$\text{MDR}_k(I) = \sum_{\text{DS}} P_k(\text{DS}, I) \times \text{CDR}_{\text{DS}} \quad [29.2]$$

where CDR_{DS} = central damage ratio corresponding to the damage state, DS.

29.2.3 Pure risk premium

The expected annual damage ratio (EADR_k) is used as a measure of the magnitude of earthquake damage to a k -type of structure built in a certain seismic zone and is defined as:

$$\text{EADR}_k = \sum_I \text{MDR}_k(I) \times \text{SH}_I \quad [29.3]$$

where $\text{MDR}_k(I)$ = average damage ratio for the k -type of structures subjected to an earthquake of intensity I , and SH_I = annual probability of an earthquake of intensity I occurring at the site. EADR_k is a unitless quantity and can be interpreted as the pure insurance premium for a unit property replacement cost.

After calculating EADR_k , the pure risk premium (PRP_k) for the portfolio of contracts is computed based on the insured value of the building (INSV) under consideration from the following relationship:

$$\text{PRP}_k = \text{EADR}_k \times \text{INSV} \quad [29.4]$$

29.2.4 Commercially priced earthquake insurance premium

The commercially priced earthquake insurance premium (CPP_k) that will be charged by an insurance company for the k -type of structure is found by increasing the PRP_k by some margin as follows:

$$CPP_k = \frac{PRP_k}{1 - LF} \quad [29.5]$$

where LF = load factor which covers the hidden uncertainties, administration, business, and taxation expenses, and a reasonable profit allowance for the insurance firm. In previous studies for Turkey, the corresponding load factor is taken as 0.4 (Gurpinar and Yucemen, 1980; Yucemen, 2005) and with this value of LF, the commercially priced earthquake insurance premium that will be charged by the insurance companies becomes:

$$CPP_k = 1.67 \times PRP_k \quad [29.6]$$

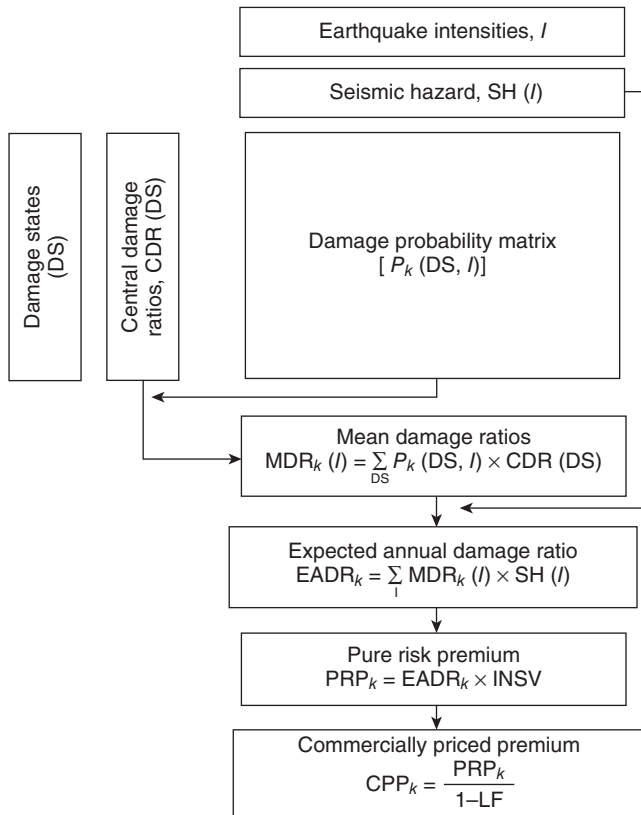
However, one should bear in mind that the insurance rate charged by a company is a function of its capital and the demand from the public. Other key factors are reinsurance rates which are generally controlled by the foreign reinsurance firms and market conditions. Another point to be noted is the fact that the earthquake risk is low-probability, high-consequence loss where a large number of dwellings are affected simultaneously by a common cause; thus the independence assumption of the law of large numbers is violated. Accordingly, the premium calculation based on the expected cost may not be the best way. However, in the case of an insurance pool, like TCIP, where the earthquake insurance is widely spread over the whole country with approximately 3.5 million insured dwellings, it is still possible to assume that the law of large numbers is valid. A flowchart of the algorithm for the computation of the earthquake insurance rates is shown in Fig. 29.1.

29.3 Application: assessment of earthquake insurance rates for different seismic zones in Turkey

Deniz and Yucemen (2007, 2009) have applied the model described above to compute the earthquake insurance premium rates for reinforced concrete buildings constructed in 21 pilot locations (cities) located in the five seismic zones of Turkey. The steps of the application of the model are summarized in the following subsections.

29.3.1 Probabilistic seismic hazard assessment

In the studies conducted by Deniz and Yucemen (2007, 2009), 21 city centers were selected for the determination of the insurance premium rates. The site group is established such that comparisons are possible with respect to cities in different seismic zones (according to the seismic zoning map accompanying the Turkish Seismic Code (Code, 1997), and with respect to cities in the same seismic zone. In conducting the PSHA, the earthquake



29.1 Flowchart for the computation of earthquake insurance premiums (adapted from Yucemen, 2005).

catalog compiled by Deniz (2006) for earthquakes occurred in Turkey was used. This catalog contains instrumental earthquake records between January 1, 1900 and July 30, 2004 where magnitudes of earthquakes given in different scales are converted to the moment magnitude scale, M_w , by using the conversion equations developed by Deniz and Yucemen (2010).

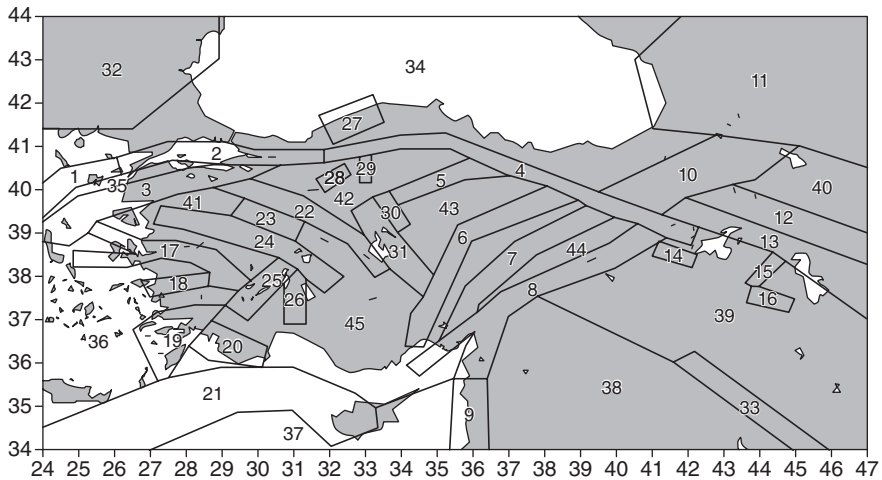
For the use of the Poisson model, which assumes independence between the occurrences of earthquakes, main shocks are differentiated from foreshocks and aftershocks, and a seismic database consisting of only main shocks is prepared as an alternative to the original seismic database consisting of all earthquakes. Seismic hazard analyses are performed by considering these two alternative seismic databases and the results are combined in the logic tree analysis (see Table 29.2). In this declustering process all earthquakes that fall inside a given space and time window around another larger magnitude event are classified as foreshocks or aftershocks (secondary events). Specifically, spatial aftershock zone sizes are determined as the

Table 29.2 Subjective probabilities assigned to alternative assumptions

Alternative assumptions	Subjective probability
All earthquakes	0.5
Main shocks only	0.5
Incomplete catalog	0.4
Artificially completed catalog	0.6
Standard least squares regression in the computation of the recurrence relationships	0.4
Maximum likelihood method in the computation of the recurrence relationships	0.6
Attenuation relationship of Musson (2000) in its original form	0.5
Attenuation relationship of Musson (2000) converted to M_w scale	0.5
Attenuation uncertainty is equal to $\sigma_1 = 0.08$	0.15
Attenuation uncertainty is equal to $\sigma_1 = 0.49$	0.60
Attenuation uncertainty is equal to $\sigma_1 = 0.80$	0.25

enveloping space windows of Gardner and Knopoff (1974), Savage and Rupp (2000), and Kagan (2002). For the time windows, on the other hand, average of the windows specified by Gardner and Knopoff (1974) and Savage and Rupp (2000) are utilized. Spatial and temporal aftershock window sizes are based on the magnitude of the main shock. Foreshock identification is achieved by using magnitude dependent spatio-temporal windows also.

Virtual seismotectonic source zones (i.e. geometrical configurations, such as polygons/lines), within which seismicity is homogenized, are delineated at regions of epicenter clustering, based on subjective judgment of experts. The configuration given by Bommer *et al.* (2002) is adopted with some local modifications. For the earthquakes that cannot be related to any of these seismogenic provinces, background seismicity regions are defined; all geographic areas/regions are covered by either well-defined faults, area/line seismotectonic source zones, or background source zones. The resulting seismic source zones and their geographical locations are shown in Fig. 29.2. The seismicity map of Turkey showing the epicentral locations of earthquakes with $M \geq 4.0$ since 1900 is given in Plate V (between pages 452 and 453). For the temporal distribution of earthquakes, the earthquake occurrence within a seismic source zone is assumed to follow the Poisson distribution, whereas the probability distribution of earthquake magnitude is described by the Gutenberg and Richter (1949) recurrence relationship. The source specific constants of the Gutenberg–Richter recurrence relationship are estimated both by carrying out least squares regression analysis



29.2 Location of seismic source zones defined for Turkey (Deniz, 2006).

and applying the maximum likelihood method to the observed data. The results are combined at the end.

Since smaller magnitude earthquakes are incomplete, the rates of small magnitude earthquakes are underestimated if the earthquake catalog is used as it is. For this purpose, Stepp's (1973) method is applied for each seismic source zone, and a complete regional earthquake catalog is developed, which can be used for PSHA. The seismicity parameters of each seismic source zone are obtained after completeness analyses, again by using both the standard least squares regression and the maximum likelihood methods.

The effect of a given earthquake with respect to the desired ground motion parameter at a certain distance is modeled by using the ground motion attenuation relationship of Musson (2000) as shown below:

$$I = 1.063 + 1.522 \times M_s - 1.102 \times \ln R - 0.0043 \times R \quad [29.7]$$

where I is the intensity, M_s is the surface wave magnitude (which can be converted to M_w scale with no systematic error through the inverse of the orthogonal conversion relationship that was developed between M_s and M_w by Deniz and Yucemen, 2010) and R is the hypocentral distance in km. The standard deviation of Eq. (29.7) is specified to be $\sigma_I = 0.49$. Three different uncertainty levels around the mean attenuation curve are assumed in the computations. The attenuation analysis of Musson (2000) is based on the earthquake catalogue of Ambraseys (1988), which is a very comprehensive 100-year catalogue of the shallow Turkish earthquakes. Since intensity is by definition related to damage, the attenuation relationship selected in terms

of an earthquake intensity scale is convenient while integrating the hazard and the earthquake damage which is known with respect to different intensity levels. Besides, it predicts the intensity as a function of the earthquake magnitude and connects a link between the earthquake catalogue in magnitude scales and the seismic hazard in intensity units. Since neither Musson (2000) nor Ambraseys (1988) made a discrimination among the earthquakes of the catalogue with respect to the soil conditions of the locations where damage occurred, our seismic hazard analysis is expected to yield results compatible with average local soil conditions.

Different seismic hazard combinations are formed under the consideration of either the whole earthquake catalogue or the main shocks only, either incomplete or complete catalogs, using either the standard least squares regression or the maximum likelihood method when obtaining the recurrence relationships, different forms of the selected attenuation relationship and different levels of attenuation uncertainty. Finally, the best estimate hazard curves are obtained to be the combination of $2^4 \times 3 = 48$ different cases. While combining the results of these cases, logic tree approach is utilized and subjective weights are assigned to each alternative case. These weights, which are shown in Table 29.2, represent the probability of each assumption being valid as compared to the alternative ones and are based mainly on the opinion of the author.

29.3.2 Estimation of earthquake damage through damage probability matrices

Most of the building stock in Turkey under earthquake hazard is low to medium-rise buildings. High-rise buildings are usually designed and constructed with special care and comply highly with superior earthquake-resistant design principles. For this reason, the focus is given to the low- and medium-rise buildings. However, a basic distinction is made depending on the construction material so that reinforced concrete and masonry buildings are treated in different categories as their earthquake responses are quite different. Here only reinforced concrete buildings are considered.

In the conventional DPM-based method, separate matrices have been formed for different seismic zones. The reason for this is that different levels of expected earthquake excitation are considered during the design and construction of buildings in different seismic zones. Gulpinar *et al.* (1978) separated the expected damage of structures depending on their compliance with earthquake-resistant design provisions and established two groups: as buildings constructed according to the Code (AC) and buildings constructed not in accordance with the requirements of the Code (NAC).

The available DPMs are complemented with empirical damage data for the ten recent earthquakes, namely, 1999 Marmara and 1999 Duzce, 2002

Table 29.3 Best estimate MDRs (in percent) for reinforced concrete buildings

Seismic zone	Intensity (EMS-98)				
	V	VI	VII	VIII	IX
Zone I	0.15	4.67	7.98	15.77	24.00
Zone II	0.11	2.19	8.97	14.31	24.06
Zone III	0.15	2.50	10.56	21.08	32.08
Zone IV	0.04	4.16	14.64	22.78	33.40

Source: Adapted from Deniz and Yucemen (2007).

Bolvadin–Cay–Sultandagi, 2003 Izmir–Urla–Seferihisar, 2003 Bingol, 2003 Malatya–Puturge–Doganyol, 2003 Denizli–Buldan, 2004 Elazig–Sivrice–Maden, 2005 Hakkari, and 2005 Cat–Karliova earthquakes. The new empirical data are based on the damage assessment reports of the above mentioned earthquakes obtained from the archives of the General Directorate of Disaster Affairs. Damage statistics of approximately 120 000 buildings are compiled and processed for this purpose (Deniz, 2006).

The empirical damage distribution data obtained from the previously compiled DPMs and the DPMs based on the study conducted by Deniz (2006) were not adequate to establish a full set of MDRs for all intensity levels. Accordingly, expert opinion DPMs given by Gurpinar *et al.* (1978) for reinforced concrete buildings are utilized to complement the empirical data. The resultant best estimate MDRs for reinforced concrete buildings are given in Table 29.3.

29.3.3 Insurance premium rates for reinforced concrete buildings located in different seismic zones of Turkey

The DPMs obtained in the previous section give the damage probability distributions explicitly for each discrete earthquake intensity level. Accordingly, the annual earthquake occurrence rates of each intensity level (intensity levels of V, VI, VII, VIII, and IX) are obtained by conducting PSHA as described in Section 29.2.1. The matrix multiplication of the hazard component and the damage component and the superposition of the results for a full range of seismic hazard levels yield the expected annual damage ratio ($EADR_k$) following Eq. (29.3). This quantity corresponds to the earthquake insurance premium for a unit property replacement cost and is obtained for reinforced concrete buildings located in different seismic zones of Turkey. Finally, the pure risk premium (PRP_k) of a property can be calculated proportional to the corresponding property's insured value. In the

calculation of the $EADR_k$, the contributions of the intensity level V are not taken into account since in the current earthquake insurance application in Turkey, damage up to 2% of the replacement cost of buildings is considered to be deductible and is not covered by the compulsory earthquake insurance scheme.

The best estimate pure and commercially priced earthquake insurance rates for reinforced concrete buildings are given in Table 29.4 for different seismic zones of Turkey. In the same table the currently charged earthquake insurance rates are also shown.

The pure insurance premium rates given in Table 29.4 consist of the contributions of the expected annual damage from different levels of earthquake hazard. Small intensity earthquakes produce less damage but occur more frequently, whereas large intensity earthquakes result in major damage although they occur rarely. The contributions of each intensity level to the pure insurance premium rates are computed and presented in Table 29.5. As observed in Table 29.5, the majority of the insurance premium rates are due to small to moderate intensity earthquakes. This observation indicates that the accurate determination of the occurrence probabilities of small and moderate magnitude earthquakes warrants special attention in the

Table 29.4 Best estimate and the currently charged earthquake insurance rates for reinforced concrete buildings

Seismic zone	Best estimate pure risk premium rates (‰)	Best estimate commercially priced earthquake insurance rates (‰)	Currently charged earthquake insurance rates (‰)
I	6.12	10.21	2.20
II	4.09	6.81	1.55
III	2.70	4.50	0.83
IV	1.99	3.32	0.55
V	0.93	1.55	0.44

Table 29.5 Ratios of the contribution of each intensity level to the pure insurance premium rates

Seismic zone	Intensity (EMS-98)			
	IX	VIII	VII	VI
I	0.08	0.17	0.28	0.47
II	0.10	0.18	0.41	0.31
III	0.09	0.22	0.40	0.29
IV	0.03	0.13	0.42	0.42
V	0	0.04	0.32	0.64

Source: Adapted from Deniz and Yucemen (2007).

Table 29.6 Variation of pure risk premium and commercially priced earthquake insurance rates for reinforced concrete buildings with respect to the degree of compliance with the seismic code

Seismic zone	Pure risk premium rate/commercially priced earthquake insurance rate (%)		
	AC	Best estimate	NAC
I	3.13/5.22	6.13/10.21	8.50/14.17
II	2.54/4.23	4.09/6.81	5.21/8.68
III	2.32/3.87	2.70/4.50	3.27/5.45
IV	1.79/2.99	1.99/3.32	2.68/4.47
V	0.82/1.36	0.93/1.55	1.55/2.58

Note: AC according to the code; NAC: not according to the code.

assessment of earthquake insurance rates. Nonetheless, seismic hazard studies usually focus on the occurrence rates of large magnitude earthquakes rather than those of the small and moderate magnitude earthquakes.

The DPMs based on expert opinion (Gurpinar *et al.*, 1978) give two sets of subjective damage probabilities for reinforced concrete frame buildings constructed in the different seismic zones of Turkey. The first set ‘AC’ corresponds to buildings that are designed and constructed in conformance with the specifications designated in the 1975 Seismic Code (1975) while in the second set ‘NAC’, it was assumed that the earthquake resistant design provisions are violated. The availability of this information facilitates the computation of the commercially priced earthquake insurance premium rates for AC and NAC class buildings. The results are displayed in Table 29.6. This aims at showing a set of earthquake insurance premium rates ranging from a minimum (corresponding to a building fully complying with seismic design provisions) to a maximum (failing to comply with seismic design provisions). The earthquake insurance premium rate of a building is expected to lie between these two bounds. The earthquake insurance premium rates based on expert opinion damage estimates showed that violation of the code requirements in the design and construction of buildings results in significantly higher insurance rates.

29.3.4 Evaluation of the computed earthquake insurance rates

The best estimates of earthquake insurance rates obtained herein show differences with respect to both what previous researchers have determined and what is being applied currently in the international earthquake insurance implementation. At the international level, the maximum earthquake

insurance rates are 5.25‰ for California, 4.30‰ for Japan and 7.27‰ for Mexico (Deniz, 2006). For other countries, the maximum earthquake insurance premium rates are significantly smaller. However, a typical deductible rate of 10–15% is applied in most of the countries. This rate is 2% in Turkey. If a deductible rate of 10% is applied in Turkey, the $EADR_k$ of intensity VI earthquakes (in all seismic zones) and the $EADR_k$ of intensity VII earthquakes in seismic zones I and II will no longer contribute to the insurance premium rates (see Table 29.5). The contribution of only the intensity VI earthquakes to the expected annual damage ratios is 47%, 31%, 29%, 42%, and 64% for seismic zones from I to V, respectively (see Table 29.5). By disregarding this contribution, the earthquake insurance rates will be decreased to the similar level of the ones implemented in other countries.

In the calculation of the commercially priced earthquake insurance rates, PRP_k s, are increased by a factor of 1.67 as has been done in the previous earthquake insurance studies in Turkey. However, it is emphasized that the surcharge (load factor) applied by an insurance company is a function of its capital as well as reinsurance rates which are generally controlled by the foreign reinsurance firms and market conditions. Reduction of this factor will also result in a decrease in the commercially priced earthquake insurance premium rates obtained in this study. As a matter of fact since TCIP is a nonprofit organization the actual increase in PRPs will be marginal and will be much less than 67%.

29.4 Implementation of earthquake insurance: Turkish Catastrophe Insurance Pool (TCIP)

Within the context of this chapter it is aimed to give an idea about the practical implementation of earthquake insurance. In this respect, here we present an overview of the compulsory earthquake insurance scheme and describe the organizational structure and operational details of TCIP. It is also aimed here to share the 12 years of experience with respect to the implementation of the obligatory earthquake insurance in Turkey, which may form a good reference for other countries that are planning to set up such an insurance scheme.

29.4.1 Background information

TCIP is a nonprofit public entity affiliated with the Ministry of State, to whom the Under-secretariat of Treasury reports. TCIP was established by the Decree Law 587 (Date: 27 December 1999) pertaining to the compulsory earthquake insurance (Akin, 2008). According to the decree law, earthquake insurance was made compulsory for all registered residential buildings within the municipality boundaries.

In Turkey, the State used to have a legal obligation (Disaster Law, No. 7269) to fund the costs of reconstructing buildings after an earthquake. This responsibility of the State brought an unplanned burden on the national economy and on the already limited central budget in the case of catastrophic seismic events. The two major earthquakes in Turkey on 17 August and 12 November 1999, which both occurred in or near urban settlements, caused widespread destruction of the building stock. Immediately after these two extremely destructive earthquakes the Government of Turkey has decided to enforce earthquake insurance on a nation-wide basis through the TCIP. The original aim of TCIP was to provide a multi-peril coverage, but currently it provides only compulsory earthquake insurance coverage.

The idea of a nation-wide obligatory earthquake insurance enforcement was first put forward in 1978 by the Ministry of Reconstruction and Settlement. Although the main aim was to reduce government's fiscal exposure, it was also intended to encourage risk mitigation and safer construction practices. To achieve these purposes, all registered residential dwellings that are located within municipality boundaries were required to purchase the compulsory earthquake insurance coverage. Initially funded by the World Bank, TCIP was founded in August 2000 and the TCIP program became effective since then. With its 4.0 million policies as of September 2012, TCIP has a potential to become the largest earthquake insurance underwriting company in the world.

29.4.2 Basic properties of the compulsory earthquake insurance scheme

In general terms, the compulsory earthquake insurance is an insurance product oriented towards dwellings within the boundaries of municipalities. The guarantee for compensation is provided by TCIP, while the marketing authority is given to 30 authorized insurance companies and their 22500 agencies. Only material damage to buildings is covered since the subject matter of insurance is limited to the building in the policy. Any damage to persons and contents is not covered. The period of circulation of the compulsory earthquake insurance is one year. Policies must be renewed prior to its expiry every year. Private dwellings located in villages, public buildings, buildings entirely used for commercial and industrial purposes, and buildings under construction are left outside the scope of the scheme.

The amount of annual premium shows variations in relation to the size of dwelling, type of construction (steel/ reinforced concrete, masonry and others), and the seismic zone (five zones) and ranges from 0.44% to 5.5%. For a dwelling of 100 m² the maximum annual premium may reach up to US\$ 110 depending on the type of construction and the seismic zone, whereas the minimum value is set to US \$25 (www.dask.gov.tr, 2012). A

Table 29.7 Statistical information on the implementation of the compulsory earthquake insurance as of 1 September 2012

Total number of policies	4 000 000
Take-up-rate	26%
Total coverage	US\$ 155 billion
Average coverage	US\$ 38 500
Average premium	US\$ 62
Total number of claim files	19 485
Total claims paid (since 2000)	US\$ 78 million

Source: Gungor (2012).

deductible of 2% is imposed in order to avoid minor claims and reduce administrative costs. The insurance premiums increase every year according to the increase in the construction costs, but they are kept at affordable levels. If the dwelling sum insured exceeds the TCIP limit of approximately US\$ 85 000, then the insured can ask for a homeowner's policy through private insurance companies in excess of the current TCIP policy. The land value is not included under the 'sum insured'.

It is noted that since the State does not provide any funds, TCIP has to pay claims through its own funds. On the other hand, TCIP's funds cannot be used for any purpose other than insurance, either by any other authority or by TCIP itself. The collected premiums are deposited in bank accounts in order to get yield. The guidelines for depositing the funds of the pool give priority to safety and liquidity. TCIP buys reinsurance with the risk charge based primarily on the probable maximum loss (PML). Protection limits are set based on detailed study of various earthquake scenarios.

The claims are generally paid within a month. Statistical data showing some of the details of the compulsory earthquake insurance as of 1 September 2012 are given in Table 29.7. The overall market penetration is about 26% if the whole building stock is considered. However, if only the eligible building stock is considered (i.e. those buildings that are registered at the cadastral records) a more realistic rate will be obtained as 35–40%. The penetration for different geographical regions closely follows the income level of the people living in these regions.

29.4.3 Objectives

The primary objectives foreseen in the establishment of TCIP can be summarized as follows:

- To provide insurance coverage for all of the dwellings within the scope of the TCIP policies against earthquake risk at affordable rates.

- To ensure risk sharing within the country and also to distribute the financial liabilities caused by earthquakes on to the international reinsurance markets.
- To mitigate the possible financial burdens on the government due to earthquakes (especially construction of new dwellings after a catastrophe).
- To utilize insurance as a means to encourage the construction of earthquake-resistant structures.
- To ensure the accumulation of long-term reserves to finance future earthquake losses.
- To contribute to the development of insurance consciousness and risk awareness in the public.

29.4.4 Organizational structure

TCIP can be considered as a public–private partnership. It is supervised by the Under-secretariat of the Treasury, which forms the public side of the partnership. The Under-secretariat develops and maintains a regulatory framework and is responsible for overseeing the program, auditing of all operations and accounts of TCIP. The other partner (private sector), the so-called pool management company, performs the administrative duties. The activities of the pool manager consist of coordinating the sale processes, settling of accounts, getting the necessary reinsurance protection, settling of claims regardless of which insurance company or agent issued the policy and indemnifying the insured, maintaining the non-stop service of the IT system, performing public relations, and managing funds of the pool (Yucemen, 2011). This operational management duty has been contracted out to leading reinsurance companies in Turkey for periods of 5 years. By engaging the private sector in the process of administration, the bureaucratic burdens and inefficiency are minimized.

The overall management of TCIP is performed by the TCIP Board of Directors. The main duties of the TCIP Board include: setting principles of all operational work and deciding for expenditures including reinsurance, and marketing and public relations expenses. The Board is composed of the following seven members: Representative of the Prime Ministry, representative of the Under-secretariat of the Treasury, representative of the Ministry of Public Works and Settlement, representative of the Capital Markets Board, representative of the Association of the Insurance and Reinsurance Companies of Turkey, operational manager, and an earthquake scientist.

29.4.5 Major problems

Although the current market penetration of 26% is much higher than those attained in other countries and the 5% penetration rate in Turkey prior to

the establishment of TCIP, it is still not as high as desired and needs to be improved substantially. The main factors which cause the penetration to remain below the desired levels are identified as follows (Yucemen, 2011):

- A fatalistic mentality in society and the habit of expecting the State to provide everything.
- A low level of income and the wrong impression that the earthquake insurance premium is expensive.
- The cultural awareness of insurance has not reached to a sufficient level in Turkey and obtaining insurance is not a high priority in daily lives of people. Parallel to this, awareness of the public about TCIP and compulsory earthquake insurance is not very high.
- Lack of trust in TCIP with respect to the compensation of losses after an earthquake.
- There are no legal penalties or fines for the enforcement of the ‘compulsory’ earthquake insurance.
- Governments follow a populist trend by displaying their willingness to provide post-disaster aid to homeowners by themselves. This destroys the homeowners’ incentive to participate in the program. Instead, the State may subsidize the earthquake insurance premium for the low-income section of the population, which will be a rational policy.

29.4.6 Future actions

TCIP has proved to be a well-designed public–private partnership due to its satisfactory performance over the past 12 years. A number of initiatives are still being undertaken to improve the system in various aspects. The major initiative is to develop strategies to increase the take-up rate. For this purpose, new public relations strategies and advertising campaigns are developed in order to enhance the risk awareness of the public and overcome the adverse effects which hinder the increase in penetration and renewal rates.

A number of mechanisms are attempted to enforce the earthquake insurance. For several years homeowners needed to present their insurance policy to the Land Registry Office whenever they have to carry out an operation concerning the building. A similar plan of obliging homeowners to present their insurance policy when opening an account for services, such as water, natural gas, electricity, and telecommunications was recently put into practice for creating additional checkpoints. This type of checkpoint also improves the risk awareness of individuals.

Besides these initiatives, two major policies are under consideration for future realization:

- Allocation of a modest portion of the annual income of the pool to mitigation investments. Examples of these are: risk avoidance and

reduction projects, revisions in land use planning, and retrofitting of public and private buildings.

- The use of earthquake insurance as a tool for the enforcement of earthquake resistant provisions of the Seismic Code. As observed from Table 29.6, significantly higher insurance premium rates were calculated for 'NAC' buildings in comparison with 'AC' buildings. Thus the compliance with the Code should be an important factor in determining the earthquake insurance rates. In other words, risk-based pricing mechanism yielding to different rates should be implemented. It is also believed that enforcement of such a criterion, will not only encourage the implementation of the code requirements with respect to the earthquake-resistant design provisions, but will create a control mechanism. For this purpose insurance premiums should be differentiated by taking into consideration the 'seismic resistance quality' of the buildings. In other words, lower premiums should be charged for buildings that comply with the requirements of the seismic code.

29.5 Conclusions and future trends

In this chapter, a simple probabilistic model was presented for the calculation of earthquake insurance rates and its application was illustrated by computing the earthquake insurance rates for reinforced concrete buildings located in different seismic zones of Turkey. The practical implementation of the earthquake insurance is also discussed within the scope of compulsory earthquake insurance scheme and the Turkish Catastrophe Insurance Pool. The main conclusions and future trends are grouped with respect to the probabilistic model for the assessment of earthquake insurance rates and compulsory earthquake insurance implementation via TCIP, and are stated separately in the following.

29.5.1 The probabilistic assessment of earthquake insurance rates

The following conclusions are stated with respect to the proposed stochastic model of earthquake insurance assessment and the insurance rates obtained from this model. Also a set of recommendations are listed for the improvement of the current implementation of the compulsory earthquake insurance scheme and for its possible adaptation by other countries.

- Assessment of the earthquake insurance premium rates requires the integration of information on future earthquake hazard and seismic vulnerability of buildings. Consequently, the earthquake insurance rates are sensitive to the assumptions on seismic hazard analysis and

damage probability matrices. In this respect it is important that future insurance risk assessment models should be developed based on an interaction among actuarial science, earthquake engineering, and earth sciences.

- Estimation of earthquake damage depends mostly on the observed damage statistics compiled from past earthquakes. However, the observed damage data is generally not sufficient and should be supplemented with expert opinion or analytical methods.
- The commercially priced earthquake insurance rates estimated based on the proposed probabilistic model (per policy basis) are significantly greater than what are currently charged by TCIP. This is partly attributed to the fact that, since TCIP, the official regulator of the tariff, is a nonprofit organization, the load factor given in Eq. (29.5) can be assumed as zero, making the commercially priced rates equal to the pure risk premium rates, decreasing the difference from the currently charged insurance rates. The remaining difference can partly be explained by the law of large numbers, which can be stated within this context: as the number of policies increases the uncertainty in the expected loss of an insured building stock decreases reducing the insurance rates requested by the insurer.
- The current deductible rate of 2% in the Turkish earthquake insurance implementation is much less than what is currently charged in other countries. At the international level, the deductible rate is in the order of 10–15%. Considering the fact that small to moderate magnitude earthquakes occur frequently but result in minor damage, the percentage of the deductible might be increased in the tariff specified by TCIP so that the insurance rates decrease.
- A major portion of the earthquake insurance premium rates results from the expected annual damage contribution of small and moderate magnitude earthquakes. Therefore special attention should be paid to the accurate assessment of the seismic hazard resulting from small and moderate magnitude earthquakes.
- In calculating the commercially priced earthquake insurance rates, the value of the load factor should be based on a detailed analysis taking into consideration the local insurance and the international reinsurance market conditions.
- Significantly higher (two to three times more) insurance premium rates that result from the violation of the code requirements strongly suggest that compliance with the seismic code should be an important factor when determining the earthquake insurance rates. In other words, significantly different rates should be charged for buildings depending on their degree of compliance with the code. It is also believed that enforcement of such a criterion will not only encourage the implementation of

the code requirements with respect to earthquake-resistant design provisions, but also create a control mechanism.

- The current compulsory earthquake insurance tariff charges the buildings in the same building class equally, irrespective of the individual features of each building. Features that affect the earthquake response of buildings in a negative way, such as soft storey, plan irregularity, weak storey, age, usage, etc., as well as the features of the soil on which buildings are located, are not considered within the current earthquake insurance implementation. The presence of these negative aspects should be taken into consideration and a certain surcharge should be added in the calculation of the earthquake insurance premiums.
- Seismic vulnerability of structures is presented through fragility curves instead of damage probability matrices in most of the recent studies. Multiplication of the elements of hazard curves by the elements of fragility curves would be a more preferable procedure in the computation of expected annual damage estimates, because in the fragility methods uncertainties can be directly incorporated into the computations. Accordingly, it will be more appropriate to utilize fragility curves in future studies concerning the quantification of earthquake insurance rates.
- Although earthquake insurance can be used as a tool for the improvement of construction practice and the enforcement of the seismic codes, its main aim is to provide a post-event response for the economical management of large-scale catastrophic earthquakes. Mitigation before such disasters and following an active prevention scheme certainly will reduce the number of damaged buildings and life losses. This issue has been addressed in a comprehensive way by the OECD as a response to the dramatic rise in the frequency and severity of catastrophes in recent years. In the year 2006, OECD has established an International Network on the Financial Management of Large Scale Catastrophes with the sole purpose of dealing with the financial management of large-scale catastrophes, with particular emphasis on earthquakes, the role of risk mitigation and insurance in reducing the impact of natural disasters,

29.5.2 The implementation of the compulsory earthquake insurance scheme and TCIP

Based on the past 12 years of experience the following comments can be made with respect to TCIP:

- TCIP has proved to be a well-designed public–private partnership due to its satisfactory performance over the past 12 years. By engaging the

private sector in the process of administration, bureaucratic burdens are minimized.

- A very large pool is created independent of the national budget. Therefore to a large extent it is free from political intentions; this is the major benefit of the system.
- TCIP has all main components and the potential for substituting for the government, in compensating post-disaster losses in residential buildings through an insurance program.
- Within the context of Turkish experience, it is observed that pooling natural catastrophe risks increases the take-up-rates significantly if it is made mandatory (with affordable rates) and improves risk awareness of the public. The current take-up rate of 26% is quite high compared with some other countries like India, Philippines, Iran, China, Romania, and Bulgaria, where the corresponding rates are all less than 5%. Furthermore, pooling natural catastrophe risks enforces standard wordings, conditions, and tariffs, and establishes a single risk transfer solution for a whole region or a country, yielding usually low administrative costs.
- In terms of program characteristics, TCIP can form a role model for countries with similar exposures and economic conditions for the mitigation of financial burdens on local authorities as well as stimulating risk awareness of the public. However, alternative models can be considered for other countries with respect to spatial extent (one or many countries) and organizational structure (public–private partnership or a purely private organization).

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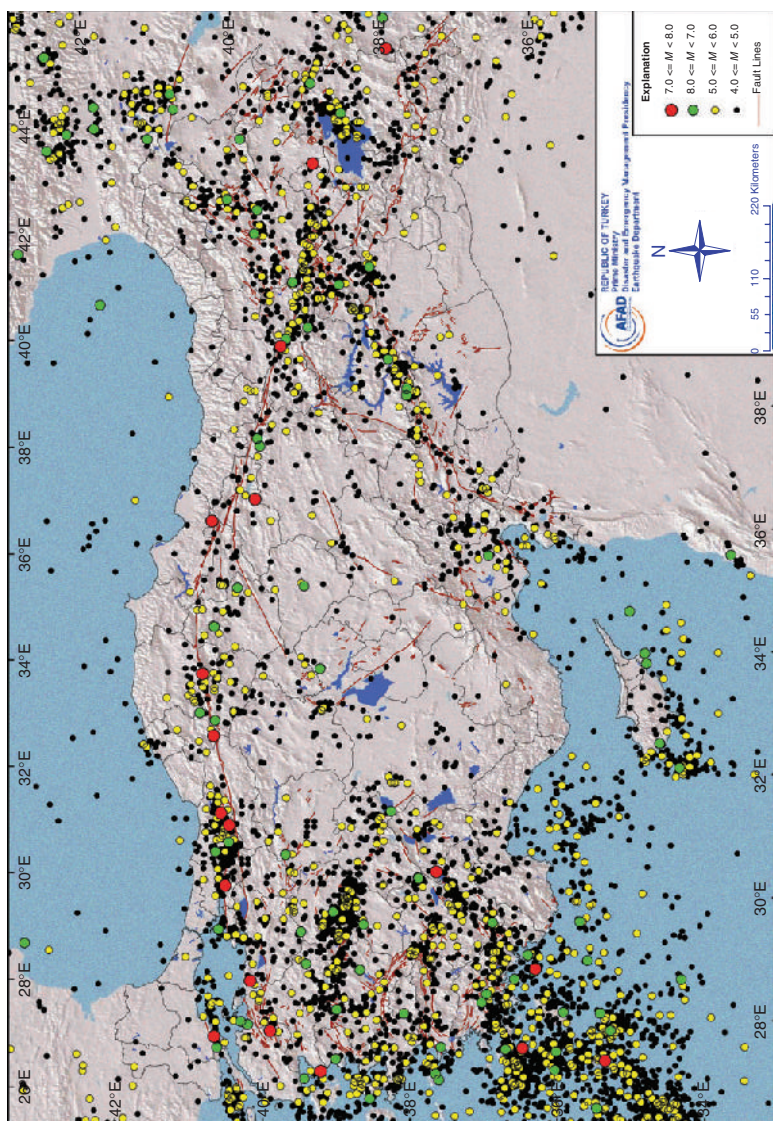


Plate V The seismicity map of Turkey showing the epicentral locations of earthquakes with $M \geq 4.0$ since 1900 (Kilic, 2012).

Assessing global earthquake risks: the Global Earthquake Model (GEM) initiative

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Abstract: The Global Earthquake Model (GEM) initiative aims to develop a global model of earthquake risk as an open source, community-driven project. In order to begin this in a structured way, a number of global components that cover the scientific modules of the model have been defined, and requests for proposals have been released, inviting international consortia to bid and lead these projects. The consortia leading these global components define standards and best practice related to the methodologies used in seismic hazard and risk assessment and in particular the collection and storage of data needed therein. This chapter introduces these global components and describes an open source software and risk assessment platform, OpenQuake, that is being developed to provide access to and community interaction with the proposed data and tools.

Key words: global, seismic hazard, seismic risk, vulnerability, exposure, community, open source.

30.1 Introduction

Almost half a million people have died in the last decade due to earthquakes. Most of these casualties were in the developing world, where risk is increasing due to rapid population growth, urbanisation and poor construction practice. In particular, many of the world's megacities of 10 million inhabitants and more, such as Delhi, Bogota, Jakarta and Lima, are situated in highly seismic active areas, as clearly emphasised in Fig. 30.1. A significant proportion of the world's population is therefore at risk from earthquakes. The recent earthquakes in Haiti, Chile, Japan and New Zealand have painfully reminded the world of the destructive impact of seismic events and the importance of reliable earthquake risk information. However, in many earthquake-prone regions, no risk models exist, and even where models do exist, they are often inaccessible due to their proprietary nature.

State-of-the-art information on earthquake risk covering all areas of the world is a necessary first step towards risk awareness and the undertaking of mitigating action. Such information is therefore a critical puzzle-piece for minimising loss of life, property damage, and social and economic

- Many current efforts use methodologies and input data that vary widely in format and quality, making it difficult to compare results across national boundaries.
- Earthquake risk estimation is a discipline where geoscience meets engineering and social science. Often efforts are focused on only one of these fields, or at maximum the combination of geoscience and engineering. Inclusion of modelling and assessment of social and economic impacts of earthquakes (on a global scale) has rarely happened until now.
- Current initiatives do not always make clear what the primary, underlying causes of risk are, whether it comes from the severity or frequency of ground shaking, the vulnerability of the building, or lack of recovery capability. Focused, effective mitigation requires in-depth knowledge of the underlying causes of risk.
- Most of the existing software and methodologies are largely developed by and for users in developed economies. Advanced (scientifically grounded) models or software rarely address risks in developing countries. Furthermore there is little transfer of technology or knowledge exchange, in order for local experts to build on their knowledge of the local circumstances and data.
- The last global scientific programme on seismic hazard was carried out in 1999 (GSHAP; Giardini *et al.*, 1999). The programme resulted in a static map of global hazards. Maps were produced using – primarily – seismicity catalogues and regional seismotectonic information. The map is a static representation of the hazard knowledge at that time.
- For what concerns seismic risk estimation, reliable information of existing infrastructure (buildings, lifelines, etc.) is indispensable, and a single database on exposed infrastructure is not currently openly available.
- Strategic decision making by governments, international bodies and (large) companies requires transparent, uniform and global information to be accessible in a single platform. At this time there is no such platform for global earthquake risk assessment.
- Most global earthquake risk assessments carried out in recent years (e.g. Dilley *et al.*, 2005; ISDR, 2009) are based on empirical data, for both the hazard and the vulnerability. Whilst an empirical approach to the definition of vulnerability has many advantages related to the use of real observed loss data that is correlated to recent past events of different levels of intensity, the same cannot be said of seismic hazard. In some cases, data from a limited number of years (20–40 years) has been used to define the seismic hazard; such an observation length is too short for low-frequency events such as earthquakes. There will be countries which have not experienced a large disaster in this period but which may nevertheless have a high probability of occurrence of a large magnitude

event; these countries will not be identified by many of the aforementioned risk assessments.

- Many global earthquake risk assessments lack sufficient consideration of the built environment in the vulnerability assessment.

In January 2009 the Global Earthquake Model (GEM) initiative launched a pilot project, GEM1, which had the objective of developing the initial IT infrastructure of GEM. As part of this effort, a number of existing hazard and risk software applications were reviewed (Danciu *et al.*, 2010; Crowley *et al.*, 2010), and the following characteristics were identified as those which should feature within GEM's software:

- allow users to upload their own hazard, vulnerability and exposure models (and thus not be tied to any specific region in the world);
- combine hazard and risk calculations within a single software, but also allow users to run hazard-only and risk-only calculations;
- be open-source software;
- be platform independent;
- have a user-friendly and intuitive user interface;
- make use of logic trees to consider the epistemic uncertainty;
- have adequate technical support and documentation;
- allow many different types of assets to be modelled (e.g. buildings, population and eventually infrastructure);
- incorporate the modelling of spatial correlation of ground motion residuals and correlation of uncertainty in building vulnerability;
- be modular and thus allow some components to be used in risk assessment for other hazards;
- be scalable, with parallelised calculators;
- produce a full spectrum of products such as stochastic event sets, ground motion fields, uniform hazard spectra, hazard curves and maps, disaggregation plots, damage and loss curves and maps.

In response to the needs outlined above and the shortcomings highlighted by the benchmark study and existing software comparisons, the GEM initiative has been launched to promote independent, uniform standards and tools to calculate and communicate earthquake risk worldwide.

30.1.1 Global earthquake risk modelling

GEM aims to build state-of-the-art, widely accepted basic datasets, models and software/tools for the assessment of seismic risk on a global scale and develop a supporting IT infrastructure by the end of 2013. Users across the globe will be able to access the basic datasets, models and accompanying tools through GEM's risk assessment platform (OpenQuake), allowing them not only to perform hazard and risk analyses, but also to make deci-

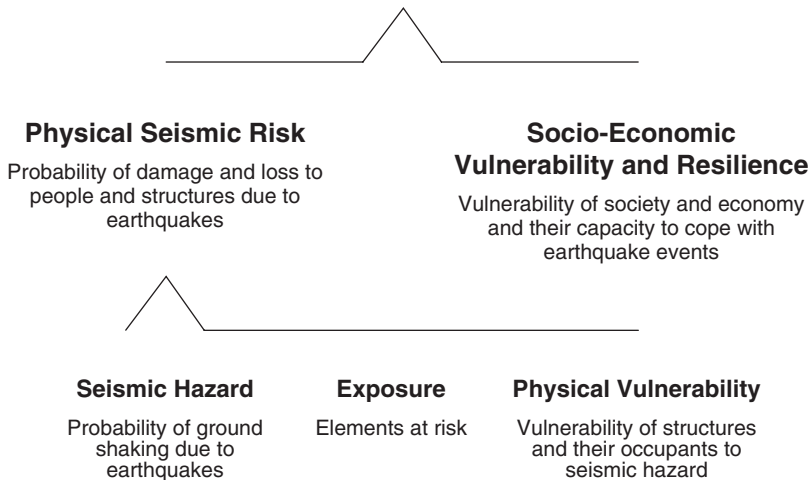
sions on retrofitting and insurance and collaborate and exchange data, results and opinions amongst each other. It is envisaged that the datasets and models will, however, be continuously evolving after 2013, and hence OpenQuake will be built in such a way that it can capture the world's best understanding of data, the earth and earthquake behaviour, and the built environment at any given moment in the future. In the years after 2013, GEM's growing user-community will be able to add data, exchange knowledge on methods for data analysis and other issues, and will in that way contribute to the continuous improvement of the risk assessment at a global scale.

The ultimate aim of GEM is to create a global earthquake risk model that is as homogeneous as possible, based on hazard and risk models that explicitly account for epistemic uncertainties through logic-trees, and that relies on highly reproducible procedures. Within GEM's first working programme (2009–2013) it will not be possible to incorporate all the data effectively available, nor cover all the countries in the world in a uniform way. Also, it is unlikely that secondary hazards such as tsunamis and landslides will be covered within the first set of models. During this first Working Programme GEM will hence create tools and standardised methods for obtaining and analysing data, and GEM collaborators/affiliated researchers will use these tools to begin the process of assembling the needed (global) datasets, hereby producing data coverage for the world that is more uniform and complete than before.

30.1.2 Scientific modules

GEM's global earthquake risk model is based on a probabilistic framework (see e.g. McGuire, 2004) and consists of seismic hazard (represented by the exceedance probability of levels of ground shaking resulting from earthquakes, within a given time span), vulnerability (defined as the probability of loss given a level of ground shaking for physical vulnerability, or through indicators that envelop the socio-economic factors known to be driving forces of vulnerability), and exposure (the elements at risk). Combining these components within the risk module allows for an estimation of direct damage and loss as well as risk indicators or indices. Indirect economic loss and the long-term impacts of earthquakes fit within the risk and impact analysis module and then decision-making tools can be built on top of the aforementioned output, as illustrated in Fig. 30.2. The development focus within GEM is on the components of hazard, physical vulnerability and exposure, calculation of direct losses (total losses and insured losses), complemented by derivation of indexes for social vulnerability and disaster resilience for calculation of total risk, and decision support tools that consider the cost–benefit analysis of retrofitting and insurance.

Integrated Seismic Risk



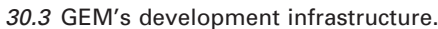
30.2 Scientific modules of GEM.

30.1.3 Collaborative development

GEM believes that global risk assessment can only be carried out in cooperation with the community and be readily adopted because the community believes in it. Experts and professionals worldwide working on issues related to seismic hazard assessment, risk assessment and risk management form the core of GEM's 'community'.

Development of the datasets, models and tools is for a great part carried out by the community, both on global and regional scales. Hundreds of institutions and professionals in the relevant disciplines carry out research and develop methods, tools and discuss standards. Developments at the forefront of scientific and engineering knowledge as well as IT processes and infrastructure are being integrated into the model, also thanks to the bottom-up and collaborative approach of development.

The GEM initiative is advancing by means of various 'building blocks'. The first building block was the GEM1 pilot project, which ended in March 2010 and delivered a proof-of-concept in terms of preliminary global hazard and risk calculations and an initial model-building infrastructure. Currently, development within GEM is based upon the core efforts of the Global Components, Regional Programmes and the Model Facility (see Fig. 30.3).



- *Global Component* projects are carried out by international consortia that are selected after a thorough process of expert elicitation, community feedback, and peer review. Experts and stakeholders working in GEM-related fields can provide feedback on the work carried out, through GEM Nexus, GEM's collaboration website.
- *Regional Programmes* provide feedback on development of the Global Components, work on data collection and create regional input models for GEM.
- The *Model Facility* oversees development of the OpenQuake risk assessment platform and the OpenQuake Engine that powers hazard and risk

calculations at any scale. This software is developed in an open environment, allowing a wider community of developers and expert-users to jointly enhance the software. Models, databases, software and toolkits for developing models and making decisions on retrofitting and insurance will be the main ingredients of OpenQuake. A dedicated Testing and Evaluation facility will test datasets and model components before they are released to the public for the first time.

In order to support collaborative development of uniform and open databases, standards, (regional) models and tools, a platform has been developed for GEM collaborators, experts and practitioners in earthquake risk assessment to collaborate, share, discuss and suggest. GEM Nexus ('a means of connection') is home to the various groups working on GEM, who have access to a set of collaboration instruments (<http://www.nexus.globalquakemodel.org/>). Nexus is also open to the wider scientific community who can follow the groups, provide feedback on the work being carried out and connect with others that are making a difference in earthquake risk assessment.

From 2014 onwards, the OpenQuake platform will be available that will allow researchers, agencies and institutions to adopt and enhance the tools, provide comments and populate the datasets, so as to continuously update and improve global risk assessment. Such a process of 'crowd-sourcing' is obviously closely monitored and data quality-controlled to ensure the highest quality of data and compatibility.

It is hoped and envisaged that scientists, practitioners, IT experts and others will actually make use of the critical research gaps that we highlight, to collaborate with us and collectively work on making the datasets, models and tools of better use for all stakeholders worldwide.

30.2 Current status of Global Earthquake Model (GEM)¹

GEM is currently funding five hazard global components projects planned to prepare global datasets with basic information necessary to create probabilistic seismic hazard analysis (PSHA) input models. These five projects – which all started in 2010 – are: the Global Earthquake History, the Global Instrumental Catalogue, the Global Active Fault and Seismic Source Database, the Global Geodetic Strain Rate Model and the Global Ground Motion Prediction Equations.

Requests for Proposals for five Global Components related to Physical Risk were drafted by a focus group and subsequently modified based on

¹This section was drafted in April 2012.

the feedback from the community through the online commenting system on the GEM website. They are: GEM Ontology and Taxonomy, Global Exposure Database, Global Vulnerability Estimation Methods, Global Earthquake Consequences Database, Inventory Data Capture Tools; all these Global Components are now operative. It is noted that during this first 5-year phase of GEM, the emphasis will be placed on the seismic risk of buildings (residential and commercial) due to budget and time constraints, whilst future extensions are expected to consider infrastructure/lifelines.

Following a discussion and iteration process of more than a year related to measurement and incorporation of socio-economic impact of earthquake events, a project on social vulnerability and disaster resilience was initiated at the start of 2012. The project directly supports GEM's mission of producing a holistic global earthquake risk model that integrates metrics of social vulnerability and disaster resilience with the ongoing activities of hazard and physical risk.

The ten Global Components as well as the project on social vulnerability and resilience are described in more detail below, followed by a description of the project on social vulnerability and disaster resilience.

30.2.1 Global Earthquake History

The Global Earthquake History is compiling an archive of historical earthquakes studies and a global parametric catalogue called Global Large Historical Earthquake Catalogue (GLHECAT) containing events with magnitude $M \geq 7.0$ that occurred in the temporal window 1000 and 1903AD (<http://www.globalquakemodel.org/hazard-global-components/earthquake-history>).

The catalogue is compiled by critically merging catalogues (e.g. Utsu, 2002, or Dunbar, 2009) or datasets taken from published and publically available material. The online archive of historical earthquake studies containing, for each event, reports and macroseismic data points (when available) accompanies the information in the catalogue. The catalogue and the archive of historical earthquake studies will build upon AHEAD, an archive of earthquake data for Europe prepared in the context of the NERIES project (see <http://www.emidius.eu/AHEAD/main/>).

This is a first global collection providing for each event in the catalogue a comprehensive compilation of the different most recent studies performed as well as the original information studies referenced by the catalogue's authors to infer the main parameters describing the event. This makes the present catalogue online an archive, a comprehensive collection of historical earthquakes and to the greatest extent a dynamic database updatable and improvable in the future. Moreover, this catalogue and the

related archive represent an important step in the creation of a global homogenised earthquake history and already proves to be an extremely important information source for PSHA. This two-year project is being coordinated by two leading institutions: INGV (Italy) and BGS (UK).

30.2.2 Global Instrumental Catalogue

The International Seismological Centre (based in the UK) in collaboration with a team of international experts is leading this initiative aimed at the creation of a homogeneous instrumental seismicity catalogue for the period 1900–2009 (<http://www.globalquakemodel.org/hazard-global-components/instrumental-catalogue>).

This global catalogue of earthquakes will contain about 25,000 events. In the interval 1900–1917 the catalogue will comprise events with magnitude greater or equal to 7.5; from 1918 until 1959 it will have events with magnitude greater or equal to 6.25 and, finally, from 1960 on the catalogue will include earthquakes with magnitude equal or greater than 5.5.

This catalogue will be composed of earthquakes with homogeneous locations and magnitude estimates, determined using the same tools and techniques to the extent possible. The magnitude determination and location procedures applied represent a synthesis of state-of-the-art methods. Each event will have a M_w value (with related uncertainty) based on seismic moment where possible (mainly for the events occurred in the window 1976–2009); in the other cases, empirical relations between M_w and M_s/m_b will be used to obtain proxy values of moment magnitude.

Most of the events in the pre-1971 period will have their M_s magnitudes computed in a systematic manner from the surface wave amplitude measurements recovered from the historical paper-based seismic station bulletins. These M_s magnitude were never available before; they are the results of a massive data entry effort completed under this project. Some IASPEI observers also provide feedback to the activities of this global component.

30.2.3 Global Active Fault and Seismic Source Database

The GEM Faulted Earth project, coordinated by GNS Science (New Zealand) and Earth Observatory Singapore is creating a database of active faults and faults sources following a common set of strategies, standards and formats (<http://www.globalquakemodel.org/hazard-global-components/active-faults>). This consortium has already created an archive of existing national or international databases to better comprehend the state-of-the-art and produced a database schema able to gather the information currently available in different formats. The final GEM Faulted



30.4 Faulted Earth tool for capturing data on faults collected by geologists. Note that the lineaments represented are just for demonstrative purposes. Map CCBYSA 2012 OpenStreetMap.org contributors (<http://www.openstreetmap.org>).

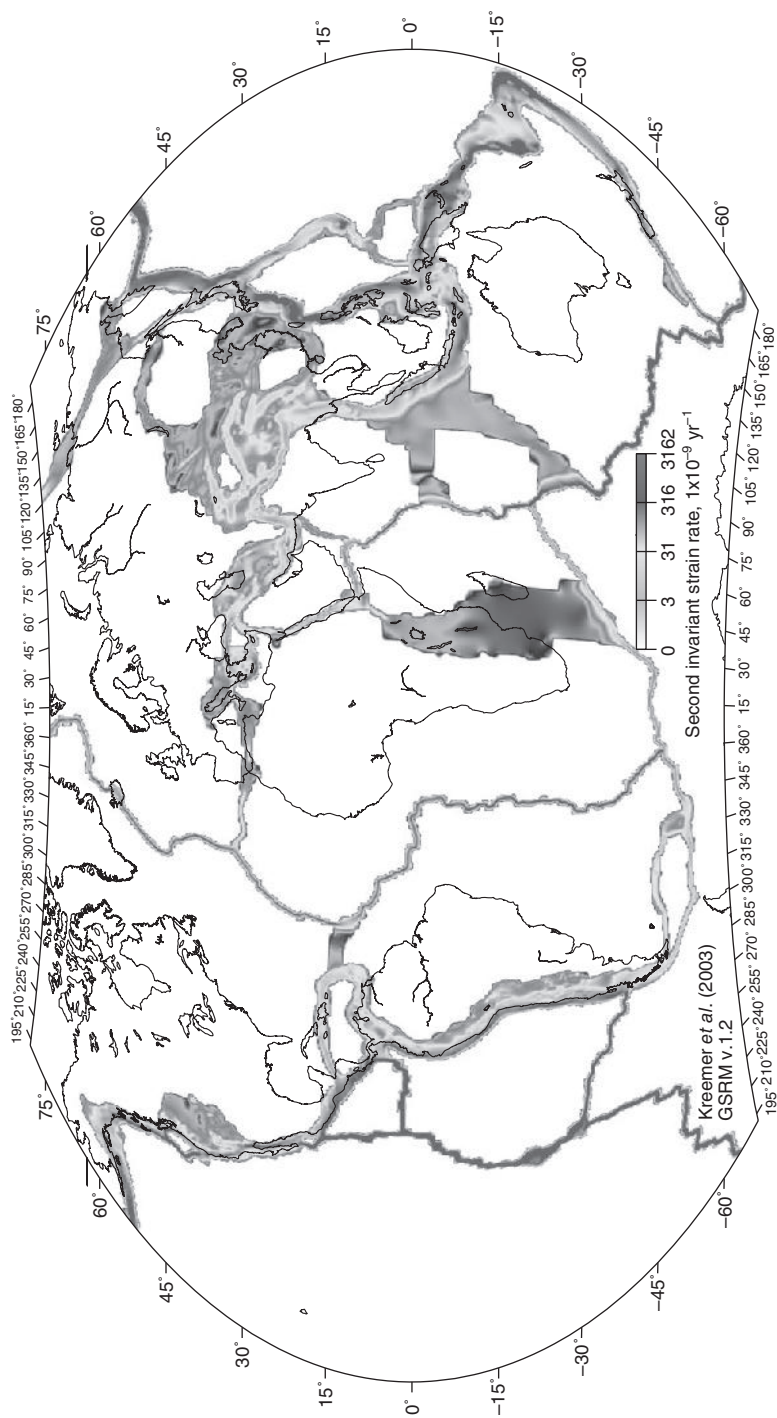
Earth database will contain information relative to active shallow faults and folds and subduction sources (using for example the SLAB 1.0 database – <http://earthquake.usgs.gov/research/data/slab/>). Further on, the Faulted Earth database will be considerably extended with contributions coming from regional groups; the compilation of the database with new information will be assisted by an ad-hoc web interface that is currently being created by the GEM Model Facility as a component of OpenQuake (see Fig. 30.4). Data collected in this Global Component will be a fundamental input to the Openquake Modeller, a suite of tools that GEM is creating to assist the hazard modeller in the creation of PSHA state-of-practice input models.

30.2.4 Global Geodetic Strain Rate Model

Strain rate is an important and independent source of information that can be used as a proxy for earthquake potential (<http://www.globalquakemodel.org/hazard-global-components/strain-rate>). The GEM Global Geodetic Strain Rate Model Global Component is taking advantage of the experience gained in the global strain rate map project (Kreemer *et al.*, 2003; see also Fig. 30.5) to create a comprehensive and uniform model for geodetic strain rates with significant updates with respect to the models that have been computed before.

The project has three main objectives: (1) create a uniform global GPS velocity database, (2) update, expand and improve the global strain rate map, and (3) disseminate the results.

The final products will be a numerical model of strain rate obtained from over 10000 GPS measurements of plate motion, covering the most important deforming areas of the globe. One version of the model will be solely based on geodetic measures, while another version will have additional



30.5 Grid representing the second invariant of the strain rate tensor computed by the Global Stain Rate Map Project (<http://gsrcm.unavco.org>).

constraints on the strain rate tensor field from earthquake focal mechanisms, seismicity distribution and active fault traces. The model will be based on secular horizontal velocities obtained from a dataset of some 4000 continuous GPS stations unevenly distributed on the globe, with the addition of thousands of published campaign-style velocity observations. The resolution of the models will be $\sim 0.2^\circ$ instead of 0.5° as in the model currently available. This new model will be the most updated version of the homogeneous global strain rate model made available by the scientific community.

Principal Investigators from the University of Nevada (USA), Ecole Normale Supérieure (France), University of California/CEA (USA) and UNAVCO are overseeing the tasks.

30.2.5 Global Ground Motion Prediction Equations (GMPEs)

The main goal of this project is the development of a harmonized and tested suite of ground motion prediction equations (GMPEs) that can be used at both the global and regional level for hazard and risk assessment, built on the most recent advancements in the field (<http://www.globalquakemodel.org/hazard-global-components/ground-motion>). The consortium is defining a uniform strategy for modelling ground motion by analysing the consistency in model parameters and site parameters. A second important task is to create, select and critically review a large set of GMPEs; this set is later used to determine a global set of recommended GMPEs eventually using the information made available through a comprehensive collection of waveforms. The consortium is also working on the inclusion of near-fault effects and on specifications for the compilation of a global database of Soil Classification.

The Pacific Earthquake Engineering Research (PEER) Center leads the project that furthermore features active participation of 27 international experts.

30.2.6 Ontology and Taxonomy

Ontology refers to the entire framework that will guide GEM's development – the set of concepts, and the relationship between those concepts that will allow determination and communication of earthquake risk, and Taxonomy is a part of the ontology, and refers to the classification of things in an ordered system (<http://www.globalquakemodel.org/risk-global-components/ontology-taxonomy>). GEM's Ontology is being developed in an 'Agile' manner (see Section 30.3) through the work of the global components and the development of the OpenQuake software. On the other hand, a group of experts being led by the Alliance for Global Open Risk

Analysis (AGORA) has been tasked within this global component to develop a building taxonomy, classifying the attributes of building typologies around the world. Version 1.0 of the Basic Building Taxonomy is available from GEM Nexus (<http://www.nexus.globalquakemodel.org/gem-ontology-taxonomy/posts>). Taxonomies for other components within hazard and risk assessment such as seismic sources and soil conditions are envisioned for future development. The terminology used in Ontology and Building Taxonomy is being collected into a GEM glossary that will be made available for review and comments from the wider scientific community on GEM Nexus.

30.2.7 Global Exposure Database

This project aims to create the first open database of global building stock and population distribution containing the spatial, structural, and occupancy-related information necessary for damage, loss and human casualty (estimation) models to be deployed in GEM (<http://www.globalquake-model.org/risk-global-components/exposure-database>). The project goes by the name of GED4GEM.

The overall structure of the project is aimed at managing three main tasks:

- Collection, analysis and homogenisation of global databases that may be available and useful for the global exposure database.
- Definition and implementation of a global exposure database based on the output of the previous task.
- Definition of ‘best practices’ aimed at populating missing layers and/or information in specific geographical areas.

The consortium will start by building on existing databases (e.g. UN, regional and other public organisations, governmental building census data, and national statistics) and published literature. It will then aim to collect population and building stock data for all countries of the world.

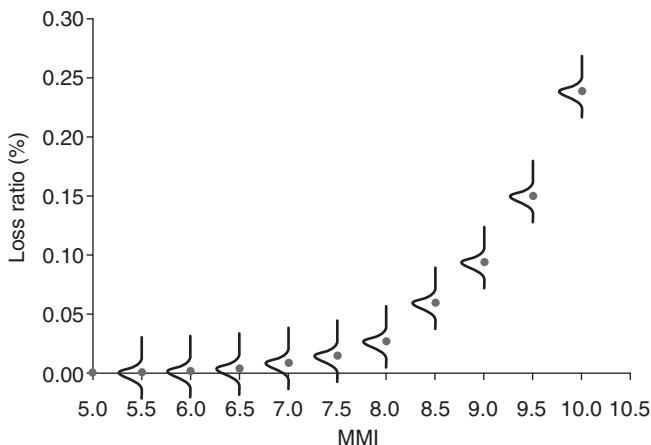
The Global Exposure Database (GED) will be built by combining raw datasets with inference algorithms which will have two levels of detail; the first level is a global set of points representing 30 arc-second quadrilaterals (which can be converted to raster layers with approximately 1 km resolution), with associated aggregate building and population information, the second level of detail accommodates individual buildings as vector data.

Each grid cell will include aggregate information on the population and the number/built area/reconstruction cost of residential buildings, standardised to a common timestamp (2012). The distribution of building typologies, classified with a number of attributes including material, lateral load resisting system, height (as described with the GEM building taxonomy)

and the occupancy of different building typologies during the day and night will be stored. Building-by-building data will also be available for a select number of areas; this data is expected to increase with time through the use of the Inventory Data Capture Tools (see Section 30.2.10). The partners making up the consortium are: the University of Pavia (coordinator), CIESIN-Colombia University, IES-CEA, IGP-CEA, ImageCat, JRC and UN-HABITAT. The USGS and EUCENTRE are advising partners.

30.2.8 Global Physical Vulnerability Estimation Methods

The physical vulnerability project has 3 years' duration and involves nine partners worldwide: University of Colorado at Boulder, University of Chile, Geoscience Australia, EERI, Stanford University, University College London, University of Bath, USGS and Willis (<http://www.globalquake-model.org/risk-global-components/vulnerability-estimation>). The project focuses on relationships between earthquake shaking intensity and physical building damage or related loss, relationships often called seismic (physical) vulnerability functions (Fig. 30.6). The project has two central objectives: to develop procedures for deriving physical vulnerability functions, and to implement those procedures and produce vulnerability functions for a wide variety of building types, following the GEM building taxonomy. The project will not produce seismic vulnerability functions for every building type everywhere in the world, but it will most likely provide a major advance, both in terms of a library of physical vulnerability functions and



30.6 Example discrete vulnerability function and uncertainty description: the probabilistic distribution of loss ratio (e.g. ratio of cost of repair to cost of replacement) conditional on the intensity measure level, which in this case is modified Mercalli intensity (MMI).

standardised procedures for adding to that library. The project will address four distinct approaches to creating seismic vulnerability functions: empirical (using regression analysis to derive seismic vulnerability functions from past observations of earthquake loss experienced by real buildings of a particular type), analytical (using first principles of structural engineering to relate damage and loss to shaking intensity), expert opinion (elicitation of the judgement of experts familiar with the building type of interest to produce a seismic vulnerability function), and empirical-national (seismic vulnerability functions are developed for entire countries, or large sub- and supranational regions, without regard to building type, to best fit past loss data).

Other elements of the project include the following efforts: to develop procedures for aiding the selection among competing seismic vulnerability functions based on a quality rating with five indices: data quality, relevance, rationality, documentation and overall quality; to relate building damage to human casualties and repair costs; to deal rigorously with uncertainties in the capacity, response, collected data, etc.; to apply different regression techniques in the derivation of vulnerability functions and identify the optimum intensity measures for different building typologies; to update seismic vulnerability functions as new data become available through Bayesian updating; to use empirical data for testing analytical functions; and to ensure that the results of the project serve the needs of loss-estimation practitioners. The 3-year project focuses its effort in year 1 on developing guidelines; year 2 is devoted to implementing those vulnerability guidelines and creating global seismic vulnerability functions; and year 3's effort focuses on peer review and efforts to disseminate the products of the research to regional efforts around the world.

30.2.9 Global Earthquake Consequences Database

An international consortium is carrying out the GEMECD, or GEM Earthquake Consequences Database project, which is a database of the consequences (in terms of damage, casualties, socio-economic consequences and recovery data) of past earthquakes due to ground shaking, landslides, liquefaction, tsunamis and fire following for a number of different structures and infrastructures (<http://www.globalquakemodel.org/risk-global-components/consequence-database>). This database will serve to inform users on consequences from past events, as a benchmarking tool for analytical loss models and to support the development of tools to create vulnerability data appropriate to specific countries, structures or building classes. Preparation of an interface enabling the impact damage from future earthquakes to be captured and uploaded to the database is also part of the project, to ensure that consequence data that is of relevance today is avail-

able for the derivation of empirical vulnerability functions, and for using the data for testing analytical functions. The data will be stored following the GEM building taxonomy, such that empirical vulnerability functions (following the guidelines of the physical vulnerability consortium) can be produced, using if desired the outputs of ShakeMap (USGS) for the description of the ground shaking from these events. The project has a 3-year duration and involves 9 partners worldwide: CAR Ltd, CRED, ERN-AL, GNS, KOERI, Kyoto University, Munich Re, SPA Risk and USGS.

The legacy of the project will be a web-accessible database for the ongoing service of GEM users. For current events, GEMECD will serve as a clearing-house of information, posted by users based on the standards and protocols set in the GEMECD documentation. In the long term, GEMECD will be a repository of the most relevant and validated data on consequences of the significant events of the last 40 years around the world.

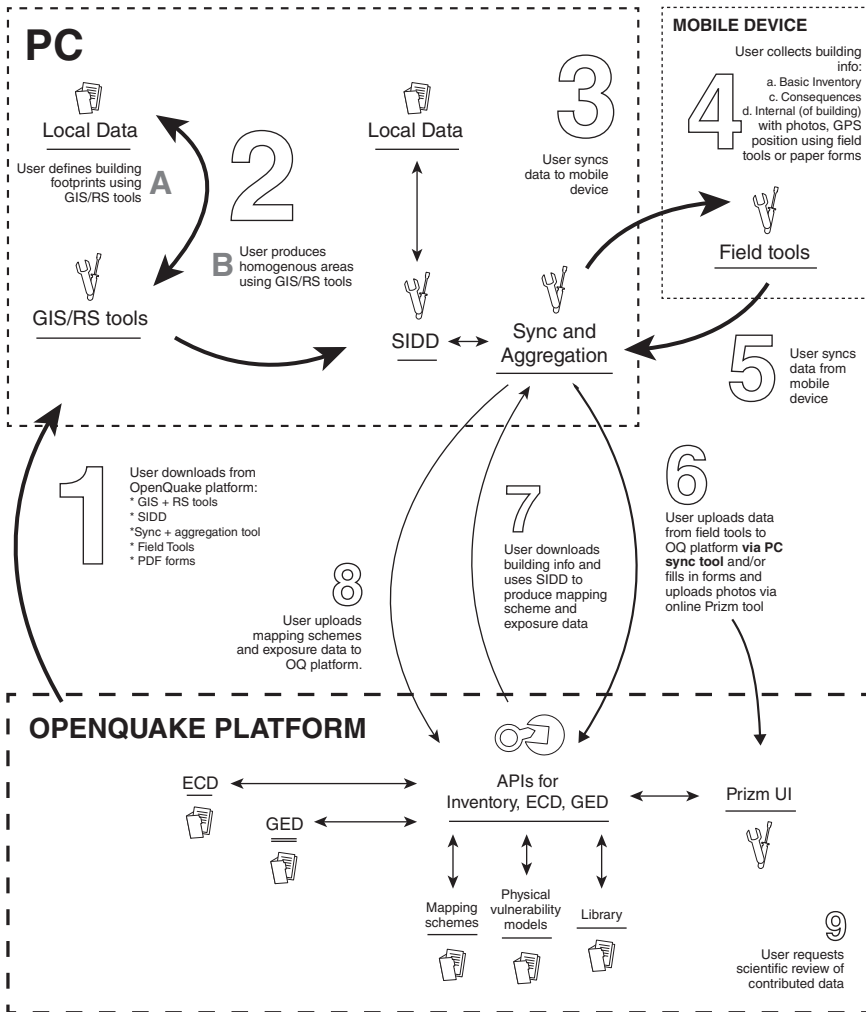
30.2.10 Inventory Data Capture Tools

The Inventory Data Capture Tools (IDCT) project addresses the inventory and damage data development needs of the GEM user community, in developing input to the Global Exposure and Global Earthquake Consequences databases (see Fig. 30.7) <http://www.globalquakemodel.org/risk-global-components/inventory-capture>.

The project aims to provide a fully operational, flexible and integrated suite of tools, protocols and guidelines that are scientifically founded, yet straightforward to use. The project capitalises on the state-of-the-art in remote sensing, GIS, field tools for building data collection, inference and extrapolation methodologies and data integration techniques. As shown in Fig. 30.7, a number of tools will be made available, from handheld devices for recording the attributes of individual buildings, to remote sensing tools for the identification of homogenous areas or for delineating building footprints. A tool for crowdsourcing inventory and damage data collection through the OpenQuake platform will also be developed. The project has a duration of 30 months and involves the following partners: ImageCat, University of Pavia, CAR Ltd, CEDIM-GFZ, SPA Risk, BGS, WAPMERR, whereas Google and Willis are both official collaborators on this project.

30.2.11 Social vulnerability and disaster resilience

When extreme events intersect with societies they become disasters, and the impacts from disasters are rarely suffered equally across space and time (<http://www.globalquakemodel.org/social-vulnerability-resilience>). For the social vulnerability and disaster resilience initiative of GEM, the starting point is to address the differential susceptibility of populations to the



Key:
 GIS: Geographic information system
 RS: Remote sensing
 SIDD: Spatial inventory database
 OQ: OpenQuake
 API: Application programming interface
 UI: User interface

30.7 Inventory Data Capture Tools workflow.

adverse impacts of earthquake events. The latter will begin by focusing on the implementation of methods, metrics and tools for the holistic evaluation of seismic risk that will make use of composite indexing to allow for the comparison of overall risk as defined by the seismic hazard, exposure, and the physical and socio-economic vulnerability and resilience of populations. The project will result in indices and tools for use at various geographic scales. The following indices are being developed:

- A social vulnerability index and database to define the susceptibility of populations to adverse impacts and loss.
- A disaster resilience index and database to gauge the ability of populations to respond to and recover from damaging events.
- An indirect loss index and database implemented as a measure of indirect economic loss vulnerability.

The following tools are also being developed:

- An indicator toolbox for user interaction with the indicator data.
- An integrated risk assessment toolkit for the direct comparison of overall earthquake risk.

While the term earthquake risk often refers to the probability and severity of physical earthquake impacts (Davidson, 1997), the integrated suite of tools developed for this project will allow one to consider the context in which seismic disasters occur by coupling the physical impacts and losses from an earthquake event with the socio-economic circumstances inherent at a given place. This provides several benefits. First, the development of an integrated approach for measuring risk will allow for direct comparison of seismic risk holistically. Second, it will be possible for users to explore how each factor that contributes to overall risk varies throughout space and time. Here, it is feasible to determine the relative contribution of different factors to overall risk to increase the user's awareness of the range of components upon which earthquake disaster risk depends. An additional benefit is the applicability of integrated risk assessments for use in benchmarking exercises. By re-evaluating the benchmarks periodically, trends in earthquake disaster risk can be monitored over time. This feature supports measuring the effectiveness of risk reduction strategies.

30.3 OpenQuake

As mentioned previously, GEM has set up a Model Facility (MF) whose mandate is to undertake the development of the OpenQuake platform that will integrate hazard and risk assessment tools and data and provide these to the community, and to enable and support all modelling developments related to the mission of GEM.

The software that will power the calculations for the regional and global hazard and risk products of GEM (OpenQuake Engine) was openly released in January 2011 through an open source development platform (GitHub) (<http://openquake.org>). An Agile framework² is applied to the development of the OpenQuake Engine software as this caters for the tuning of requirements and priorities as they evolve.

For the hazard calculations, the software will allow users to obtain the following outputs (illustrated in Fig. 30.8) through classical PSHA or through a stochastic event-based approach:

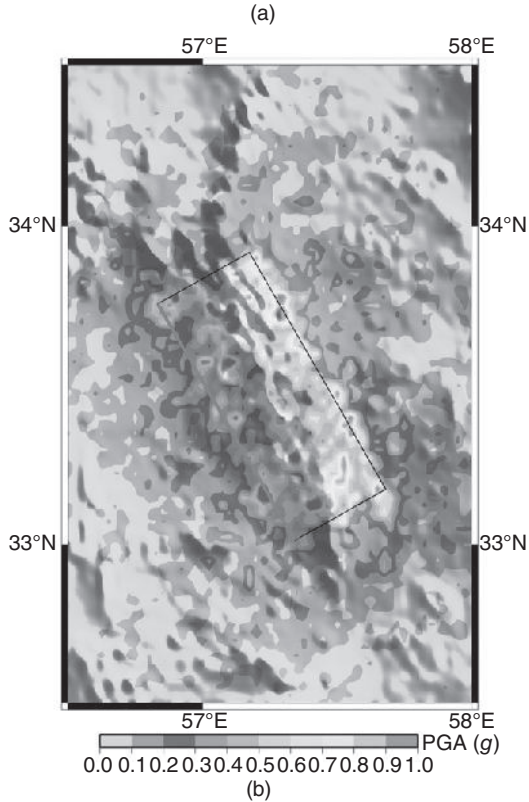
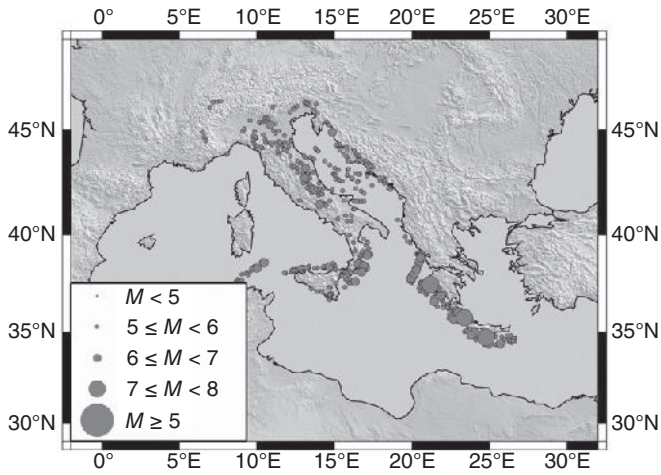
- hazard curves and hazards maps (for different probabilities of exceedance within a given time span); the mean, a given fractile, or for each branch of the logic tree;
- stochastic event sets and associated ground-motion fields;
- disaggregation plots in terms of different combinations of magnitude, distance, epsilon, tectonic region type;
- uniform hazard spectra.

For the physical risk calculations, OpenQuake Engine will allow users to obtain the following results, through a number of different methodologies:

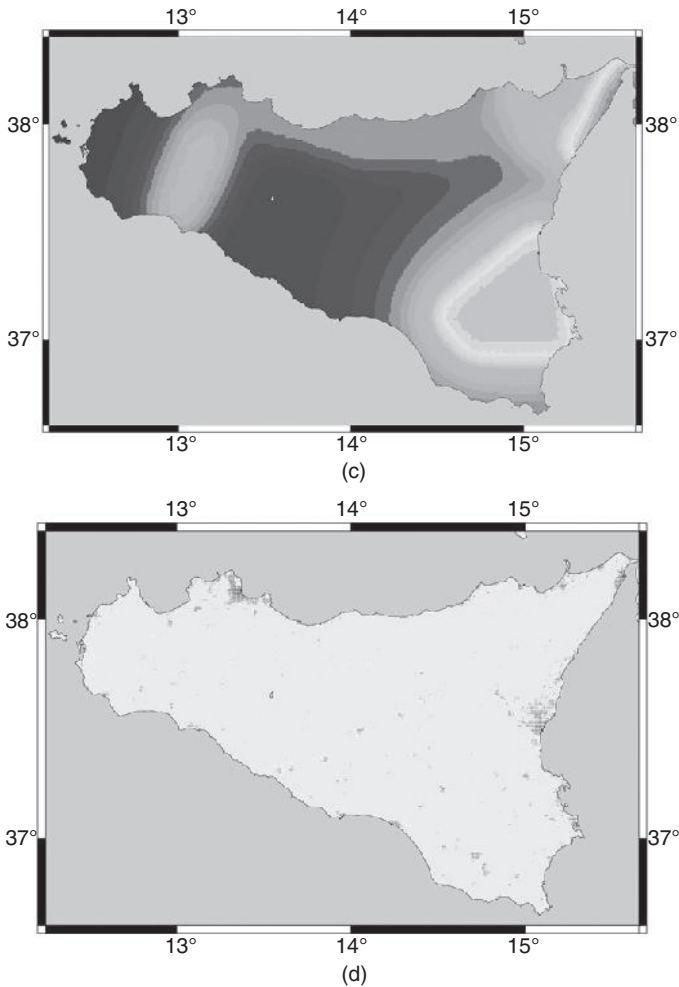
- loss exceedance curves (loss versus probability of exceedance in a given time span, both for single assets and portfolios of assets);
- conditional loss maps describing the geographical distribution of values of loss with a fixed probability of exceedance in a given time span;
- mean loss maps describing the geographic distribution of mean loss within a given time span;
- damage maps describing the geographical distribution of mean number/area/percentage of damage at a given limit state for a given deterministic event;
- loss statistics per event or across all events (mean loss, standard deviation of loss, etc.).

The OpenQuake Engine is hence key to sophisticated calculations. Users will be able to access and interact with the global datasets and models through the OpenQuake platform. Use of the platform in for-profit contexts will have a fair cost, to facilitate academic and basic use worldwide. Reports and manuals with background to the science and tools will also be available, and spaces where users can share, discuss and help improve them. There will be several suites of the OpenQuake platform, attuned to the needs of the various groups GEM aims to serve.

²Agile software development is a group of software development methods based on iterative and incremental development, where requirements and solutions evolve through collaboration between self-organising, cross-functional teams.



30.8 Various typical outputs of the OpenQuake Calculator: (a) a stochastic event set, (b) a ground-motion field with spatial correlation of ground-motion residuals, (c) a seismic hazard map with 10% probability of exceedance in 50 years, (d) a loss map in terms of fatalities with 10% probability of exceedance in 50 years.



30.8 Continued.

The OpenQuake expert suite is meant for academia and for any organisation or governmental agency that wants to do custom hazard and risk assessment and carry out its own modelling and calculation. It is a comprehensive suite of tools for exploring unique global datasets, PSHA, exposure and vulnerability models, indices for social vulnerability, pre-calculated hazard and loss maps for the entire globe, and other pre-calculated output. Users will be able to select GEM models as input to calculations, or import their models of choice (parsers will ensure compatibility of the models of other initiatives with GEM's Natural Hazards Risk Markup Language) and make them suitable to be run on the GEM platform. The suite furthermore

contains an extensive toolkit for modeling and pre-processing data as input for seismic hazard, loss and risk calculations with own or combined datasets. Experts can capture new data, which after quality control will find its way into the applicable databases. And there are tools for post-processing data, performing cost–benefit analyses, and other tools that support decision making.

The OpenQuake essentials suite of tools focused on exploring earthquake risk and decision making is for all organisations and individuals that are in need of detailed information about earthquake hazard and risk, but would not need the option to model or calculate themselves. Users will have access to a large number of datasets with tools to directly understand and visualise them, and to the full series of pre-computed output such as hazard/risk maps, spectra and events sets. Users will also be able to create overlays of a great number of maps related to hazard, loss and risk, and will be able to zoom into areas of their interest, in order to understand what is driving the earthquake hazard and risk results. Cost–benefit analysis, tools for retrofitting prioritisation and other tools to support management and decision making are envisaged as key elements of this suite.

30.4 Outlook for Global Earthquake Model (GEM)

GEM strives to be a community-driven initiative, and provides a number of mechanisms for those interested in seismic risk to become a part of the development process. This chapter has focused on the global components (in particular those related to hazard and physical risk which began during 2010), which involve a large number of international organisations and expert individuals. The global components have kick-started the scientific activities, but GEM will rely on a large network of parties to develop its model, e.g. the members of the Governing Board (which includes both public and private participants), the Scientific Board (with experts in hazard, risk, socio-economic issues and IT), the Technical Advisory Pool (see Fig. 30.2), those involved in regional programmes, the users of GEM Nexus, those providing comments on the website, software developers from the open source community, as well as governments, organisations or even individuals providing data to the OpenQuake platform.

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Strategies for rapid global earthquake impact estimation: the Prompt Assessment of Global Earthquakes for Response (PAGER) system

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Abstract: This chapter summarizes the state-of-the-art for rapid earthquake impact estimation. It details the needs and challenges associated with quick estimation of earthquake losses following global earthquakes, and provides a brief literature review of various approaches that have been used in the past. With this background, the chapter introduces the operational earthquake loss estimation system developed by the U.S. Geological Survey (USGS) known as PAGER (for Prompt Assessment of Global Earthquakes for Response). It also details some of the ongoing developments of PAGER's loss estimation models to better supplement the operational empirical models, and to produce value-added web content for a variety of PAGER users.

Key words: rapid, earthquake impact, fatalities, economic loss, Prompt Assessment of Global Earthquakes for Response (PAGER).

31.1 Introduction

The ability to quickly assess the potential impact (in terms of potential injuries, deaths, physical damage to housing and infrastructure, affected population, and immediate and or long-term economic consequences) due to any earthquake worldwide is essential for coordinating post-earthquake response and recovery efforts. Improved scientific and engineering assessments following significant earthquakes can directly help execute post-earthquake activities efficiently. The activities include assessing immediate search and rescue needs, planning and consolidating available resources for response operations, identifying shelter and health care needs, and providing aid in terms of food, safe drinking water, and sanitation to the affected communities. Each of these tasks can pose a significant challenge depending upon the scale of the disaster and on-the-ground circumstances. Rapid post-earthquake loss estimation systems are developed primarily to address several aspects of these needs; however, there are significant challenges involved in deriving such estimates.

In a given built environment, many factors influence estimation of earthquake ground shaking characteristics and its consequences in terms of casualties and losses. Among these, some of the key factors are: (a) basic earthquake characteristics such as magnitude, location, depth, and source mechanism of an event, (b) geographic site conditions, (c) spatial variation of ground shaking intensity, (d) structural characteristics of building and infrastructure stock exposed, (e) distribution of the population within vulnerable buildings, and (f) ability of the affected population to cope with disasters. The complex interdependencies of these factors within a given built environment are difficult to quantify, making the near real-time damage and casualty estimation problem challenging. Nonetheless, the need to quickly gauge the impact of worldwide earthquakes is critical, given the severe consequences of delays or inaction in the rescue and response phases of a disaster.

Through this chapter article, we briefly discuss the state-of-the-art of rapid earthquake loss estimation systems in the next section. We then introduce the U.S. Geological Survey's Prompt Assessment of Global Earthquakes for Response (PAGER) system, and subsequently provide detailed discussion on three loss-modeling approaches that are implemented within the PAGER system. Towards the end, we provide some early assessments on PAGER's loss estimates for recent earthquakes, and then discuss some of the on-going developments and challenges related to rapid loss estimation efforts.

31.2 State-of-the-art of rapid earthquake loss estimation systems

Rapid post-earthquake loss estimation systems are designed to address specific needs. For example, the Global Disaster Alert and Coordination System (GDACS, <http://www.gdacs.org>) is a multi-hazard disaster monitor and alert system for earthquakes, tsunamis, floods, volcanoes, and tropical cyclones. The World Agency of Planetary Monitoring and Earthquake Risk Reduction (WAPMERR, <http://www.wapmerr.org>) operates a real-time QLARM system that provides estimates of the number of fatalities and injured following significant earthquakes worldwide. Similarly, at the regional level, the Joint Research Activity JRA-3 of the European Union project called NERIES (Network of Research Infrastructures for European Seismology) operates Earthquake Loss Estimation for the Euro-Med Region (ELER, http://www.koeri.boun.edu.tr/depremmuh/eski/ELER/ELER_v3_Manual.pdf) program. ELER aims to estimate earthquake-induced damage, casualties, shelter, and food requirements following an earthquake in the Euro-Mediterranean region. Commercial earthquake catastrophe models, such as those developed by AIR, RMS, and EQECAT,

are also used for estimating insured losses and economic impacts; however, much of the data and models within these platforms are not made publicly available.

Country- or region-specific rapid loss estimation tools have also been developed in the past. Early efforts in this arena include the EPEDAT (Early Post-Earthquake Damage Assessment Tool) system that was developed around the time of the 1994 Northridge earthquake in California, United States (Eguchi *et al.*, 1997). Similar to EPEDAT, the Italian SIGE system, which is the Emergency Management Information System that was developed by the Department of Civil Protection in the mid-1990s, provides a range of capabilities including scenario earthquake loss analysis and post-earthquake damage/loss estimation using INGV's near-real time earthquake parameters (Di Pasquale *et al.*, 2004).

In the United States, the National Institute of Building Sciences and Federal Emergency Management Agency's Hazus software (<http://www.fema.gov/plan/prevent/hazus/>) contains comprehensive datasets and models for estimating potential losses from earthquakes, floods, and hurricanes. It uses geographic information systems (GIS) technology to facilitate the estimation of physical, economic, and social impacts of disasters. Hazus is used for mitigation and recovery as well as preparedness and response.

Following the Hazus development in the United States, many countries such as Taiwan, Australia, Israel, and Canada have developed their own country-specific 'Hazus-like' applications by using and or modifying Hazus methodologies for their domestic applications. These tools now provide an important step forward in identifying, understanding and mitigating potential disaster losses in several other parts of the world.

Stafford *et al.* (2007) further details methodological aspects of seismic hazard, vulnerability, and loss estimation approaches developed over the last decade. Similarly, Erdik *et al.* (2011) provide a brief assessment of the technical and functional capabilities of some of the systems discussed above and summarize the key advances in the evolution of rapid earthquake loss estimation systems over the past decade.

A wide variety of modeling strategies, lack of transparency or open availability of data, models, and tools, their regional nature, and the limited reporting of results make systematic comparison of these various applications difficult. Indeed they often serve different purposes and stakeholders. Most systems are applicable for specific earthquakes or regions. For example, commercial models serve specific clients, but in general they are opaque ('blackbox') and reporting is normally limited to specific economic impacts, normally insured, and typically these numbers are only provided days after a very significant event. Additionally, alerts or loss estimates produced from many of these systems are not publicly available: for ELER, SIGE, and Hazus, aggregated results are only made available to or via the respective

government agencies but not directly to general public, responders, the media, or the international community.

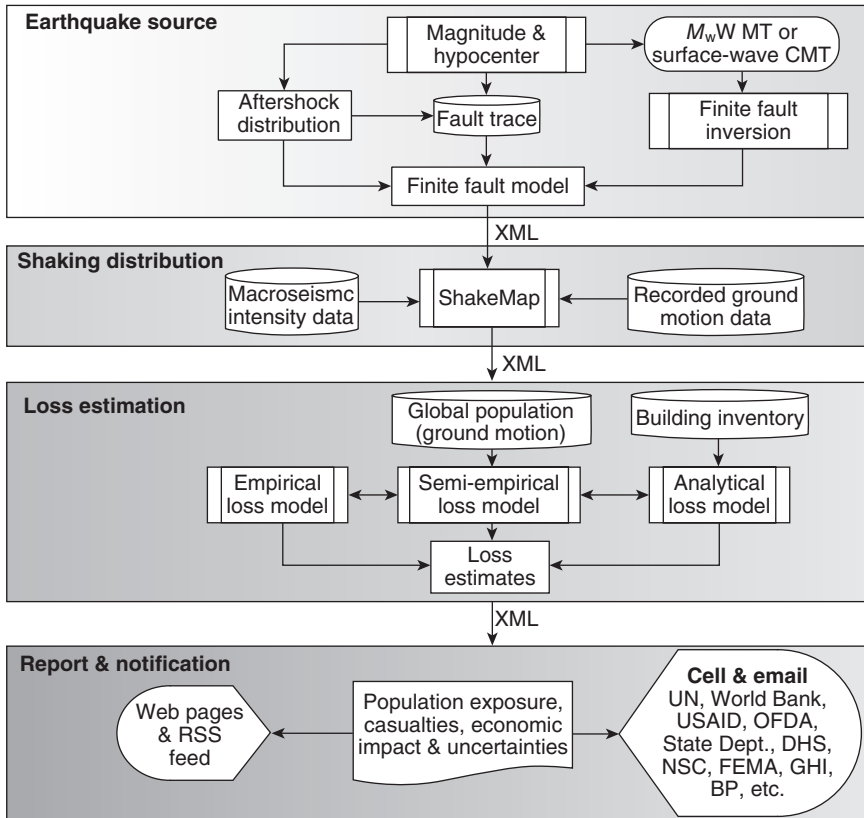
Furthermore, while each of these tools/systems relies on basic earthquake parameters to estimate losses, the modeling assumptions and the methodologies implemented within the tools in order to constrain both earthquake ground shaking and to perform exposure and vulnerability analyses vary greatly. It is thus quite challenging to test the results of one tool or system against another for the same earthquake or scenario since many of these tools and models output different loss metrics at different resolutions.

Despite these differences, these tools and systems (some of them being global and others being region-specific) represent important and much needed development in the field of rapid loss estimation. It is for these reasons above that we embarked on the PAGER project, which was meant to provide an open framework for data collection and dissemination, hazard and loss algorithms, intermediate products to be used by others (namely ShakeMap), and the resulting loss estimates. Documentation of these data and models are provided online and in the peer reviewed literature.

31.3 Prompt Assessment of Global Earthquakes for Response (PAGER) system development

Following the 2004 Sumatra earthquake ($M_w 9.1$) and tsunami, the USGS began the development of a new system PAGER. In 2006, with a prototype PAGER system, the USGS started issuing population exposure results for all significant global earthquakes ($M_w 3.5$ and above in the United States, and $M_w 5.5$ and above globally), providing both estimated intensities as well as estimates of populations exposed to each shaking level. Events with large population exposures at higher modified Mercalli intensities (MMIs) clearly warranted attention. Yet, the differences in vulnerabilities given these exposures were not specifically addressed. Finally, in September 2010, the PAGER system began publicly releasing impact-based alerts (for likely casualties and economic losses) following significant earthquakes around the globe by incorporating empirically based losses. Publicly providing fatality and economic loss estimates required a new way of communicating uncertain losses; therefore, the USGS introduced an earthquake alert scale (Wald *et al.*, 2011).

At the core of the PAGER system there are four modules that are integral to the operational earthquake impact estimation (Fig. 31.1). The first module consists of earthquake source characterization. This operation is dependent upon retrieving earthquake source parameters that are produced by National Earthquake Information Center (NEIC) following large earthquakes, and regional seismic networks in some areas of the United States. These include estimation of the earthquake magnitude, location,



Key:
 MT: Moment tensor
 CMT: Centroid moment tensor
 XML: Extensible markup language
 RSS: Rich site summary

31.1 Modular systems to characterize earthquake source, shaking hazard, loss estimation, and product dissemination operated within the PAGER system.

hypocentral depth, and focal mechanism. These initial parameters aid in developing additional source constraints, particularly the fault rupture dimensions, which are requisite in the process of robust shaking estimation for large earthquakes.

The second module consists of generating a uniform (roughly 1 km square) grid of shaking hazard estimates using standardized strategies developed for the USGS ShakeMap system (Wald *et al.*, 2005). The third module, which deals with earthquake loss estimation, is the primary focus of this chapter and will be subsequently discussed in detail. Finally, the

fourth module consists of disseminating uncertain hazard and impact information rapidly, robustly, and coherently. The importance of properly communicating uncertain losses to a wide variety of users required substantial considerations on this front, and continued development of graphics, web pages and products that aid in this endeavor are also discussed later. These four PAGER modules have been developed in an open environment such that the intermediate data, products, and results are available in near-real time via USGS web pages, and the data, background, and algorithms are openly available through the scientific literature (listed at <http://earthquake.usgs.gov/pager/>).

31.4 Earthquake loss models within the Prompt Assessment of Global Earthquakes for Response (PAGER) system

Researchers in the past have attempted to estimate earthquake-induced casualties, damage, and economic losses at various levels of detail and different scales of resolution (e.g., structure-specific versus regional or aggregate loss estimation). In general, the three approaches that are commonly used for loss estimation are: empirical, hybrid, and analytical. The empirical approach consists of correlating earthquake-induced consequences (such as structural damage, casualties or economic loss) directly in terms of earthquake's ground shaking characteristics. In contrast to the empirical approach, the analytical procedure involves multi-step analyses such as seismic hazard analysis (estimating ground shaking defined in terms of specific shaking parameters, such as peak ground acceleration, PGA and pseudo-spectral acceleration, PSA), structural analysis (assessing response of a structure given its shaking intensity), damage analysis (estimating fragility characteristics for a given response of a structural system), and loss analysis (estimating fatalities and injuries due to structural and non-structural damage) (e.g., NIBS-FEMA, 2009). In short, earthquake damage and loss estimation are performed by modeling structural response in terms of engineering-based seismic demand, and using nonlinear static or dynamic analysis procedures. The hybrid approaches vary largely from one study to another, but in general the process consists of simplifying the engineering-based structural and damage analysis procedures using expert-judgment or using empirical data or both. It is still performed at the building level and thus can be applied to regional or building-specific casualty assessment (ATC, 1985; Coburn *et al.*, 1989; Shiono *et al.*, 1991; Murakami, 1992; Yamazaki *et al.*, 1996; Shakhramanian *et al.*, 2000).

The PAGER loss estimation module consists of three separate loss-estimation approaches; however, the automated impact-alert estimation is cur-

rently based only on PAGER's empirical loss models. It was recognized that for some regions where numerous and frequent damaging earthquakes have occurred in the recent past, the empirical fatality and economic loss estimation models can be used for quickly assessing the overall impact for operational response purposes (Wald *et al.*, 2008a). The semi-empirical and analytical loss estimation models implemented within the PAGER system require building inventory, vulnerability, and economic exposure datasets at the country level. These models can provide approximate damage and loss estimates only in regions where the requisite-quality data and models are available. The following subsections describe each of these models, though for brevity the reader is referred to relevant literature for additional details.

31.4.1 Empirical model

Empirical modeling has been performed in a variety of ways depending upon the earthquake damage data available. Researchers in Japan attempted casualty estimation as early as the 1950s. Kawasumi (1951) developed a measure of earthquake hazard and expectation of maximum intensity in Japan. Similarly, early casualty estimation efforts in the United States were scenario specific and based on estimation of the casualty rate per 100 000 people and use of engineering judgment (Algermissen *et al.*, 1972). Ohta *et al.* (1983) developed an empirical relationship for estimating the number of casualties as a function of the number of completely destroyed houses. Oike (1991) proposed a relationship between earthquake magnitude and earthquake fatalities. A log-linear relationship of fatalities as a function of magnitude and population density was derived by Samardjieva and Badal (2002) by analyzing 450+ global earthquakes. Nichols and Beavers (2003) studied the fatality catalogue of the twentieth century and established a bounding function using the fatality count and the USGS-assigned earthquake magnitude.

The PAGER empirical model estimates earthquake fatalities or expected economic losses $E(L)$ at each level of shaking intensity s (typically represented by MMI). The estimates are based on the exposure (either the total population exposed, or the total economic exposure, i.e., the monetary value of all the exposed physical assets) and the *loss ratio* (which can be fatality rate v or economic loss ratio r) corresponding to each level of MMI directly obtained from ShakeMap (<http://earthquake.usgs.gov/shakemap/>). Total losses are computed by summing the losses from different intensities to estimate total losses for a given earthquake j :

$$E(L_j) = \sum_s (\text{Exposure})_{s,j} \times (\text{Loss Ratio})_{s,j} \quad [31.1]$$

Empirical fatality model

In order to estimate total fatalities in any given earthquake, the key challenge is to estimate both population and fatality rate at each intensity level. PAGER employs hazard input from ShakeMap and the LandScan population database (<http://www.ornl.gov/sci/landscan/>) so in near real-time population exposure (the total number of people) at each level of shaking intensity can be estimated by simply overlaying a shaking intensity map on top of a gridded population map and then summing the population in each intensity level bin.

The country-specific, mean fatality rate is estimated solely as a function of the MMI. The fatality rate is expressed as the number of fatalities divided by the total population exposed at each level of shaking intensity. Estimating the fatality rate at each level of shaking intensity for all regions of the world is a difficult task. In order to estimate this rate from historical data for a given earthquake, we would need to know the total population and also the number of fatalities at each intensity level. Casualty survey data generally do not provide geospatial fatality statistics but rather aggregated statistics either as city totals or, usually, only as an earthquake total.

Instead of collating data on fatality occurrences at each intensity level and then estimating the fatality rates, Jaiswal *et al.* (2009) showed that the fatality rate ν could be expressed in terms of a two-parameter lognormal cumulative distribution function of shaking intensity s written as:

$$\nu = \Phi \left[\frac{1}{\beta} \ln \left(\frac{s}{\theta} \right) \right] \quad [31.2]$$

where Φ is the standard normal cumulative distribution function. In the present application, the shaking intensity s ranges from 5.0 to 9.0 (i.e., MMI V to IX), where total exposure at MMI IX and above is aggregated and assigned to IX. The terms θ and β are the two unknown parameters of the distribution and correspond, respectively, to the mean and standard deviation of the natural logarithm of the intensity measure (by definition, the logarithm of the intensity measure is normally distributed). Thus, for a given country with a number of earthquakes with known total losses, one can estimate these two parameters, without knowledge of the fatality rates geospatially. Once the two parameters of this functional form are known, one can determine the fatality rate ν at each intensity level s . Let $P_i(s_j)$ denote total estimated population exposed to shaking intensity s at level j for an event i . The expected number of fatalities E_i can be denoted as:

$$E_i = \sum_j \nu_i(s_j) P_i(s_j) \quad [31.3]$$

If we suppose that O_i is the number of recorded deaths for an earthquake i , then we can determine parameters of the fatality rate in such a way that

the residual error ε (that is, the error estimate between estimated and recorded deaths) is minimized:

$$\varepsilon = \ln \left[\sqrt{\frac{1}{N} \sum_{i=1}^N (E_i - O_i)^2} \right] + \sqrt{\frac{1}{N} \sum_{i=1}^N [\ln(E_i/O_i)]^2} \quad [31.4]$$

We use a standard iterative search technique often called Nelder–Mead optimization available in MATLAB® for minimizing the objective function (Eq. 31.4) with the two free parameters of the distribution function, θ and β . The combined-norm approach is simple and suitable for countries with at least three or more fatal earthquakes in the catalog, thus providing earthquake fatality rates for a large number of countries. Although the choice of the two-parameter lognormal cumulative distribution function for defining the fatality rate, and the selection of a specific minimization norm (shown in Eq. 31.4) were based on limited analysis. Owing to the limited observation period of earthquakes (for damage and losses covering only post-1973 earthquakes) for any given country, there are only a few to several high-loss earthquakes in a set dominated by frequent low-loss earthquakes. The proposed norm provides reasonable balance for all historical losses in a given country, without losing accuracy towards estimating the few high-loss earthquakes (Jaiswal *et al.*, 2009).

In order to estimate the uncertainty associated with the model's prediction for future earthquakes, we define ξ as a variable (a measure of the variability or dispersion associated with actual deaths) representing the standard deviation of the logarithm of actual deaths given the logarithm of model-estimated deaths. If the dispersion is constant across areas of low and high losses in a scattergram for a given country, we can estimate this dispersion using the following expression:

$$\xi = \sqrt{\frac{1}{N-2} \sum_{i=1}^N [\ln(O_i) - \mu_{\ln O_i | \ln E_i}]^2} \quad [31.5]$$

If the conditional dispersion ξ varies between low and high losses, the regression would have to be obtained differently due to heteroscedasticity. The expected value of $\ln(O_i)$ given $\ln(E_i)$ is assumed to be the same as model-estimated loss $\ln(E_i)$ in the forward prediction as given by Eq. 31.3. Note that we do not have all the information necessary to systematically quantify the total variability associated with actual impacts from any given earthquake, given the real-time computation involved in PAGER operations. We only use the model's predictability to hindcast past earthquake losses as a measure to infer the uncertainty that may be associated with future losses. The sampling uncertainties and the uncertainties associated with heteroscedasticity or bias or due to the factors that are not fully reflected in the historical earthquake catalog are not considered in the present calculations. Given the objective of the PAGER system to provide

quick assessment of likely impact, we chose to disseminate earthquake loss alerts both in terms of color-coded alert levels (selected based on that level containing the median loss estimate) and in terms of associated uncertainty about the model-estimated losses, obtained using Eq. 31.5.

As an example of country-specific model development, we present the case of Turkey here. The EXPO-CAT catalog v1.0 (1973–2008, see Allen *et al.*, 2009) contains 81 earthquakes in Turkey, of which 40 were fatal. We used both fatal and significant, but excluding non-fatal, earthquakes to estimate the empirical model parameters for Turkey. The largest earthquake that struck Turkey since 1973 was the magnitude 7.6 Kocaeli, Turkey earthquake on August 17, 1999, which caused 17483 deaths and 43953 injuries. The estimated population exposure was 571 400 people at shaking intensity IX and above, and approximately 1.53 million people at shaking intensity VIII. The fatality rate estimated using an empirical model with parameters $\theta = 11.07$ and $\beta = 0.11$ is shown in Fig. 31.2(b) with log residual error (ξ) of 1.53. The model corresponds to a fatality rate of one death per 38 people exposed to shaking intensity IX, and one death per approximately 900 people exposed at shaking intensity VIII. The fatality rate at the lower intensity is extremely low which is evident from the fact that Turkey has experienced numerous moderate-shaking earthquakes over the last 35 years that did not cause any fatalities.

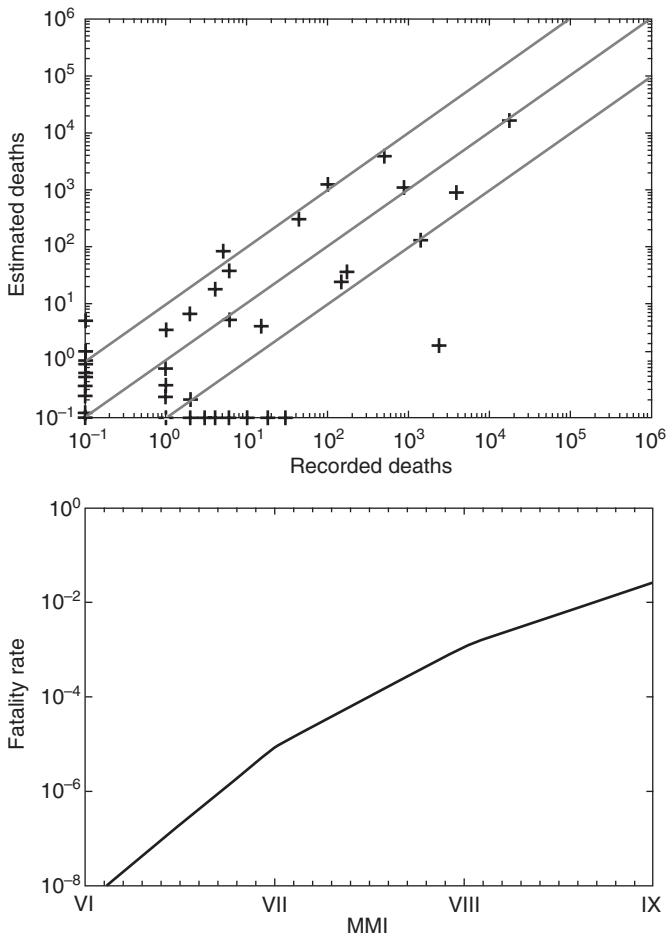
Empirical economic model

Following the development of the empirical fatality model, Jaiswal and Wald (2011) provided a logical extension of that approach in order to empirically estimate economic losses for the PAGER system. In principle, the economic loss ratio r , defined as the total direct economic loss (that includes structural, non-structural and content losses), normalized by the total economic exposure, is similar to the fatality rate v defined earlier.

The total economic exposure in each shaking intensity bin can be approximated in terms of the total gross domestic product (GDP) exposed (which is estimated as total population exposed times per capita GDP of the region) and the country- or region-specific exposure correction factor α as:

$$\text{Eco.Exposure}_s = \alpha_{\text{region}} \times \text{Total GDP}_{\text{region},s} \quad [31.6]$$

The exposure correction factor α is simply the ratio of per capita wealth to the per capita GDP estimate of the country. The per capita wealth data are taken directly from the World Bank (2006), whereas the per capita GDP estimates, obtained for the year 2008, are used for the derivation of an exposure correction factor for each country. The World Bank study considered several important factors including estimates of the capital produced, the physical capital stock, consumption, investment, natural resources, and



31.2 Recorded and estimated fatalities (shown in top portion) for historical earthquakes obtained using fatality ratio functions (shown in bottom) derived for Turkey ($\theta = 11.07$, $\beta = 0.11$, $\xi = 1.53$).

other factors in an effort to estimate the per capita wealth in the year 2000 for 119 countries. The wealth estimate per capita is an important and useful indicator of economic well-being of a region or country. Since the per capita wealth data are only available for the year 2000 (and not for other years in the past), they are assumed constant here. However, if higher resolution per capita wealth data become available for different time periods, one could easily compute the exposure correction factors α_{region} specific to each region, and use them for better hindcasting historical losses. Similarly it is also plausible to infer per capita economic exposure using other indicators, e.g., household wealth data (Davies *et al.*, 2007). Moving forward, we are

exploring the potential of incorporating high-resolution subnational GDP dataset, in order to better characterize the spatial variation of economic exposure within a given country.

Following the formulation as described in Eq. 31.3, the total expected economic loss following an earthquake i could be estimated by summing the product of total economic exposure and loss ratio at each shaking intensity level j using the following equation:

$$E_i = \sum_j r_i(s_j) P_i(s_j) \quad [31.7]$$

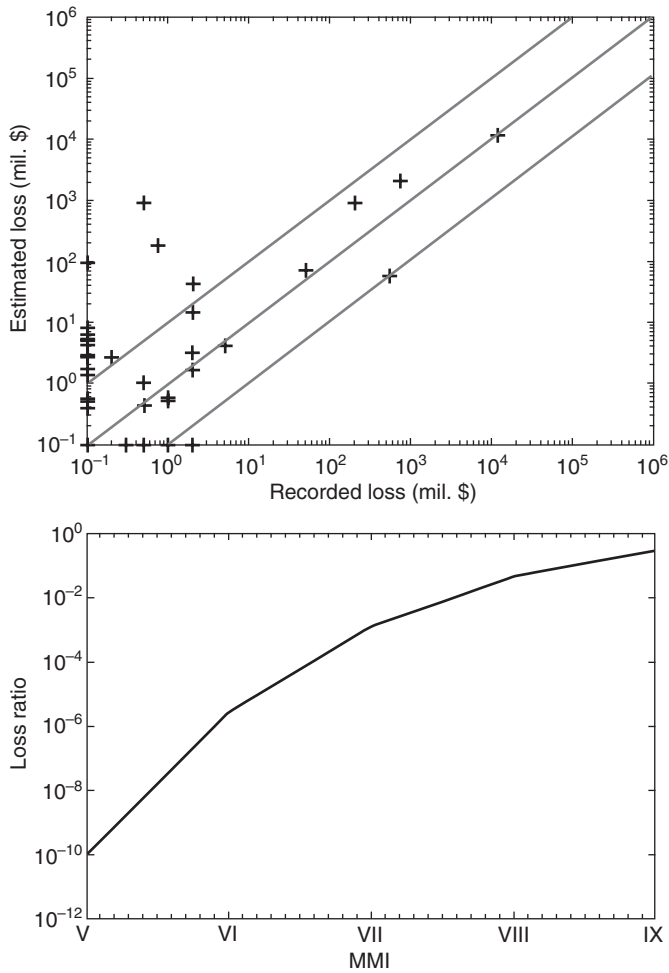
The loss ratio function $r(s)$ is defined in terms of a two-parameter log-normal cumulative distribution function of shaking intensity s as shown in Eq. 31.2. If we suppose that O_i is the recorded economic loss (in millions of US dollars, USD, taken from the Munich Re catalog as shown in Table 31.1) for an earthquake i , and there are N such earthquakes for that country, then we can determine the parameters of the loss ratio function by minimizing the total error ε between expected loss $E(L)$ (also termed as model-estimated loss, abbreviated as E_i) and reported or observed loss O_i . This is analogous to the fatality rate estimation approach defined in the previous section.

Figure 31.3 illustrates recorded versus model-estimated losses for historical earthquakes in Turkey and Italy. Each symbol on the top portion of the plot represents one earthquake for which losses are estimated using the population exposure, GDP, the correction factor, and the loss ratio function shown in the bottom portion of the country-specific plot. We mapped 62 earthquakes in Turkey (1980–2008) with the EXPO-CAT catalog for which population exposure and economic loss estimates were available. Among these, the August 17, 1999, Izmit earthquake caused \$12 billion of direct economic loss according to the Munich Re catalog. Using a nonlinear optimization procedure, we determine the loss ratio function parameters θ and β that best predict both high and low economic loss events. In the case of

Table 31.1 Snapshot of Munich Re economic loss data for selected events

MR number	Country	Direct overall loss (USD mil., original values)	Insured loss (USD mil.)	Deaths (total)
MR199401A043	Northridge CA USA	44 000	15 300	61
MR199501A046	Kobe, Japan	100 000	3 000	6 430
MR199909A017	Chi Chi Taiwan	14 000	750	2 368
MR200805A004	Sichuan China	85 000	300	69 227

Source: NatCat Service (2008).



31.3 Recorded and estimated economic losses (shown in top portion) for historical earthquakes obtained using loss ratio functions (shown in bottom) derived for Turkey ($\theta = 9.46$, $\beta = 0.1$, $\alpha = 11.93$).

Turkey, the estimated parameters are $\theta = 9.46$, $\beta = 0.1$ with normalized standard deviation $\xi = 1.3$ as shown in Fig. 31.3. The economic loss ratio function estimated \$11.46 billion USD of total economic loss for the 1999 Izmit Turkey earthquake (+ symbol at far right).

31.4.2 Semi-empirical model

Increasing urbanization of the world's population in earthquake-prone regions has resulted in a significant increase in the earthquake risk.

Earthquakes in the past have repeatedly illustrated the extent of undesirable consequences (in terms of loss of human life and property) whenever they have struck vulnerable areas of populations. Historically, earthquakes have caused gravely disproportionate losses in certain countries. Except for a few developed nations, such as the United States, New Zealand, and Japan, most fatality or casualty-related losses have occurred in countries from the developing world, where vulnerable building stock caused huge death tolls. Spence (2007a) summarized some of the specific achievements and failures of earthquake risk reduction in the last 40 years for countries from both the developed and developing world.

The dominant commonalities shared by vulnerable countries, are low per capita income, rapid population growth followed by urbanization, development with little attention to safety, poor earthquake-resistant building design codes, and/or lack of enforcement of existing building codes. Despite some dedicated efforts within a number of countries to develop new building codes, the existing residential building stock is hardly affected by these changes. Bird and Bommer (2004) observed that unreinforced masonry and reinforced concrete buildings are the most vulnerable to collapse. Building collapses do remain the dominant cause of deaths and injuries in earthquakes (Coburn and Spence, 2002). It appears that a broad level of understanding of regional building stock and its vulnerability during past earthquakes could provide an opportunity to attempt a first order estimation of likely earthquake-related casualties from earthquakes.

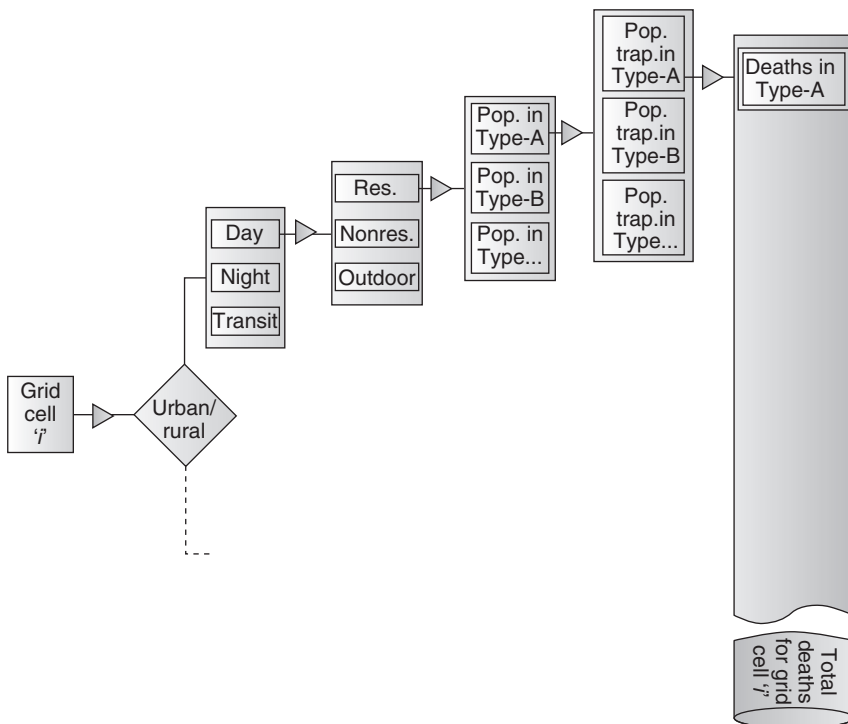
As discussed in the previous section, the empirical vulnerability models rely directly upon the relationship between population exposure and earthquake fatality rates at each level of shaking intensity in order to estimate total fatalities. What is added in the semi-empirical model is detailed data on building types, their collapse vulnerability, and the fatality rate associated with structural collapses. In the current implementation, we focus primarily on estimating structural collapse, which usually is the dominant cause of earthquake fatalities.

The quantitative model incorporates shaking hazard on a latitude/longitude grid from the ShakeMap system, population exposure by building type within a given country, fragility of building type expressed in terms of probability of collapse at each intensity, and fatality rate given building collapse. Building inventory and vulnerability data have been compiled at the country level. For each country, the inventory database provides the population distribution according to different structure types for two occupancy types (residential and non-residential) and two population density types (urban and rural). Details about inventory database development are discussed in Jaiswal and Wald (2008).

At each grid cell (approx. 1 km by 1 km), we have the total population, urban/rural categorization of that cell based on Global Rural–Urban

Mapping Project (GRUMP v1.0, see <http://sedac.ciesin.columbia.edu/gpw/>) dataset, and the ShakeMap shaking intensity estimate. We use three time intervals to define time of day (day, night, and transit). For each earthquake's time of day, we estimate the total indoor population among three broad occupancy categories, i.e., residential, non-residential and outdoor population, using a global workforce database (Jaiswal and Wald, 2010). Similarly, for vulnerability assessment, we collated the data specific to collapse probability by construction classes from different countries, using both empirical data as well as expert judgment surveys, and then developed a set of collapse probability functions (Jaiswal *et al.*, 2011).

By combining Hazus level 4 injury rates for the complete damage state with collapse (NIBS-FEMA, 2009) and fatality rates by structure type for the D5 damage grade proposed by Spence for the European LessLoss study (Spence, 2007b), we developed fatality rates specific to PAGER structure types (PAGER-STR, Jaiswal and Wald, 2008). Loss computations are performed at the grid-level as shown in Fig. 31.4 in which the grid-specific



31.4 Overview of grid-based population exposure and hazard information used to compute earthquake damage and losses within PAGER's semi-empirical and analytical loss models.

population count and shaking estimate form the key input for damage and loss analyses for both the semi-empirical and analytical models. The population exposure, inventory and vulnerability data are used to estimate total casualties and thus account for spatially varying shaking intensity and exposure at different grid cells as discussed in Jaiswal and Wald (2010).

31.4.3 Analytical model

In the analytical approach, the same building inventory and occupancy-related databases derived for the semi-empirical approach are used; however, the structural collapse rates are determined from the Hazus the capacity-spectrum methodology which estimates the response of a structure from spectrum demand and spectral-capacity curves (NIBS-FEMA, 2009). In principle, the demand spectrum represents the site-adjusted input ground motion, typically accounting for nonlinear structural behavior and hysteretic energy dissipation, whereas the spectral capacity of a structure is expressed in terms of an idealized curvilinear graph defined by its yield and ultimate control points. The capacity-spectrum method provides an estimate of median response of an idealized nonlinear single degree of freedom (SDOF) oscillator where the spectral-capacity and demand curves intersect. This point is referred to as the performance point. The spectral displacement S_d associated with the performance point forms an input to fragility functions that give the probability of different damage states.

The damage and casualties associated with slight, moderate, and extensive damage states are ignored for PAGER purposes since they form a very small fraction of total fatalities. Porter (2009) simplifies the iterative process for PAGER and directly tabulates the mean-collapse fragilities and indoor fatality rates as a function of 5% damped spectral accelerations at either 0.3 s or 1.0 s periods. The fatality rates given structural collapse are the same as in the semi-empirical approach. The mean-collapse fragilities are derived primarily for Hazus model building types (MBTs) using the Hazus capacity and fragility parameters. For non-US building types, the basic MBTs were mapped to PAGER-STR types (see Jaiswal and Wald, 2008, for details) using a preliminary mapping scheme. In general, the structural capacities are quite different between the basic MBTs and corresponding PAGER-STR as discussed in D'Ayala *et al.* (2010).

In order to understand and evaluate the differences between structural capacity and fragility parameters and to propose procedures for estimating collapse vulnerability of non-US building types, a parallel effort is underway under the auspices of the WHE (World Housing Encyclopedia)-PAGER Project. Preliminary analyses suggest that there could be significant differences in estimated structural capacities of reinforced concrete types depending upon the modelling assumption or methodology adopted for

inelastic analyses for given structure types. The differences could be larger in case of the ultimate displacement S_{du} , which is more difficult to estimate (given the structural model-specific assumptions involved) during inelastic analysis. Nonetheless, significant progress has been made in compiling the requisite data for non-US building types to be used within the operational PAGER system.

31.5 Earthquake impact scale

For all significant earthquakes around the world, the NEIC locates, assesses and disseminates earthquake information through a variety of means. Its user-subscribed Earthquake Notifications Service (ENS; Wald *et al.*, 2008b) presents fundamental improvements for USGS earthquake alerting in that users can customize their alerting levels based on magnitude, hypocenter, and time of day, and they can receive messages on multiple devices or electronic addresses (each with, potentially, different triggering criteria) in near-real time. ENS alerts are sent to over 345 000 subscribed users including critical responders, non-governmental organizations (NGOs), governments, the media, as well as individuals (see Wald *et al.*, 2008b).

Despite the benefit of ENS over earlier list-servers, the alerting criteria are currently limited to magnitude- and location-based triggers. While experienced users can take advantage of earthquake magnitude, depth, and location to make informed decisions, most users do not have sufficient expertise to tie these parameters to population exposure and region-specific vulnerability in order to confidently assess the potential impact. In addition, users must either set a low magnitude trigger level not to miss significant events (and potentially get many more alerts than desired), or risk missing an important event by setting a higher threshold.

PAGER employs a new earthquake impact scale (EIS) that is based on two complementary criteria. The first criterion is the estimated cost of damage; this is most suitable for domestic events and those in earthquake-resistant communities. The second criterion, representing the estimated ranges of fatalities, is generally more appropriate for global events, particularly in developing countries. Simple thresholds, derived from the systematic analysis of past earthquake impacts and associated response levels, turn out to be quite effective in communicating predicted impact and response needed after an event, characterized by alerts of green (little or no impact), yellow (regional impact and response), orange (national-scale impact and response), and red (international response). Corresponding fatality thresholds for yellow, orange, and red alert levels are 1, 100, and 1000, respectively. For damage impact, yellow, orange, and red thresholds are triggered by estimated losses reaching \$1 M, \$100 M, and \$1 B, respectively.

The rationale for this dual approach to earthquake alerting stems from the recognition that relatively high fatalities, injuries, and homelessness dominate in countries where local building practices typically lend themselves to high collapse and casualty rates. It is these impacts that drive prioritization for international response. In contrast, it is often financial and overall societal impacts that trigger the level of response in regions or countries where prevalent earthquake resistant construction practices greatly have reduced building collapse and resulting fatalities. Since PAGER calculations are available well in advance of ground-truth observations or news accounts, PAGER information can play a primary alerting role for domestic as well as international earthquake disasters. For US events, the trigger threshold is lower than the global magnitude trigger, with Shake-Maps produced for magnitude 3.5 to 4.5 depending on the region.

An example of the PAGER summary product, or *onePAGER*, is presented in Plate VI (between pages 452 and 453). This summary page shows the earthquake impact alert produced by operational PAGER system within minutes following the M_w 9.0 Tohoku earthquake in Japan indicating a red alert for economic losses (median, i.e., 50 percentile economic loss value from shaking-only causes exceeding 8.0 billion USD). The PAGER alert indicated that there was a likelihood of hundreds of shaking-related fatalities and that economic losses induced from shaking-related damage could be widespread, and thus the event would require national or international level response based on historical precedence. Note that the losses estimated by the PAGER system are from shaking-related causes only, and thus it does not account losses caused due to landslide, liquefaction, tsunami, or fires following earthquakes. According to recent estimates, this earthquake caused at least 15867 (NPAJ, 2012) deaths, mainly from tsunami-related causes, with a much smaller fraction of total deaths, in the hundreds, due to shaking-related causes. Similarly, this event caused an enormous economic impact of the order of 235 billion (*Los Angeles Times*, 2012), of which at least 77 billion (Khazai *et al.*, 2011) might have been attributed due to shaking-related causes.

When the two individual alert estimates differ, the higher alert level is automatically chosen by the PAGER system. Each individual alert level is based on the expected value of loss; uncertainty in the alert level can be gauged by the histogram showing the percentage likelihood that adjacent alert levels (or loss/fatality ranges) occur. Accompanying text clarifies the nature of the alert based on experience from past earthquakes and provides context on the total economic losses in terms of the percent of the GDP of the country affected. Not all the uncertainties are incorporated while determining the earthquake fatalities and associated economic losses, and hence the actual impact of the earthquake may be different from the automatically generated PAGER alert estimates.

Though PAGER uses simple and intuitive color-coded alerting criteria, it preserves the necessary uncertainty measures by which one can gauge the likelihood for the alert to be over- or underestimated. Utilizing the earthquake impact scale, PAGER's rapid loss estimates, despite their uncertainties, can be used to adequately recommend alert levels and suggest appropriate response protocols. Requiring or awaiting observations or loss estimates with a high level of accuracy may delay response and could eventually result in increased losses.

31.6 Loss estimation for recent earthquakes

The EIS scheme helps to organize *quantitative* information in a *qualitative* way such that decision makers can take *effective* and *timely* actions under the conditions of uncertainty. Following a sizable earthquake (with minimum threshold of $M_w 3.5$ within the United States, and $M_w 5.5$ worldwide), the arrival of a ShakeMap automatically triggers the PAGER system to produce impact alerts.

After one and a half years of operation, we can now provide a summary of the performance of PAGER's impact-based alerting for a subset of the events. A complex system such as this requires constant examination, reanalysis, and updating. The PAGER team has analyzed 746 earthquakes from September 2010 through September 2011 for potential losses, 70 of which warranted alerting PAGER email recipients (see Plate VII (between pages 452 and 453)). Of these, 39 had earthquake shaking-based alerts of 'yellow' or greater, indicating an estimated economic loss of 1 million USD or higher and/or estimated fatalities of one or more. During this period, 707 'green' alerts were generated; in many cases these 'no concern' alerts were also incredibly valuable for decision-makers, often avoiding unwarranted attention despite earthquakes with large magnitudes. For the 16 EIS Alert PAGER runs in this period, the initial Summary Alert level was correct six times, an over-prediction five times, and an under-prediction five times. The final Summary Alert level was correct nine times, with most of them resulted to be high-consequential earthquakes, an over-prediction six times, and an under-prediction one time. Even when under- and over-predicting, results were within the neighboring alert level.

Table 31.2 provides a list of earthquakes with estimated and reported economic losses for some of the significant earthquakes of 2010–2011. There are large variations in reported economic losses for many earthquakes in the list.

We used widely cited loss estimates and compared them with the PAGER estimated median economic losses obtained from the most recent version of the ShakeMap. In general, the median loss estimates varied from 25% to 250% of the reported losses and, in most cases, they fall within the proper

Table 31.2 Economic loss estimates produced by the PAGER system for recent worldwide earthquakes

Earthquake	Reported losses (in USD)	Estimated direct shaking-related losses (median estimate ¹ in USD)	Loss fraction (reported/ estimated)
M7.0 January 12, 2010 Haiti earthquake	7.8 billion ² (4.3 billion direct)	3.0 billion	1.43
M8.8 September 24, 2010 Chile earthquake	30 billion ³	15 billion	2.0
M7.0 September 3, 2010 Canterbury, New Zealand earthquake	2.2 to 2.9 billion ⁴	2 billion	1.10 to 1.45
M6.1 February 22, 2011 Christchurch, New Zealand earthquake	16.5 to 25 billion ⁵	37 billion	0.45 to 0.68
M9.0 March 11, 2011 Tohoku, Japan earthquake	At least 77 billion ⁶ (direct losses due to shaking only)	31 billion	2.48
M6.0 June 13, 2011 Christchurch, New Zealand earthquake	4.83 billion ⁷ (6.0 billion NZ dollars)	4.0 billion	1.21
M7.1 October 23, 2011 Eastern Turkey earthquake	500 million to 1.0 billion ⁸	2.0 billion	0.25 to 0.50
M5.8 August 23, 2011 Virginia, USA earthquake	200 to 300 million ⁹	236 million	0.85 to 1.27
M6.0 May 20, 2012 Bologna, Northern Italy	at least 626 million	~2 billion	0.31

¹Median value of shaking-related direct economic losses from most recent version of ShakeMap system.

²<http://www.haitispecialenvoy.org/relief-and-recovery/key-statistics/>.

³http://www.aon.com/attachments/reinsurance/201012_if_annual_global_climate_cat_report.pdf.

⁴http://www.nzherald.co.nz/business/news/article.cfm?c_id=3&objectid=10709579.

⁵<http://www.stuff.co.nz/business/rebuilding-christchurch/4984173/Quake-rebuild-will-eat-into-GDP>.

⁶<http://www.itas.fzk.de/tatup/113/khua11a.htm>.

⁷<http://tvnz.co.nz/national-news/christchurch-quake-bill-could-rise-6-billion-4227216>.

⁸<http://www.erra.pk/TurkeyEQ2011/FactFile.asp>.

⁹http://www.washingtonpost.com/local/region-tallies-earthquake-damage-mostly-uninsured/2011/08/24/gIQAfDxScJ_story.html.

interval of PAGER loss estimates, most certainly within the order of magnitude loss-ranges originally desired for the PAGER rapid loss estimation system.

31.7 Prompt Assessment of Global Earthquakes for Response (PAGER) products and ongoing developments

The primary objective of ongoing development of the PAGER system is to ingest readily available scientific data on specific earthquakes and then create information products that help communicate an earthquake's hazards and potential impacts to a wide variety of users. Although PAGER provides fatality and economic impact-based alerts immediately following any significant earthquake, the media, general public and responders have ever-increasing expectations for useful, rapid information. This is especially true given the severe consequences of recent large earthquakes, e.g., the 2008 Wenchuan, China, 2009 L'Aquila, Italy, and 2010 Haiti earthquakes.

The PAGER system is now at the point where re-evaluation of the fundamental goals and implementation strategies is in order. Several years of experience with operations, hazard calculations, loss models, alerting protocols, analysis of our success rate for recent events, as well as direct interactions with many users allow us to review the current status of PAGER and set priorities for further research and development.

Responders want, first and foremost, to know whether a given earthquake could become a crisis situation, but also to then identify potential zones of impact, understand the dominant vulnerable constructions and the scope of potential humanitarian needs (injuries, displaced persons, shelter, food, etc.).

Clearly, the list of such desirables cannot be completely addressed at a global scale. Yet, we highlight some of the additional tools and capabilities that PAGER can offer to aid decision makers and responders. These developments include automatically identifying vulnerable building types in any given area, estimating earthquake-induced damage and loss statistics by building type, and developing visualization aids that help locate areas of concern for improving post-earthquake response efforts. While such information is highly useful and is much desirable, significant improvements are still necessary to portray such uncertain information in practical and informative ways.

31.7.1 Portraying population exposure

Given the significance of spatial variability of ground shaking and its impact on the built environment, it is necessary to highlight the popula-

tion exposure that is subjected to high levels of shaking and hence the most likely areas of concentrated damage and loss. The exposure maps currently produced by the operational PAGER system are unable to provide functionality such as zooming in and out or distinguishing areas of high exposure, infrastructure or zones of importance given the spatial distribution of shaking, which are critical in near real-time response environments.

By rendering ShakeMap and the LandScan population database in a Keyhole Markup Language (KML) format, it is now possible to depict the relative population impacted by different levels of shaking in applications like Google Earth (Jaiswal *et al.*, 2011). Plate VIII (between pages 452 and 453) shows a snapshot of a Google Earth window showing a 3D color-coded bar chart for the 2010 Haiti earthquake. The bar height indicates the affected population and the color indicates the ShakeMap estimated shaking intensity shown using the standard ShakeMap color palette. To avoid a user's misinterpretation of vertical bars as buildings, an appropriate legend is provided to explain the distinction. Aggregate estimates of total population at integer intensity levels (V, VI, VII, etc.) are also produced for the extent of the ShakeMap as shown separately in the panel at the upper left of the map shown in Plate VIII (between pages 452 and 453).

31.7.2 Building stock inventory characterization

To characterize the most vulnerable country-specific building stock in any given earthquake, we are working in collaboration with the Earthquake Engineering Research Institute's (EERI)'s WHE team, to develop a country-specific library of building types, taking existing information from WHE's housing prototype and also extending the coverage further by adding more countries. This library serves multiple purposes but its primary benefits are to: (1) list the building types that are prevalent in a given country, (2) describe certain key aspects of these structure types in terms of their construction or vulnerability to past earthquakes, (3) identify the types that potentially dominate the risk of major losses in a given country, e.g., to help create awareness for long term mitigation, and (4) identify structure types that could pose challenges during post-earthquake rescue and response.

Delivery of content from the library for pre-earthquake mitigation, planning, and education will take the form of country-specific building descriptions and photos, both contributed by USGS/PAGER and EERI/WHE (Fig. 31.5). The library not only helps document the general features of construction practices in different countries but also communicates overall risk; i.e., a combination of structural vulnerability/fragility and occupancy. Prototype webpages have been created for 42 countries via the USGS PAGER and



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PAGER - Common Building Types

New Zealand

Timber Frame Construction

This is a light timber frame construction and is most commonly used for single or multifamily dwellings. It is usually a one or two-story frame with a corrugated iron roof, weatherboard or veneer wall cladding, and a strip or piled foundation. According to the report prepared by Beacon Pathway, the timber frame construction has varied over several decades and each type represents a different era of housing e.g., Villa, Bungalow, Art Deco, pre-1970 (pre-insulation). Timber frame constructions have performed exceedingly well during earthquakes of very strong shaking. Most common damages includes failure of masonry chimney, cracks in stucco or veneer, foundation failure due to liquefaction or excessive ground movement, out-of-plane failure of perimeter walls.

Reference: USGS PAGER Team - Kishor Joshi




Unreinforced Brick Masonry Construction

This is a low to mid-rise bearing wall construction type with a timber or concrete slab floor. Dwellings of this type are generally used as commercial or multifamily residential buildings. Due to usage of weak building material, lack of redundancy, and inadequate lateral load resisting capacity, these types often tend to perform poorly during earthquakes. Laws in New Zealand instated in 1965 require that local governments establish policies for retrofit of earthquake prone buildings, including URM's (EERI 2011). Tens of thousands of unreinforced masonry buildings were severely damaged or collapsed during the recent Christchurch earthquakes of September 2010 and February 2011.

Reference: USGS PAGER Team - Kishor Joshi



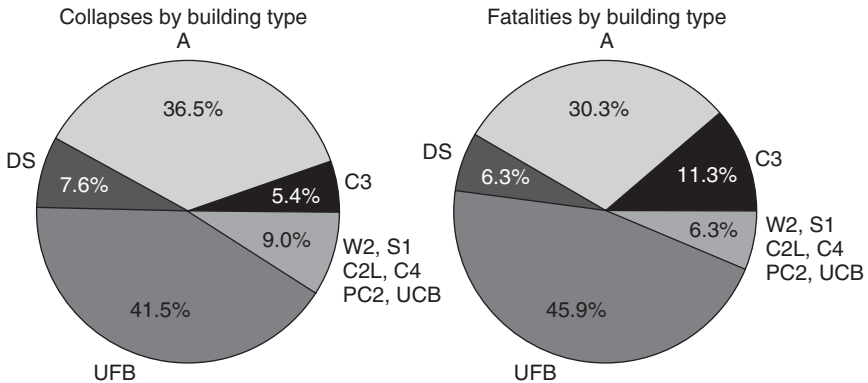
31.5 Country-specific description of common building typology library for PAGER response.

Earthquake Program web pages which are also linked through the EERI/WHE web pages (<http://www.world-housing.net/>).

31.7.3 Damage and loss characterization

In order to understand the demands related to shelter requirements, one needs to estimate the total number of dwellings that might have been severely damaged or collapsed during an earthquake. While the engineering-based damage and loss estimates are not made public from an operational perspective, since they are not equally applicable to all countries, the PAGER system produces a suite of outputs related to fatality and damage estimates from the semi-empirical model.

Figure 31.6 shows one such output automatically produced following the $M_w 7.1$ Van, Eastern Turkey earthquake on October 23, 2011. Figure 31.6 shows that most of the fatalities are expected from the collapse of unreinforced brick masonry bearing wall, adobe, and nonductile reinforced



Key:

A: Adobe, DS: Dressed stone masonry, UFB: Unreinforced fired brick masonry, C3: Nonductile reinforced concrete moment frame with infill, W2: Heavy wood frame, S1: Steel moment resisting frame, C2L: Low rise reinforced concrete shear wall, C4: Nonductile reinforced concrete frame without infill at one or more story, PC2: Precast concrete frame with shear wall, UCB: Unreinforced concrete block masonry.

31.6 Estimated collapse (left) and fatality (right) distribution according to PAGER-STR building typologies produced by the semi-empirical model following the $M_w 7.1$ Van, Turkey, earthquake on October 23, 2011.

concrete frame constructions. The earthquake resulted in 604 fatalities, 4152 injuries and at least 33,016 buildings that sustained serious damage or collapsed (http://www.eeri.org/wp-content/uploads/Van_Turkey_eq-report.pdf).

Early reports indicated that most fatalities were caused due to the collapse of nonductile concrete frame and bearing wall construction as predicted by the semi-empirical model. In order to present the building types that are at risk in a hierarchical order, for each earthquake and for each country, the PAGER team will need to further improve the PAGER building inventory and vulnerability models for those structures. At present, such tools can help in creating awareness about potentially vulnerable buildings in a given area. These tools can potentially also help for planning mitigation exercises, and in aiding emergency response decision-making, including assessment of rescue equipment likely to be required by Urban Search and Rescue (USAR) teams within certain geographic areas.

31.8 Conclusions

Earthquakes have highly variable effects on society; the complex and variable nature of the effects can be attributed to a number of contributing factors, primarily the highly variable nature of the hazard distribution (pre-

dominantly, shaking intensity), the population exposure, the vulnerability of the built environment, and the resilience of the communities affected.

Significant efforts have been made in the past two decades to develop state-of-the-art rapid, earthquake-loss estimation models. These efforts have benefited emergency response and recovery planning operations and generally served as a basis for post-earthquake decision making. For example, faster and improved earthquake source characterization has helped in constraining fundamental earthquake parameters, which in turn improve and speed up shaking estimates. Similarly, evolution of rapid earthquake shaking estimation tools such as ShakeMap, *Did You Feel It?*, and incorporation of near real-time strong motion data, and finite-fault models now makes it possible to characterize spatial aspects of earthquake ground shaking more systematically and efficiently than before.

Given the global variability in the built environment and uncertainty associated with inventory and structural vulnerability data, as well as the knowledge about past casualties in different countries, PAGER relies on three parallel loss estimation approaches. PAGER efforts now focus primarily on further refining each of these three separate loss methodologies and from them, producing alerts based on fatality and economic loss estimates (as well as their uncertainties) for a wide variety of global hazard and risk environments. These data, tools, and models are valuable for other engineering and seismological studies (e.g., performing scenario hazard and loss analyses, developing mitigation priorities), and are openly and freely available. In addition to the primary audience in terms of earthquake response and aid agencies, PAGER's data, products and models are being used in variety of different applications by, e.g., loss-modellers (global Vs30 maps, ShakeMaps, inventories, vulnerabilities), reinsurers (catastrophe bonds), and non-governmental agencies (risk analyses).

31.9 Acknowledgments

The authors thank participants of the WHE-PAGER, and NEHRP funded projects, who contributed to the collaborative development of the inventory and vulnerability data repository. The PAGER team also benefited from collaborations with GEM Foundation partners and colleagues. Munich Re provided economic loss data, which was essential for our economic loss model development effort. Mention of trade names or commercial products does not constitute endorsement or recommendation for use by the US government.

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M 8.8, NEAR THE EAST COAST OF HONSHU, JAPAN

Origin Time: Fri 2011-03-11 05:46:23 UTC (14:46:23 local)

Location: 38.32°N 142.37°E Depth: 24 km

FOR TSUNAMI INFORMATION, SEE: tsunami.noaa.gov

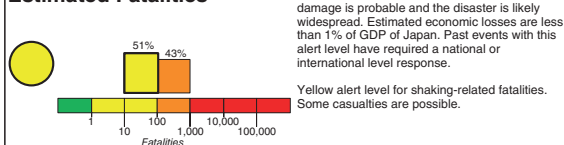


PAGER

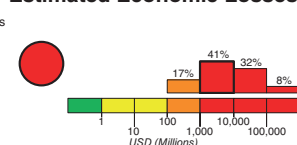
Version 2

Created: 42 minutes, 13 seconds after earthquake

Estimated Fatalities



Estimated Economic Losses

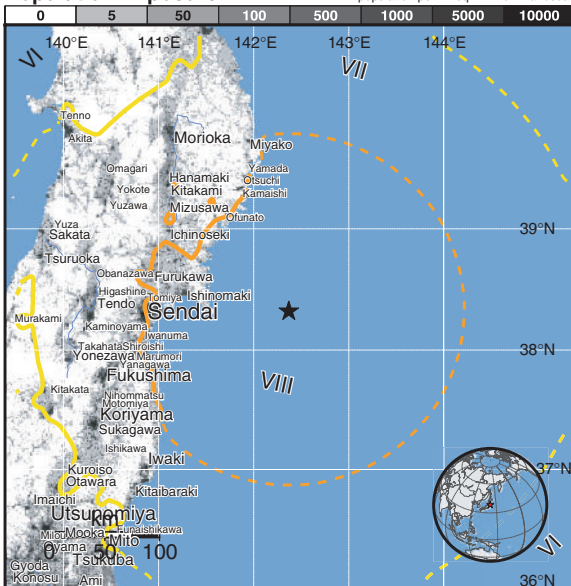


Estimated Population Exposed to Earthquake Shaking

ESTIMATED POPULATION EXPOSURE (k = x1000)	--*	--*	--*	--*	4,099k*	7,081k	1,876k	0	0
ESTIMATED MODIFIED MERCALLI INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+
PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very Strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	Resistant Structures	none	none	none	V. Light	Light	Moderate	Moderate/Heavy	Heavy
	Vulnerable Structures	none	none	none	Light	Moderate	Moderate/Heavy	Heavy	V. Heavy

*Estimated exposure only includes population within the map area.

Population Exposure



Structures:

Overall, the population in this region resides in structures that are resistant to earthquake shaking, though some vulnerable structures exist. The predominant vulnerable building types are ductile reinforced concrete frame and heavy wood frame construction.

Historical Earthquakes (with MMI levels):

Date (UTC)	Dist. (km)	Mag.	Max MMI(#)	Shaking Deaths
1998-06-14	363	5.7	VII(428k)	0
1994-12-28	263	7.7	VII(132k)	3
1983-05-26	369	7.7	VII(174k)	104

Recent earthquakes in this area have caused secondary hazards such as tsunamis, landslides, and fires that might have contributed to losses.

Selected City Exposure

from GeoNames.org

MMI City	Population
VIII Ishinomaki	117k
VIII Yamoto	32k
VIII Shiogama	60k
VIII Kogota	20k
VIII Rifu	35k
VIII Furukawa	76k
VII Sendai	1,038k
VII Yamagata	255k
VII Morioka	295k
VII Fukushima	294k
VI Utsunomiya	450k

bold cities appear on map

(k = x1000)

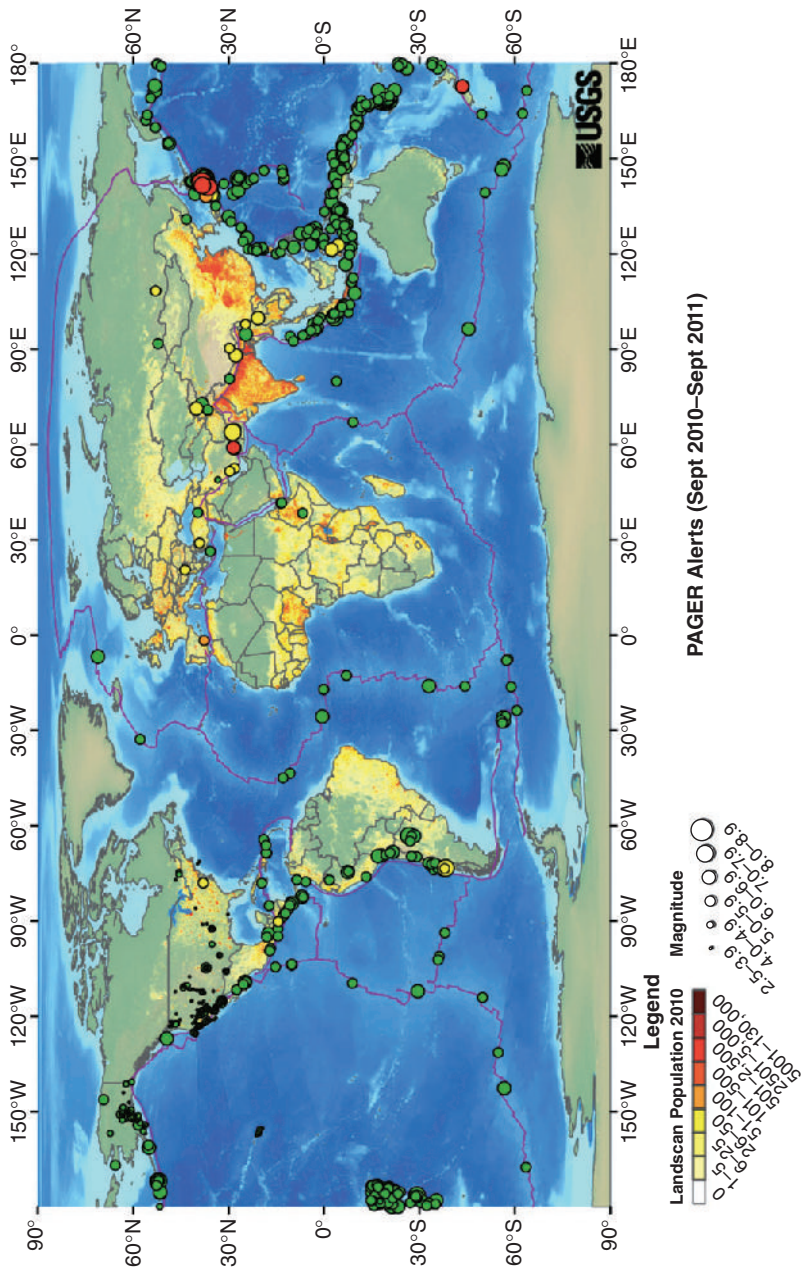
Event ID: usc0001xgp

PAGER content is automatically generated, and only considers losses due to structural damage.

Limitations of input data, shaking estimates, and loss models may add uncertainty.

<http://earthquake.usgs.gov/pager>

Plate VI An automated PAGER impact estimate showing a 'red alert' based on the operational economic and fatality loss model following M_w 9.0 Tohoku earthquake in Japan.



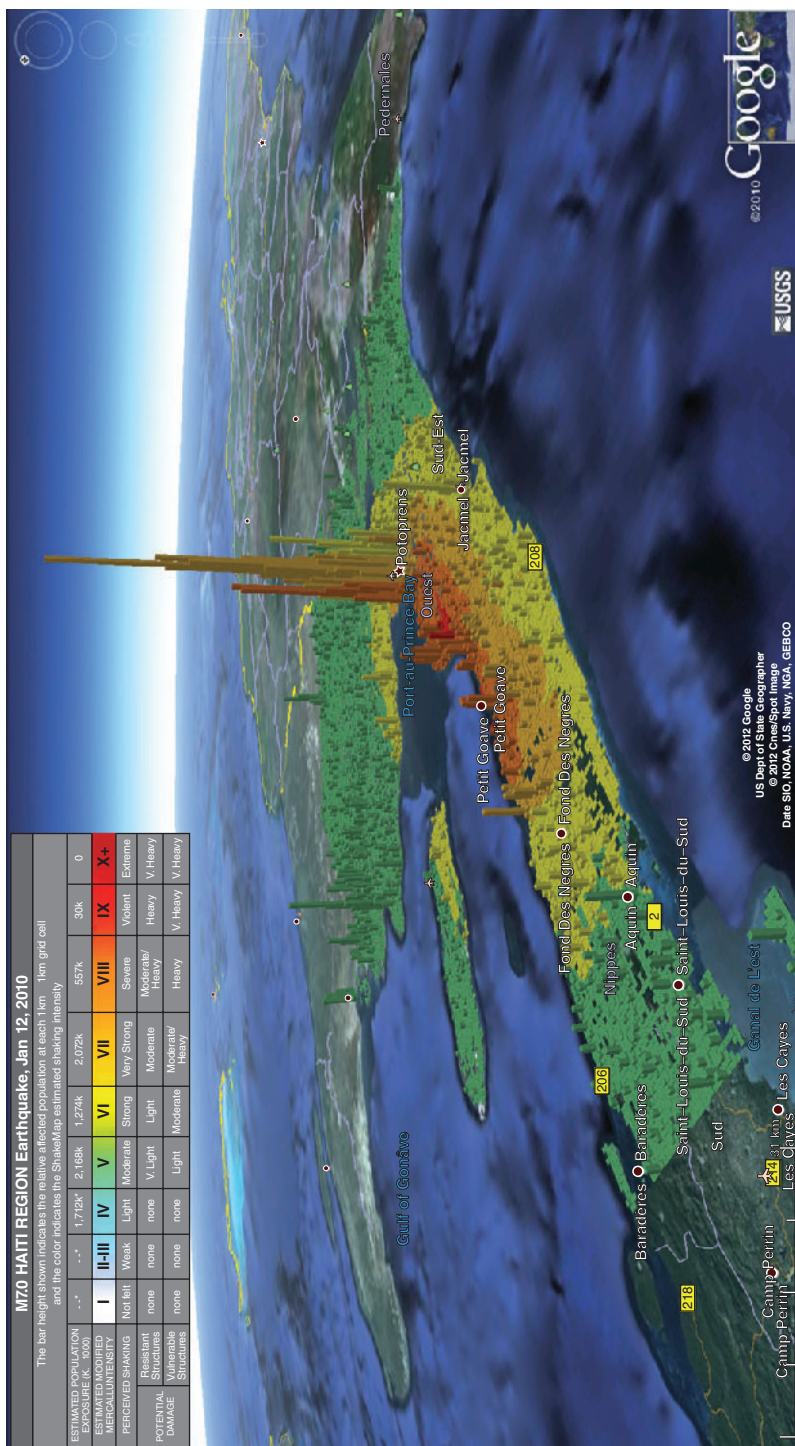


Plate VIII PAGER shaking and exposure estimate overlay on the LandScan population database in Google Earth for the 2010 Haiti earthquake. The bar height indicates the relative population per square km and the color indicates the ShakeMap estimated shaking intensity. (© 2012 Google; US Dept of State Geographer; © Cnes/spot image; Data Sio, NOAA, U.S. Navy, INGA, GEBCO).

-
- AB06 GMPE, 585
 - ABAQUS, 723
 - abutment modeling, 583
 - acceptable risk, 150–4
 - Adaptive Neuro-Fuzzy Inference System (ANFIS), 182
 - advanced earthquake engineering analyses, 17–23
 - AeroDyn, 719
 - Agile framework, 834
 - Akaike Information Criterion, 528, 765
 - aleatory uncertainties, 143
 - Alliance for Global Open Risk Analysis (AGORA), 827–8
 - ALOHA program, 699–700
 - American Concrete Institute Committee, 516
 - American Lifelines Alliance (ALA), 631, 696
 - Amplification* class, 485
 - analytical model, 723–7, 854–5
 - analytical recovery functions, 283
 - Arias intensity, 721
 - artificial neural networks (ANN), 176
 - assistance deficit, 458
 - ASTM E119, 372, 373, 379
 - asynchronous excitation, 614–19
 - higher modes based on measured data, 617–19
 - central section of Evripos cable-stayed bridge, 617
 - observed vs predicted coherency loss, 618
 - seismic code framework, 614–17
 - limiting length to consider spatial variability effects, 616
 - sources of spatial variation of earthquake ground motion, 614
 - attenuation relationships *see* ground motion prediction equation (GMPE)
 - AutoCAD, 663
 - axial load response analysis, 379–80
 - assumed temperature distributions, 380
 - axial-shear-flexure interaction (ASFI), 375
 - Basic Building Taxonomy, 828
 - basic probability assignment, 147–8
 - Bayesian approach, 728
 - Bayesian belief network (BBN), 175–202
 - future trends, 201–2
 - knowledge elicitation from data, 179
 - overview, 175–6
 - performance measure, 179–81
 - kappa statistic calculation, 180
 - metrics for the assessment of classification results, 180
 - two-class prediction outcomes, 181
 - regional damage estimation, 187, 189–94
 - regional seismic risk assessment, 190–4
 - schematic diagram, 177
 - seismic risk analysis, 175–202
 - site-specific hazard assessment, 181–7
 - vulnerability and damage assessment of buildings, 195–201
 - assessment of building damage, 196
 - basic input parameters, states and unconditional probability, 197
 - building damage state classifications, 198
 - building performance modifiers from the walk down survey, 201
 - conditional probability development, 199
 - CPT for building damage, 197
 - damage quantification, 200–1
 - data structure, 198–9
 - intermediate nodes, 201
 - performance modifiers for the building vulnerability, 199
 - seismic hazard and performance modifiers, 200
 - Bayesian Information Criteria (BIC), 528
 - Bayesian network, 494
 - Bayesian statistical framework, 521–2
 - Becker Penetration Test (BPT), 118
 - between-earthquake correlation, 58, 73
 - between-earthquake residuals, 61
 - between-earthquake standard deviation, 62
 - between-earthquake variability, 70, 71, 72
 - bridge foundation soil system, 83–4
 - transverse direction and seismic response parameters, 83
 - bridges
 - effect of soil-structure interaction and spatial variability of ground motion, 597–619
 - earthquake excitation scenarios for interactive soil–bridge systems, 610–19
 - embankment–backfill–abutment–superstructure interaction, 607–9
 - soil–foundation–pier–superstructure interaction, 599–607

- IDA for seismic risk assessment, 561–92
 - case study for continuous 4-span bridge, 580–91
 - fragility curves development using IDA results, 579–80
 - future trends, 591–2
 - record selection, 568–78
 - structural modeling, 564–7
 - uncertainty sources, 567–8
- PBEE approach to estimate structural performance targets, 536–58
 - future trends, 556–8
 - performance-based seismic evaluation framework (PEER approach), 537–42
 - performance-based seismic evaluation of ordinary highway bridges, 546–56
 - probabilistic seismic demand analysis (PSDA), 542–4
 - vector-valued probabilistic seismic hazard assessment (PHSA), 544–5
- Building Census (BC), 473
- building stock inventory
 - characterisation, 860–1
 - country-specific description of common building typology, 861
- buildings
 - deterioration stochastic modeling, 410–32
 - general deterioration process, 411–14
 - reinforced concrete (RC) bridges
 - deterioration, 418–30
 - stochastic semi-analytical approach (SSA), 414–18
 - probabilistic assessment of earthquake insurance rates, 787–811
 - different seismic zones in Turkey, 795–803
 - future trends, 808–11
 - implementation of Turkish Catastrophe Insurance Pool (TCIP), 803–8
 - model, 789–95
 - seismic risk models, 387–406
 - combined progressive and shock-based deterioration, 403–6
 - deterioration approximation, 399–403
 - deterioration mechanism, 388
 - shock-based damage accumulation models, 391–9
 - structural degradation, 389–91
- California Department of Transportation (CALTRANS), 601
- capability approach
 - benefits, 265
 - framework, 258
 - seismic risk analysis and management, 255–66
- capacitated facility location problem (CFLP), 455, 457
- capacity spectrum method (CSM), 352
- capacityPerCapita*, 482
- Cascadia subduction zone, 8
- cast-in-drilled hole (CIDH) pile, 613
- cast-iron-pipes (CIP), 629
- central damage ratio (CDR), 793
- centroid, 456, 478
- civil infrastructure
 - complex interconnected, seismic vulnerability analysis, 465–510
 - application, 499–509
 - future trends, 509
 - model, analysis type and interactions, 468–70
 - object-oriented model (OOM) of infrastructure and hazards, 470–2
 - performance metrics, 488–93
- deterioration stochastic modeling, 410–32
 - general deterioration process, 411–14
 - reinforced concrete (RC) bridges
 - deterioration, 418–30
 - stochastic semi-analytical approach (SSA), 414–18
- earthquake-induced liquefaction risk
 - assessment and management, 113–35
 - hazard identification, 117–21
 - hazard quantification, 121–5
 - overview, 113–16
 - response to liquefaction hazards, 125–33
 - tolerable risks and performance levels, 133–4
- earthquake risk management, 229–53
 - building procurement, 247–50
 - evidence of performance measurement, 245–7
 - fuzziness, incompleteness and randomness (FIR), 238
 - managing technical risks to structures, 230–2
 - process models and project progress maps (PPM), 243–5
 - reliability theory for uncertainty and risk analysis, 232–5
 - risk inevitability, 229–30
 - seismic vulnerability, 235–7
 - systems thinking, 238–43
- GMPE uncertainty in PSHA, 29–54
 - future trends, 51–3
 - GMPE development, 37–45
 - GMPE overview, 30–7
 - model components sensitivity, 45–51
- ground motion selection for seismic risk analysis, 79–109
 - bias in seismic response analysis, 104–5
 - bridge foundation soil system, 83–4
 - future trends, 108–9
 - generalised conditional intensity measure (GCIM), 84–93
 - methodology application, 93–103
 - performance assessment, 80–3
 - seismic demand curve computation, 105–7
 - software implementations, 108
- ground motion spatial correlation in seismic hazard estimation, 57–74
 - evaluation and analysis, 58–69
 - future trends, 74
 - seismic loss assessment, 69–74
- main classes description of building, 472–88
 - 'CriticalFacility' abstract class and sub-classes, 483–4
 - 'InhabitedArea' class, 472–4
 - network abstract class and sub-classes, 475–83
 - 'Seismic class', 484–8
- probabilistic assessment of the model, 493–9
 - available methods, 494–5
 - implemented simulation network, 495–9
 - problem uncertainties, 493–4

- probabilistic seismic hazard analysis, 3–25
 - advanced earthquake engineering analyses, 17–23
 - future trends, 23–5
 - past developments and current trends, 3–7
 - simulation, 7–17
- resilience-based design (RBD) modeling to assess seismic hazards, 268–300
 - case studies, 292–9
 - future trends, 300
 - performance-based design (PBD)
 - development, 270–1
- risk assessment and management, 437–60
 - network hierarchical representation, 442–6
 - network modeling, 440–2
 - optimal resource allocation, 454–9
 - systems and networks, 438–40
- seismic risk analysis and management, 141–70
 - uncertainty in risk analysis, 143–9
- seismic risk models, 387–406
 - combined progressive and shock-based deterioration, 403–6
 - deterioration approximation, 399–403
 - deterioration mechanism, 388
 - shock-based damage accumulation models, 391–9
 - structural degradation, 389–91
- structural vulnerability analysis, 209–27
 - advantages, 226–7
 - risk analysis, 218–23
 - vulnerability, hazard and risk, 210–12
 - vulnerability identification, 212–18
 - vulnerability of infrastructure networks, 223, 225–6
- time, space and stake holder dimensions of the problem, 466–8
 - three dimensions in infrastructure vulnerability study, 468
- clustering, 214–15, 442–3
 - sequences, 216
- CMS-Crustal, 573–4
- CMS-Inslab, 573–4
- CMS-Interface, 573–4
- coefficient of variation (COV), 395–7, 639
- collapse, 344, 352
 - mechanism, 342–54
 - computation of limit lateral capacity of masonry façades, 343
 - FaMIVE procedure flow chart, 345
 - performance drift value for damage limit states, 346
 - prevention, 234
- commercially priced earthquake insurance premium, 794–5
- common connectivity model, 475–8
 - diagram of network class and sub-classes down to EPN and WSS components, 477
 - topological features of network-like systems, 475
- community competence dimension, 276–7
- community resilience
 - seven dimensions, 275–81
 - components and subcomponents of PEOPLES framework, 278–9
 - layer model of PEOPLES, 280
- MCEER centre methodology for RBD
 - based on control (feedback loop) approach, 282
 - resilience scoring using PEOPLES methodology, 281
 - spatial vs temporal scale, 274–5
 - dimension of RBD using PEOPLES approach, 274
- complementary cumulative distribution function (CCDF), 308
- component-level metrics, 489
- component responses, 669–72
 - LADWP system component responses, 670
 - regressions for estimating pipeline damage, 670–2
 - repair rate vs PGV for cast iron and duct iron pipelines, 671
 - repair rate vs PGV for concrete, riveted steel and steel pipelines, 672
- compound Poisson process, 392
- compound process, 392
- computable general equilibrium (CGE) analysis, 676
- compute-Demand* method, 478
- compute-Flow* method, 478
- computed earthquake insurance rates
 - evaluation, 802–3
- conditional mean spectrum (CMS), 571–4, 771
 - CMS using BA08 and Z06 GMPs, 574
 - seismic deaggregation results for Vancouver at $T = 0.7s$, 574
- conditional probabilities, 178–9
- conditional probability table (CPT), 178, 179, 183, 202
- Cone Penetration Test (CPT), 118
- continuous 4-span bridge
 - case study, 580–91
 - abutment modeling, 583
 - bridge properties and modeling assumptions, 580–2
 - record selection criteria and spectral shape effects, 583, 585
 - seismic hazard in Montreal, 585
 - seismic performance and risk assessment, 585–91
 - damage states and developing backbone curves, 582–3
 - deformation at damage states for bridge columns, 583
 - structural modeling of the bridge, 584
- continuum finite element, 600
- copula, 765–7
 - seismic risk management of insurance losses using extreme value theory, 760–84
 - earthquake insurance portfolio analysis, 774–83
 - future trends, 783–4
 - insurer's earthquake risk exposure modeling, 767–74
 - statistical modeling and extreme data, 763–7
- corrosion, 426–9
 - cumulative distribution function for T_{corr} , 428
 - model parameters for T_{corr} prediction, 427
 - steel in concrete, 516–18
 - simplified schematic of chloride-induced corrosion, 517

- cost-benefit analysis (CBA), 160, 162–4
- coupled finite element method/boundary element method (FEM/BEM), 603
- cumulative distribution function (CDF), 91–2, 284, 414, 416, 423–6, 428
- cumulative probability function (CPF), 71
- cybernetics, 438
- cyclic resistance ratio (CRR), 117, 183
- cyclic stress ratio (CSR), 117
- damage analysis, 310–11
- damage consequence index (DCI), 492
- damage limitation, 344, 351, 355–6
- damage measures (DMs), 563
- damage probability matrix (DPM), 236, 336–7, 792
 - earthquake damage estimation, 799–800
 - best estimate MDRs for reinforced concrete buildings, 800
- damage propagation analysis, 452–4
- hierarchy-based correlation function, 453
- damage ratio (DR), 793
- Darcy's law, 479
- decision support system (DSS), 467
- Decree Law 587, 803
- defineStatCompState*, 482
- deformation demand model, 729–30
- demand-driven, 481
- Dempster-Shafer (DS) theory, 147–9
 - DS rule of combination for two decision makers, 149
- dependence modeling, 765–7
- design requirement-related (DR) uncertainty, 567
- deteriorating events, 215–17
- deterioration, 640–1
 - deterioration factors for pipe joints, 641
 - schematic profile of joint deformation capacity, 641
 - seismic reliability of RC bridges, 514–32
 - effects of deterioration on reliability of bridges, 519–31
 - mechanism of deterioration, 515–19
- deterministic demand model, 719–21
- deterministic seismic hazard analysis (DSHA), 81
- Dijkstra algorithm, 450
- directed acyclic graph (DAG), 177, 178
- disaster resilience index, 831, 833
- discretiseEdge* method, 478
- double exponential function, 290
- ductile cast-iron-pipes (DCIP), 629
- dynamic loading, 603–4
- Early Post-Earthquake Damage Assessment Tool (EPEDAT), 841
- earthquake, 268–9
 - low-cycle fatigue damage, 429–30
 - record selection tools, 611–12
 - reinforced concrete building fire performance, 370–9
 - analysis results and observation, 377–9
 - building structure performance with and without seismic damage, 372–3
 - building structure specifications, 372
 - reinforced concrete wall-frame building specimen, 371
 - short column axial deformation capacity response, 378
 - short column axial load capacity response, 378
 - time history of the axial deformation ratio response, 379
 - residual seismic resistance of fire-damaged columns, 379–83
 - stiffness deterioration, 423–6
 - structure seismic response, 369–70
 - structure vulnerability assessment for reinforced concrete, 366–85
- earthquake disaster risk index (EDRI), 192, 194
- Earthquake Engineering Research Institute (EERI), 860
- earthquake hazards
 - pipeline response, 696–9
 - event tree assessing likelihood fatality from thermal exposure, 700
 - fire hazard from leak, 701–2
 - ignition likelihood, 701
 - leak vs rupture, 701
 - relating computed strain to loss of pressure integrity, 701
 - representation of pipe-soil interaction using nonlinear spring elements, 697
- earthquake impact scale, 855–7
- earthquake-induced ground motion, 668–9
- earthquake-induced liquefaction
 - hazard identification, 117–21
 - hazard quantification, 121–5
 - overview, 113–16
 - effect on infrastructure, 115
 - liquefaction risk assessment, 114, 116
 - risk-based approach in designing a civil infrastructure, 116
 - response to liquefaction hazards, 125–33
 - risk assessment and management to civil infrastructure, 113–35
 - tolerable risks and performance levels, 133–4
- earthquake insurance
 - seismic risk management using extreme value theory and copula, 760–84
 - earthquake insurance portfolio analysis, 774–83
 - future trends, 783–4
 - insurer's earthquake risk exposure modeling, 767–74
 - statistical modeling and extreme data, 763–7
- earthquake insurance portfolio analysis, 774–83
 - building portfolios of wood-frame houses in Vancouver, 774–5
 - building groups location, 775
 - reinsurance risk exposure assessment, 782–3
 - ratio of risk measures VaR and TVaR based on statistical models, 783
 - statistical modeling of earthquake risk exposure, 775–82
 - copula density plots comparison using joint POT data, 780
 - Kendall's coefficient of seismic loss data as separate distance function, 781
 - mean express plot and shape parameter plot for group 2, 776

- scatter/plot and marginal distribution plots of joint POT data, 778
- scatter/plot and marginal distribution plots of transformed copula samples, 779
- target original seismic loss samples vs simulated seismic loss samples, 781
- earthquake insurance rates
 - future trends, 808–11
 - implementation of compulsory earthquake insurance scheme and TCIP, 810–11
 - probabilistic assessment, 808–10
 - probabilistic assessment for buildings, 787–811
 - different seismic zones in Turkey, 795–803
 - implementation of Turkish Catastrophe Insurance Pool (TCIP), 803–8
 - model, 789–95
- Earthquake Loss Estimation for the Euro-Med Region (ELER), 840
- earthquake loss models, 844–55
 - analytical model, 854–5
 - empirical model, 845–51
 - economic, 848–51
 - fatality, 846–8
 - semi-empirical model, 851–4
 - grid-based population exposure and hazard information, 853
- earthquake motion tools, 612
- Earthquake Notification Service (ENS), 855
- earthquake risk, 833
- earthquake risk management
 - building procurement, 247–50
 - dome detailed design processes, 252
 - interdependence of BCIOD+R processes, 249
 - MARIUN, 251
 - processes, 248
 - retrofitting process, 250
 - civil infrastructure, 229–53
 - evidence of performance measurement, 245–7
 - Italian flag, 246
 - simple map of three processes, 246
- fuzziness, incompleteness and randomness (FIR), 238
- managing technical risks to structures, 230–2
 - ductility, 231–2
 - foundations, 232
 - layout and form, 231
 - materials, 232
 - stiffness, 232
- process models and project progress maps (PPM), 243–5
- reliability theory for uncertainty and risk analysis, 232–5
- risk inevitability, 229–30
- seismic vulnerability, 235–7
- systems thinking, 238–43
- economic development dimension, 277
- efficient IM, 543
- elastic rebound theory, 790
- elasto-plastic constitutive models, 121
- electric power networks (EPN), 470–1
 - model, 481–3
- embankment–backfill–abutment–superstructure interaction, 607–9
- dynamic mobilisation under seismic excitation, 607–9
- static, 607–8
 - finite element models of TY3 overpass along Egnatia Highway, Greece, 608
 - force displacement (pushover curve), 610
 - plastic strains developed at backfill of TY3 bridge (longitudinal direction), 610
 - plastic strains developed at backfill of TY3 bridge (transverse direction), 609
- Emergency Management Information System, 841
- empirical distribution function (EDF), 91–2
- empirical economic model, 848–51
- empirical ground-motion model *see* ground motion prediction equation (GMPE)
- empirical model, 38–9, 845–51
 - economic, 848–51
 - recorded and estimated economic losses, 851
 - snapshot of Munich re economic loss data for selected events, 850
- fatality, 846–8
 - record and estimated fatalities, 849
- process of partitioning the observed variability to inter-event and intra-event component, 39
- empirical recovery functions, 283
- empirical strong-motion database, 42–3
- engineering demand parameters (EDP), 21, 105–6, 310, 537
- vectors, 316, 319–21
 - generating process, 322
- enhanced system serviceability index (ESSI), 490
- environmental/ecosystem dimension, 276
- EPANET, 663
- epistemic uncertainty, 143, 354–60
- Epsilon-based record selection, 571
- Eurocode 8, 616
- European LessLoss study, 853–4
- European macroseismic scale (EMS), 236
- European Strong-Motion database (ESD), 66
- European Urban Audit (EUA), 473
- exceedance probabilities, 99–103
 - amplitude scale factors vs magnitude distance distribution of ground motion, 101
- deaggregation of the SA(3.0) seismic hazard, 103
- distributions, 100
- mean values of magnitude and source-to-site distance of selected ground motion, 102
- median IM_i values, 100
- excessive demand, 415
- excessive deterioration, 415
- expected annual damage ratio (EADR), 794
- EXPO-CAT catalogue, 848
- extreme value theory, 763–5
 - seismic risk management of insurance losses using copula, 760–84
 - earthquake insurance portfolio analysis, 774–83
 - future trends, 783–4
 - insurer's earthquake risk exposure modeling, 767–74
 - statistical modeling and extreme data, 763–7

- factor of safety, 19
- Failure Mechanism Identification and Vulnerability Evaluation (FaMIVE), 335, 342, 344, 348, 359–60, 362
- failure scenario, 215–17
- failure time analysis, 416–18
- Fatigue, Aerodynamics, Structures and Turbulence (FAST), 719–20
- feasibility, 543
- fictitious links, 444–5
- fictitious networks, 445
- fictitious nodes, 444
- finite difference method, 600
- finite element, 696
 - models, 723
- fire
 - lateral load resistance of fire-damage column using hybrid method, 383
 - reinforced concrete building fire performance following an earthquake, 370–9
 - analysis results and observation, 377–9
 - building structure performance with and without seismic damage, 372–3
 - building structure specifications, 372
 - reinforced concrete wall-frame building specimen, 371
 - short column axial deformation capacity response, 378
 - short column axial load capacity response, 378
 - time history of the axial deformation ratio response, 379
 - residual seismic resistance of fire-damaged columns, 379–83
 - structure response, 368–9
 - lateral deformation of columns due to the thermal expansions, 369
 - structure vulnerability assessment for reinforced concrete, 366–85
- fire-damaged columns
 - lateral load resistance using hybrid method, 383
 - residual seismic resistance, 379–83
- fire resistance test, 368
- first-order reliability method (FORM), 523–4, 568
- first-order-second moment method (FOSM), 568
- Fisher–Tippett–Gnedenko theorem, 763
- flow failure, 123
- Fourier amplitude spectrum, 40
- fragility, 211, 733
 - assessment, 524–5
 - curve, 354
- full-scale model testing, 709–10
 - facility setup for axial soil-pipe restraint testing, 711
 - Physical Modeling Facility layout at UBC and test components, 710
- fuzziness, incompleteness and randomness (FIR), 238
- GA09 model, 770
- gamma process, 400–1
- GEM Nexus, 822
- generalised conditional intensity measure (GCIM), 84–93
 - amplitude-scaled as-recorded ground motions, 90–1
 - distribution realisations, 93–4
 - vector $IM|IM_j$ for the particular site, 94
 - distributions, 88
 - ground motion vs implicit causal parameters, 92–3
 - IM^{sim} realisations, 88–90
 - IMs consideration, 86–7
 - motion representations, 91–2
 - theoretical details, 84–5
- generalised extreme value (GEV), 763
- generalised information theory (GIT), 143–4
- generalised Pareto (GP) model, 762
- geographic information system (GIS)
 - technology, 841
- GH08 model, 770
- GIS, 663
- Global Active Fault, 824–5
 - faulted Earth tool for capturing data on faults collected by geologists, 825
- Global Component projects, 821
- Global Disaster Alert and Coordination System (GDACS), 840
- Global Earthquake Consequences Database, 830–1
- Global Earthquake History, 823–4
- Global Earthquake Model (GEM)
 - assessing global earthquake risks, 815–37
 - global earthquake risk modeling, 818–19
 - mega-cities, seismic hazards and recent losses, 816
 - OpenQuake, 833–7
 - outlook, 837
 - collaborative development, 820–2
 - GEM's development infrastructure, 821
 - current status, 822–33
 - scientific modules, 819–20
 - schematic diagram, 820
- global earthquake risks
 - Global Earthquake Model (GEM) initiative, 815–37
 - current status, 822–33
 - OpenQuake, 833–7
 - outlook, 837
- Global Exposure Database (GED), 828–9
- Global Geodetic Strain Rate Model, 825–7
 - grid representing the second invariant of strain rate tensor, 826
- global indices, 441
- Global Instrumental Catalogue, 824
- Global Large Historical Earthquake Catalogue (GLHECAT), 823
- Global Physical Vulnerability Estimation
 - Methods, 829–30
 - discrete vulnerability function and uncertainty description, 829
- Global Rural-Urban Mapping Project (GRUMP), 852–3
- Google Earth, 860
- graceful degradation *see* progressive degradation
- graph theory, 469
- Graphical Iterative Response Analysis of Flow Following Earthquakes (GIRAFFE), 673

- gross domestic product (GDP), 848–9
- ground failure, 17
- ground motion
 - effect of soil-structure interaction and spatial variability, 597–619
 - earthquake excitation scenarios for interactive soil-bridge systems, 610–19
 - embankment–backfill–abutment–superstructure interaction, 607–9
 - soil–foundation–pier–superstructure interaction, 599–607
- ground-motion attenuation equations, 65
- ground motion correlation
 - evaluation and analysis, 58–69
 - correlation models estimation from empirical data, 64–9
 - definitions, 58–9
 - distribution of strong-motion residuals, 65
 - loss probability distribution parameters, 67
 - parameters numerical simulation, 64
 - procedure of analysis, 59–62
 - site-to-site (spatial) correlation functions, 68
 - spatial correlation models, 63
 - future trends, 74
 - seismic hazard estimation to civil infrastructure, 57–74
 - seismic loss assessment, 69–74
- ground-motion model *see* ground motion prediction equation (GMPE)
- ground motion parameter, 58, 72
- numerical simulation, 64
- ground motion prediction equation (GMPE), 4, 8–9, 11, 14, 15–17, 24, 81, 85, 109, 485, 539, 569
- development, 37–45
 - compilation of empirical strong-motion databases, 42–3
 - generic types, 38–42
 - mixed-effects regression analysis, 43–5
- future trends, 51–3
- ergodic assumption, 51–2
- extended ranges of applicability, 53
- semi-empirical approaches, 52–3
- model components sensitivity, 45–51
- magnitude uncertainty during derivation of a prediction equation for Arias intensity, 49
- model-specific epistemic uncertainty, 46
- prediction intervals and parameter uncertainty, 49–51
- uncertainty in independent variables, 47–9
- overview, 30–7
 - role within seismic hazard and risk applications, 31–2
 - types, 32–7
- PSHA of civil infrastructure, 29–54
- ground motion selection
 - bias in seismic response analysis, 104–5
 - bridge foundation soil system, 83–4
 - future trends, 108–9
 - generalised conditional intensity measure (GCIM), 84–93
 - methodology application, 93–103
 - exceedance probabilities, 99–103
 - GCIM distribution realisations, 93–4
 - weight vector effect, 94–9
- record, 315
- seismic demand curve computation, 105–7
- seismic performance assessment, 80–3
- seismic risk analysis of civil infrastructure, 79–109
 - software implementations, 108
- Gumbel copula, 766
- Gutenberg–Richter law, 791–2
- Gutenberg–Richter model, 486
- hard risks, 229–53
- hard system model uncertainty, 238, 240
- hard system parameter uncertainty, 238, 240
- harmonically over-damped recovery function, 285
- hazard, 211
- hazard function, 393
- hazard impact index (HII), 262
- hazard rate-based stochastic model, 393–4
 - systems abandoned failure probability, 394
- Hazus software, 696, 841, 854
- head-driven, 481
- heat penetration, 376
 - column temperature distribution, 377
- heavy right tail (HRT) copula, 766
- hierarchy-based optimisation, 455–7
- holon, 439
- H2ONET, 663
- Hurwicz criterion, 168–9
 - linguistic degree of optimism to *w*-index values, 169
- hybrid method, 383
- immediate occupancy (IO), 234
- implemented simulation network, 495–9
 - graphical representation of typical simulation run, 498
 - simple infrastructure made up of two interconnected systems, 497
- ImportanceSamplingKmeans*, 495
- imprecise (interval) probability, 145
- incremental dynamic analysis (IDA), 563–4
 - applied to seismic risk assessment of bridges, 561–92
 - case study for continuous 4-span bridge, 580–91
 - future trends, 591–2
 - uncertainty sources, 567–8
- damage measures (DMs) and IMs, 563
- fragility curves development using IDA results, 579–80
- different limit states, 579
- IDA curves, 563
- record selection, 568–78
 - CMS, 571–4
 - spectral shapes of the records and epsilon values, 568–9
 - strategies, 568–71
- record selection influence on IDA result, 574–8
 - bridge properties, 575
 - IDA results at $T_1 = 0.7$ s, 576
 - summary of IDA results for different damage states at $T_1 = 0.7$ s, 577
- structural modeling, 564–7
 - backbone curve parameters, 565

- summarising results, 563–4
 - IDA curves developed using 44 crustal records for 4-span bridge, 565
- individual risk, 151–2
 - fatality accident rate for various activities, 151
 - policy factor value as function of voluntariness and benefit, 152
- infrastructure-level metrics, 493
- infrastructure system, 500–1
- InhabitedArea class, 472–4
 - polygons on different layer indicate the geographical units, 474
 - super-graph comprising all systems with their dependency edges, Plate I
- input–output (I–O) analysis, 676
- installation/operation cost, 458
- instantaneous failure rate, 394–5
- Institute of Engineering Seismology and Earthquake Engineering (ITSAK), 618
- insurance premium rates
 - RC buildings located in different seismic zones in Turkey, 800–2
 - best estimate and currently charged earthquake insurance rates, 801
 - contribution ratios of each intensity level, 801
 - variation of commercially priced earthquake insurance, 802
- insurer's earthquake risk exposure modeling, 767–74
 - seismic hazard analysis and generation of seismic intensities, 767–70
 - seismic loss estimation framework, 768
 - vulnerability and assessment loss of wood-frame houses, 770–4
 - damage-loss functions for different occupancy types, 773
 - incremental dynamic analysis curves of four UBC-SAWS models, 772
 - mean yield and collapse maximum inter-story drift capacity ratios, 773
- Integrated System for Structural Analysis and Record Selection (ISSARS), 612
- intensity measure, 84–5, 87–92, 93–6, 99–101, 104, 106, 537
 - ground motion selection consideration, 86–7
 - empirical correlation equations, 87
- inter-cluster relationships, 445
- inter-level relationships, 446
- International Seismological Centre, 824
- intra-cluster relationships, 446
- Inventory Data Capture Tools (IDCT), 831
 - workflow diagram, 832
- Italian Accelerometric Archive (ITACA), 66
- Japan Meteorological Agency, 744
- Japan Water Works Association (JWWA), 631
- JONSWAP/Pierson-Moskowitz spectrum, 720
- JUSTISE, 247, 249
- K-NET/KIK-NET database, 575, 770
- Kaimal model, 720
- Kappa statistic, 179
- Kendall's coefficient, 779–80
- Keyhole Markup Language (KML), 860
- Kolmogorov–Smirnov (KS) test statistic, 90
- Land Use Plan (LUP), 473
- LandScan population database, 860
- Laplace criterion, 167
- lateral load resistance, 380–2
 - axial load and deformation ratio relations, 381
 - fire-damage column using hybrid method, 383
 - displacement response of a six-storey building, 385
 - hybrid fire testing (HFT) method, 384
 - lateral load and lateral deformation ratio response analysis, 382
- lateral spreading, 122–3
- Latin hypercube sampling, 723
- life-cycle costing (LCC), 160, 162–4
 - seismic design level, 163
- life quality index (LQI), 153–4
- life safety (LS), 234
- liquefaction hazard
 - identification, 117–21
 - deterministic vs probabilistic approaches, 119–20
 - laboratory testing, 120
 - numerical modeling, 120–1
 - infrastructure response, 125–33
 - assessment methods, 131–3
 - bridge deck collapse, 129
 - buildings, 127, 129, 131
 - floatation of underground structures, 130
 - foundation damage, 132
 - lateral spreading damage to roads and utilities, 128
 - linear systems, 127
 - port structures and warehouses damage, 131
 - ports and harbours, 127
 - sand ejecta, 126
 - quantification, 121–5
 - liquefaction hazard curve, 19
 - liquefaction potential, 117–19
 - simplified approach for soil assessment, 118
- local indices, 441
- localisation model, 486
- lognormal probability density recovery function, 288
- long-term recovery models, 283
- Los Angeles Department of Water and Power (LADWP), 662
 - overview of water supply system, Plate II
- loss analysis, 310–11
- low-cycle fatigue damage, 429–30
 - $\Delta\Delta f_v$ and Y_m data, 429
- low-probability-high-consequence (LPHC), 272
- marginal distribution modeling, 763–5
- MARIUN, 245, 249
- Markov processes, 392–3
- masonry buildings
 - seismic vulnerability assessment, 334–62
 - collapse-mechanism approach, 342–54
 - empirical and analytical approaches, 336–42
 - stochastic and epistemic uncertainty quantification, 354–60

- materials degradation
 - mechanical properties, 373–6
 - column damage after shaking table test, 374
 - column lateral load-drift response, 375
- MATLAB, 108, 470, 612, 847
- matrix system reliability method (MSRM), 494
- maximax criterion, 167
- maximum considered earthquake (MCE), 580, 632
- mean time to failure (MTTF), 396–7, 404–5
- metamodel, 290
- Metropolitan Water District (MWD), 663
- minimax regret criterion, 167–8
 - regret matrix, 168
- mitigation approaches
 - reducing risk to pipelines, 702–5
 - ground densification using vibro-displacement at river bank in Vancouver, 704
- mixed-effects regression analysis, 43–5
- model correction, 721–3
 - explanatory functions for demand models, 722
- Model Facility, 821–2, 833
- modeling (MDL) uncertainty, 568
- modern portfolio theory (MPT), 272
- modified compression field theory (MCFT), 375
- modified maximum likelihood (MML) method, 792
- modified Mercalli intensities (MMI), 842
- Mohr–Coulomb plasticity model, 726
- moment demand model, 731–2
- Monte Carlo simulation, 4, 7, 8, 12, 495, 568, 673, 770
- Morison's equation, 720
- MR-based record selection, 570–1
- multi-event model, 744–7
- Multidisciplinary Centre for Earthquake Engineering Research and American Technology Council, 601
- multiple criteria decision making (MCDM), 161–2, 164–9
 - decision matrix for the five retrofit alternatives, 166
 - eight evaluation criteria description, 165
 - normalised payoffs values for five retrofit alternatives, 166
- National Building Code of Canada, 158, 570, 767
- National Earthquake Information Centre (NEIC), 842
- National Institute of Building Sciences and Federal Emergency Management Agency, 841
- Natural Hazards Risk Markup Language, 836–7
- near collapse, 344, 352
- Nelder–Mead optimisation, 847
- network
 - hierarchical representation, 442–6
 - model, 444–6
 - model elements, 445
 - modeling, 440–2
- network abstract class
 - sub-classes, 475–83
 - common connectivity model, 475–8
 - WSS model, 478–81
- Network of Research Infrastructures for European Seismology (NERIES), 840
- New Generation Attenuation (NGA) model, 65, 68
- Newmark approach, 693
- numerical modeling, 708
- numerical simulations, 652–4
 - profiles of water-supply level, 653
 - schematic diagram of three seismic investment schemes, 653
- setup, 652
- object-oriented model (OOM)
 - infrastructure and hazards, 470–2
 - high-level class diagram, 471
 - modeling interactions, 472
- Oil and Gas transmission and distribution network, 470–1
- ontology, 827–8
- OpenQuake, 833–7
 - various typical outputs of OpenQuake calculator, 835–6
- OpenSees, 525
- OpenSHA, 108–9, 423
- optimisation-based decision-making, 454–5
- ordinary highway bridges
 - performance-based seismic evaluation, 546–56
 - demand-time histories of nonlinear dynamic analysis, 548
 - distribution of type 2-EDP vs scale factor for separate-scale-factor case, 549
 - PBEE framework for evaluating the vertical ground motion effects, 546
 - type 2-EDP vs scale factor for one-scale-factor case, 549
- organised governmental services dimension, 276
- P - Δ effects, 376–7
- Pacific Earthquake Engineering Research (PEER) Centre, 827
- parameterless scale intensity (PSI), 337
- peak ground acceleration (PGA), 3, 6, 17, 18, 336, 337, 744
- peak ground velocity (PGV), 3, 502, 669, 744
- peak-over-threshold (POT) method, 763
- peak velocity amplitudes, 68
- Pearson's linear correlation coefficient, 779–80
- PEER-NGA database, 575, 612, 724–5, 770
- PEOPLES approach, 275–6
- performance-based design (PBD), 270–1
- performance-based earthquake engineering (PBEE), 6, 270, 271, 308–21
 - annual rate of exceeding a threshold value, 312–13
 - derivation, 309–12
 - schematic diagram, 309
 - EDP vectors, 319–21
 - framework, 308–21
 - high-rise buildings seismic performance assessment, 321–31

- implementation, 313–19
 - EDP matrix, 316
 - fragility curve, 314
 - repair quantity table sample, 315
 - structural and nonstructural components, 313–15
- seismic risk assessment for new and existing building, 307–32
- structural performance targets estimates for bridges, 536–58
 - future trends, 556–8
 - performance-based seismic evaluation framework (PEER approach), 537–42
 - performance-based seismic evaluation of ordinary highway bridges, 546–56
 - probabilistic seismic demand analysis (PSDA), 542–4
 - vector-valued probabilistic seismic hazard assessment (PHSA), 544–5
- performance-based engineering (PBE), 233, 234
- performance-based seismic evaluation framework (PEER approach), 537–42
 - damage measure, 541–2
 - earthquake engineering framework, 538
 - engineering demand parameter, 540–1
 - PSHA and ground motion intensity measures, 539–40
- performance metrics, 488–93
 - component-level, 489
 - infrastructure-level, 493
 - probabilistic, 492–3
 - system-level, 489–92
 - triangular network and evaluation of simple and weighted connectivity loss, 491
- PERIMETA, 247
- permanent ground deformation (PGD), 121–2, 124–5, 668–91
 - five-point qualitative scale of likelihood, 125
 - ground movement at fault crossings, 690
 - ground movement due to landslides, 691
 - liquefaction-induced ground movement, 690
 - risk-based assessment of liquefaction occurrence, 125
- physical infrastructure dimension, 276
- Pickands–Balkema–de Haan theorem, 762, 764
- Pipeline Research Council International (PRCI), 707
- pipelines
 - challenges and issues, 705–8
 - acceptable risk, 706–7
 - older pipelines conditions, 705–6
 - pipeline strain vs rupture size, 705
 - risk-based code provisions, 707–8
 - future trends, 708–11
 - analytical approaches to capture nonlinear material behaviour, 708–9
 - field modeling and testing of pipeline performance, 710–11
 - full-scale model testing, 709–10
 - numerical modeling, 708
 - geotechnical strains, 705
 - uncertainties with quantifying lateral spread displacement hazards, 706
 - oil and gas, seismic risk assessment in, 682–712
 - consequences of pipeline damage, 699–702
 - determining hazard likelihood, 691–2
 - determining severity hazard, 692–6
 - key steps, 685–8
 - mitigation approaches to reduce risk to pipelines, 702–5
 - pipeline response to earthquake hazards, 696–9
 - purpose, 683–5
 - seismic hazard types, 688–91
- Poisson distribution, 479
- Poisson model, 796
- Poisson process, 392, 395
- Poisson's ratio, 602
- population and demographics dimension, 276
- population exposure, 859–60
- possibility theory, 146–7
 - probability measure based on the possibility function, 147
- possible maximum loss (PML) analysis
 - seismic risk of reinforced concrete structures, 741–58
 - analytical procedure for assessing seismic risk, 743–7
 - case studies, 747–57
 - future trends, 757–8
- potential earthquake damage, 792–4
- powerflow, 482
- probabilistic assessment
 - earthquake insurance rates for buildings, 787–811
 - different seismic zones in Turkey, 795–803
 - future trends, 808–11
 - implementation of Turkish Catastrophe Insurance Pool (TCIP), 803–8
 - model, 789–95
- probabilistic demand models, 718–19, 727–32
 - Bayesian updating, 728
 - deformation demand model, 729–30
 - measured vs predicted deformation demands, 730
 - posterior statistics of parameters, 729
 - model selection, 728
 - moment demand model, 731–2
 - measured vs predicted moment demands, 732
 - posterior statistics of parameters, 732
 - shear demand model, 730–1
 - measured vs predicted deformation demands, 731
 - posterior statistics of parameters, 731
- probabilistic liquefaction hazard analysis (PLHA), 6, 17–21, 24–5
 - cyclic stress ratio vs cyclic resistance ratio, 20
- probabilistic metrics, 492–3
- probabilistic model, 789–95
 - commercially priced earthquake insurance premium, 794–5
 - computation flowchart, 796
 - formulation, 520–1
 - potential earthquake damage estimation, 792–4
 - damage probability matrix (DPM), 793
 - pure risk premium, 794
 - seismic hazard analysis, 790–2
- probabilistic seismic demand analysis (PSDA), 542–4

- probabilistic seismic demand model (PSDM), 541
 - scalar IMs, 550–2
 - distribution of type 1-EDP vs transverse and longitudinal IMs, 551
 - distribution of type 2-EDP vs transverse and longitudinal IMs, 551
 - scalar PHSA for structural demand, 552–3
 - stage 1 for near fault site for type 1-EDP, 552
 - stage 1 for near fault site for type 2-EDP, 553
 - vector-valued PSDM and vector-valued PSHA, 553–6
 - distribution of type 1-VF and horizontal and vertical spectral acceleration, 554
 - distribution of type 2-VF and horizontal and vertical spectral acceleration, 555
 - structural demand hazard curves at stage 2 for near fault size for type 1-EDP, 556
 - structural demand hazard curves at stage 2 for near fault size for type 2-EDP, 557
- probabilistic seismic hazard analysis (PSHA), 119–20, 539–40, 664, 795–9
 - advanced earthquake engineering analyses, 17–23
 - civil infrastructure, 3–25
 - future trends, 23–5
 - ground-motion prediction equation (GMPE)
 - of civil infrastructure, 29–54
 - development, 37–45
 - future trends, 51–3
 - model components sensitivity, 45–51
 - overview, 30–7
 - location of seismic source zones defined for Turkey, 798
 - past developments and current trends, 3–7
 - general procedure and extension to advanced earthquake, 5
 - seismicity map of Turkey, Plate V
 - simulation, 7–17
 - current issues, 15–17
 - GMPE comparison for western Canada, 16
 - ground motion parameters using synthetic earthquake catalogue and multiple GMPEs, 11
 - Gutenberg–Richter model and finite-fault model, 10
 - methodology, 7–13
 - procedure, 13–15
 - seismic activities and delineated regional seismic zones in Canada, 9
 - uniform hazard spectra and seismic deaggregation construction, 12
 - subjective probabilities assigned to alternative assumptions, 797
- probabilistic seismic risk analysis (PSRA), 6, 21–3, 24–5
 - incremental dynamic analysis and seismic risk curve, 22
- probability density functions (PDF), 71, 285
- probability distribution, 416–17
- process models, 243–5
 - responsibility and accountability, 244
- progressive degradation, 390, 399–403, 406
 - approximation, 400
 - combined shock-based degradation
 - basic formulation, 403–4
 - combined damage results, 404
 - RV model parameter, 404
 - combined shock-based deterioration, 403–6
 - gamma process, 400–1
 - models comparison, 402–3
 - gamma model, 402
 - trend-dependent model, 403
 - RC structures, 399–400
 - trend-dependent, 401–2
- project progress maps (PPM), 243–5
- Prompt Assessment of Global Earthquakes for Response (PAGER) system
 - damage and loss characterisation, 861–2
 - estimated collapse and fatality distribution, 862
 - development, 842–4
 - modular system, 843
 - loss estimation for recent earthquakes, 857–9
 - economic loss estimates for recent worldwide earthquakes, 858
 - fatality-based impact alerts between September 2010 to September 2011, Plate VII
 - products and ongoing developments, 859–62
 - building stock inventory characterisation, 860–1
 - portraying population exposure, 859–60
 - shaking and exposure estimate overlay on the LandScan population database for the 2010 Haiti earthquake, Plate VIII
- rapid global earthquake impact estimation
 - strategies, 839–63
 - earthquake impact scale, 855–7
 - earthquake loss models, 844–55
 - state-of-the-art of rapid earthquake loss estimation system, 840–2
 - ‘red alert’ based on operational economic and fatality loss model, Plate VI
- prototype buildings, 322–5
 - damage states and repair costs for each performance groups, 326
 - fragility curves, 325
 - isotropic and plan view, 322
 - modal properties comparison, 324
 - performance group assessment, 324
- public policy, 264–5
- pure risk premium, 794
- qualitative risk analysis, 143, 150
- quantile-quantile (Q-Q) plot, 765
 - seismic loss data, 777
- quantitative risk analysis, 143, 150
- railway network (RWN), 470–1
- Railway Technical Research Institute (RTRI), 748
- railway viaduct, 747–50
 - annual expected loss ED for two viaduct types, 749
 - seismic loss function, 749
 - seismic risk analysis procedure, 748
 - seismic risk curves, 750
- random effects approach, 44
- random realisations (IM^{sim}), 88–90
- rapidity, 275

- record selection strategies, 568–71
 - CMS-based record selection, 571
 - Epsilon-based record selection, 571
 - MR-based record selection, 570–1
 - uniform hazard spectrum (UHS)-based approach, 570
- record-to-record (RTR) uncertainty, 567
- redundancy, 275
- regional programmes, 821
- regional seismic risk assessment, 190–4
 - data transformation, 190–2
 - earthquake disaster risk index, 190, 194
 - input parameters, 192
 - sample analysis weights, 191
 - transformed basic input parameters, 193
 - WAM and BBN normalised EDRI, 194
- regional seismicity model, 502–3
- reinforced concrete (RC)
 - structure vulnerability assessment for fire and earthquake risk, 366–85
 - building fire performance, 370–9
 - future trends, 383–5
 - lateral load resistance using hybrid method, 383
 - residual seismic resistance, 379–83
 - structure response to fire, 368–9
 - structure seismic response, 369–70
- reinforced concrete (RC) bridges, 418–30
 - analysis results and discussion, 430
 - probability distribution for n_F , 431
 - probability distribution for t_F , 432
 - case study, 419–21
 - bent global structural properties, 420
 - column structural properties, 419
 - pushover curve for bent column, 420
 - typical bridge bent, 419
 - corrosion effect, 426–9
 - deterioration effects on seismic reliability, 523–4
 - fragility assessment, 524–5
 - deterioration effects on structural capacity and seismic demand, 520–3
 - configuration, 522
 - model assessment using Bayesian statistical approach, 521–2
 - probabilistic models formulation, 520–1
 - variables for pristine single-bent bridge, 522
 - variation over time of deformation and shear capacities and demands, 524
- deterioration mechanism, 515–19
 - ASR, 518
 - corrosion of steel in concrete, 516–18
 - service life structure, 518–19
 - effects of deterioration on reliability of bridges, 519–31
- low-cycle fatigue damage due to earthquake, 429–30
- model updating based on data from field inspections, 529–31
 - definition of equivalent time, 530
- reliability of an RC bridge subject to deterioration, 525–7
 - deformation, deformation and shear failure mode and shear failure mode, 526
 - deformation, shear and deformation and shear failure mode, 527
 - seismic deformation demand and capacity, 421–3
 - seismic hazard curves for San Francisco, CA., 422
 - seismic fragility increment functions, 527–9
 - fragility estimates for the RC bridge, 529
 - seismic reliability, 514–32
 - stiffness deterioration due to earthquakes, 423–6
 - CDF comparison, 425
 - Δ_s/H and T_N/T_N , 424
 - observed *versus* predicted values of $\ln(Z_m)$, 426
- reinforced concrete (RC) buildings
 - comparative risk assessment with different capacities, 756–7
 - relationship between loss ratio and Q_u/Q_{uns} , 757
 - various cities in Japan, 754–6
 - comparison of various risk indices based on recurrence time of 475 years, 756
- reinforced concrete (RC) piers
 - highway bridge, 750–4
 - flow diagram of seismic risk assessment, 752
 - relationship between PML475 and seismic capacity-demand index, 754
 - seismic risk curves for NEL and PML, 753
 - structural specifications, 751
- reinforced concrete (RC) structures
 - case studies, 747–57
 - buildings in various cities in Japan, 754–6
 - comparative risk assessment of buildings with different capacities, 756–7
 - piers of highway bridge, 750–4
 - railway viaduct, 747–50
 - seismic risk and possible maximum loss (PML) analysis, 741–58
 - analytical procedure for assessing seismic risk, 743–7
 - future trends, 757–8
 - reliability assessment, 447–9
 - methodology, 448
 - reliability theory, 232–5
 - residual drifts, 376–7
 - residual seismic resistance
 - fire-damaged columns, 379–83
 - strength recovery, 382–3
- resilience, 273–4
 - contour map, 277
 - schematic diagram, 274
- resilience-based design (RBD)
 - case studies, 292–9
 - buildings in treasure island in San Francisco bay, 298
 - discrete probability damage state for buildings, 299
 - old medieval housing units of L'Aquila downtown, 294–5
 - resilience vs functionality for different scenario events, 297
 - selected housing units in L'Aquila downtown and road network, 294
 - software implementing the MCEER methodology, 296
 - treasure island in San Francisco bay, 295–9

- communicating risk, 290–1
- future trends, 300
- modeling of civil infrastructure to assess seismic hazards, 268–300
- performance-based design (PBD)
 - development, 270–1
- recovery models, 281, 283–90
 - harmonically over-damped recovery function, 287
 - lognormal cumulative distribution recovery function, 286
 - lognormal probability density recovery function, 289
 - Rayleigh probability density recovery function, 288
 - uniform cumulative distribution recovery function, 284
- resilience, 273–4
- resilience performance levels, 291–2
- seven dimensions of community resilience, 275–81
- spatial vs temporal scale of community resilience, 274–5
- structures, 273
- uncertainties, 290
- resilience performance levels (RPL), 291–2
 - recommended seismic performance objectives for, 292
 - survey results on the acceptable recovery process, 293
 - three-dimensional resilience performance objectives matrix, 293
- resource allocation, 264–5
- resourcefulness, 275
- response analysis, 310–11
- REXEL, 612
- risk acceptability, 263–4
- risk analysis, 218–23
 - civil infrastructure networks, 437–60
 - network hierarchical representation, 442–6
 - network modeling, 440–2
 - optimal resource allocation, 454–9
 - hierarchical decomposition second and fourth level, 459
 - sample, 457–9
 - propagation hierarchy for evidence for a failure scenario, 219
- sample, 221–3
 - assessment of structural risk for sample 2, 224
 - failure scenarios for the structure in sample 2, 222
 - structure in sample 2 and the corresponding clustering hierarchy, 222
 - systems and networks, 438–40
- risk calculation, 220–1
 - structural risk as function of vulnerability, 221
- risk communication, 154–5
- risk determination, 259–62
 - choosing appropriate capabilities, 260
 - choosing appropriate indicators, 260–2
 - scaling and combining indicators, 262
- risk management
 - civil infrastructure networks, 437–60
 - network hierarchical representation, 442–6
 - network modeling, 440–2
 - optimal resource allocation, 454–9
 - generation of disasters strategy and designed networks, 460
 - hierarchical decomposition second and fourth level, 459
 - sample, 457–9
 - systems and networks, 438–40
- risk measurement, 259–62
 - choosing appropriate capabilities, 260
 - choosing appropriate indicators, 260–2
 - scaling and combining indicators, 262
- risk perception, 150
- risk tolerability, 263–4
- risk transfer, 155–8
 - wealth process subject to catastrophic earthquake risk, 156
- road networks (RDN), 470–1
- robustness, 211, 275
- Ruaumoko program, 581
- SAFIR, 372
- SAP2000, 612
- scaling robustness, 543
- scientific modules, 819–20
- second-order reliability method (SORM), 523–4, 568
- Seismic Analysis of Wood-frame Structures (SAWS), 771
- Seismic class, 484–8
 - sample simulation of primary IM (PGA) for magnitude 7 event, 487
- seismic code framework, 614–17
- seismic deformation capacity, 421–3
- seismic deformation demand, 421–3
- seismic demand curve, 105–7
 - computation using various conditioning intensity measures, 107
 - uncertainty comparison for annual exceedance probability, 107
- Seismic Design for Building Structure, 271
- seismic failure
 - lifeline system, 627–41
 - deterioration effects, 640–1
 - seismic performance of pipelines, 631–40
 - seismic damage to lifeline system, 627–31
 - distribution network in service district of water supply system, 628
 - old and new types of CIP and DCIP joints, 630
 - pipe materials used distribution area of water supply system, 629
 - restraining mechanism of new type DCIP joint, 631
 - seismic performance old and new types of joints, 630
 - transmission network of water supply, 628
- seismic fragility model, 499–500
- Seismic Guidelines for Water Pipelines, 632
- seismic hazard analysis, 309–10, 315, 790–2
- seismic hazards, 31–2, 664–9
 - civil infrastructure resilience-based design (RBD) modeling, 268–300
 - case studies, 292–9
 - future trends, 300
 - performance-based design (PBD)
 - development, 270–1

- determining severity, 692–6
 - displacements due to post-earthquake pore water pressure dissipation, 695–6
 - ground displacements due to landslides, 695
 - regional assessments of permanent deformations, 694–5
 - site-specific estimation of permanent ground deformations, 694
 - wave propagation strain, 693–4
- earthquake-induced ground motion, 668–9
- future trends, 74
- ground motion spatial correlation to civil infrastructure, 57–74
- LADWP characterisations using 59 scenario earthquakes, 665–8
 - characteristics, 667–8
 - spatial distribution of faults, 666
- Montreal, 585
 - uniform hazard spectra, 585
- seismic demand on system components, 669
 - PGV contour surface for the Verdugo scenario earthquake, Plate III
- seismic loss assessment, 69–74
- structural vulnerability analysis of civil infrastructure facing, 209–27
 - advantages, 226–7
 - risk analysis, 218–23
 - vulnerability, hazard and risk, 210–12
 - vulnerability identification, 212–18
 - vulnerability of infrastructure networks, 223, 225–6
- types, 688–91
 - permanent ground deformation, 689–91
 - volcanic hazards, 691
 - wave propagation, 689
- seismic loss
 - estimation application, 69–72
 - characteristics of loss distribution, 70
 - parameters of site-to-site within-earthquake correlation functions, 67
 - estimation recent trends, 72–4
- seismic performance, 80–3
 - analysis of new segmented joint, 635–8
 - ground and structural strains along pipe-stretching axis, 637
 - pipe elongation induced by surrounding soil, 636
 - shear stress and strain on the pipe surface and surrounding ground, 636
 - contemporary ground motion selection, 82–3
 - design procedure of existing segmented pipelines in Japan, 632–5
 - flow chart, 634
 - design procedure of existing segmented pipelines in USA, 631–2
- fragility curves for segmented pipelines, 638–40
 - damage rates for CIP and DCIP, 640
 - seismic spectral response velocity parameters, 639
- ground motion selection, 80–1
 - coupled and uncoupled seismic response computation, 80
- probabilistic seismic hazard analysis (PSHA), 81–2
- risk assessment, 585–91
 - fragility curves for different damage states, 589
 - IDA results and estimated probability of exceeding different damage states, 588
 - IDA results developed for regular bridge, 587
 - mean annual rate of exceeding damage states, 591
 - seismic hazard curves for Montreal class C, 590
- seismic performance assessment
 - high-rise buildings, 321–31
 - initial structural and content costs, 327
 - loss curves of the prototype building, 331
 - maximum floor acceleration in H1 direction, 329
 - maximum inter-story drift ratio in H1 direction, 328
 - prototype description, 322–5
 - repair cost distribution of the prototype model., 330
 - response analysis, 326–7
 - results, 327–31
 - site seismic hazard, 325–6
- seismic reliability
 - deteriorating reinforced concrete (RC) bridges, 514–32
 - deterioration effects, 519–31
 - deterioration mechanism, 515–19
- seismic response analysis, 104–5
 - seismic demand distribution bias checking, 105
- seismic risk, 31–2
 - possible maximum loss (PML) analysis of RC structures, 741–58
 - analytical procedure for assessing seismic risk, 743–7
 - case studies, 747–57
 - future trends, 757–8
- seismic risk assessment, 150–5, 641–50
 - analytical procedure, 743–7
 - event risk curve based on multi-event model, 744–7
 - seismic risk curve based on analytical method, 743–4
- Bayesian belief network (BBN), 175–202
 - future trends, 201–2
 - overview, 175–6
 - regional damage estimation, 187, 189–94
 - site-specific hazard assessment, 181–7
 - vulnerability and damage assessment of buildings, 195–201
- capability approach, 255–66
 - benefits, 265
 - framework, 258
- civil infrastructure, 141–70
- definition of seismic performance, 642–3
 - seismic performance and failure modes, 643
- general framework for evaluating seismic risk, 660
 - framework of water supply systems, 661
- ground motion selection of civil infrastructure, 79–109
 - bias in seismic response analysis, 104–5
 - bridge foundation soil system, 83–4
 - future trends, 108–9

- generalised conditional intensity measure (GCIM), 84–93
- methodology application, 93–103
- performance assessment, 80–3
- seismic demand curve computation, 105–7
- software implementations, 108
- hazard surrounding lifeline system, 643–7
 - scenario earthquake accelerogram generated from an active fault, 644
 - simulated accelerogram of worst ground motion, 645
 - velocity response spectra of worst ground motion, 645
- incremental dynamic analysis (IDA), 561–92
 - case study for continuous 4-span bridge, 580–91
 - fragility curves development using IDA results, 579–80
 - future trends, 591–2
 - record selection, 568–78
 - structural modeling, 564–7
 - uncertainty sources, 567–8
- key steps, 685–8
 - acceptable annual probabilities, 686
 - relative uncertainty in vulnerability assessment parameters, 688
- oil and gas pipelines, 682–712
 - challenges and issues, 705–8
 - consequences of pipeline damage, 699–702
 - determining hazard likelihood, 691–2
 - determining severity hazard, 692–6
 - future trends, 708–11
 - mitigation approaches to reduce risk to pipelines, 702–5
 - pipeline response to earthquake hazards, 696–9
 - seismic hazard types, 688–91
- performance-based earthquake engineering (PBEE), 307–32
 - framework, 308–21
 - high-rise buildings seismic performance assessment, 321–31
- performance failure, 648–50
- restoration process after an earthquake, 649
- water serviceability in life-cycle stage, 649
- purpose, 683–5
 - system performance metrics and acceptance criteria, 684
- soil-structure interaction and spatial variability of ground motion of bridges, 597–619
 - earthquake excitation scenarios, 610–19
 - embankment–backfill–abutment–superstructure, 607–9
 - soil–foundation–pier–superstructure interaction, 599–607
- structural failure, 647–8
- uncertainty, 143–9
 - theories by levels of their generality, 144
- water supply system, 659–78
 - component responses, 669–72
 - economic and social consequences, 676–7
 - future trends, 677–8
 - seismic hazards, 664–9
 - system characteristics, 660–4
 - system responses, 672–6
- wind turbine support structures, 716–37
 - demand models for support structure of offshore wind turbines, 719–32
 - fragility estimates, 733–5
 - future trends, 737
 - probabilistic demand models, 718–19
- seismic risk curve, 743–4
 - based on analytical method, 743–4
 - procedure, 745
 - based on multi-event model, 744–7
 - integrated seismic risk curves, 746
- seismic risk management, 155–69
 - capability approach, 255–66
 - benefits, 265
 - framework, 258
 - public policy and resource allocation, 264–5
 - risk acceptability and tolerability, 263–4
 - risk determination and measurement, 259–62
- civil infrastructure, 141–70
- insurance losses using extreme value theory and copula, 760–84
 - earthquake insurance portfolio analysis, 774–83
 - future trends, 783–4
 - insurer's earthquake risk exposure modeling, 767–74
 - statistical modeling and extreme data, 763–7
- prevention and mitigation, 158–62
 - alternative retrofit strategies for seismic upgrading, 161
 - seismic design forces evolution in NBCC, 159–60
 - total fatalities vs monetary loss for world earthquakes, 161
- water pipeline networks, 626–57
 - future trends, 656–7
 - risk assessment, 641–50
 - seismic failure of lifeline system, 627–41
 - seismic risk mitigation, 650–6
- seismic risk mitigation, 650–6
 - case study of sample network system, 652–6
 - numerical sample, 652–4
 - simulation results, 654–6
 - criteria for decision making, 652
 - decision making, 650–1
- seismic risk models
 - ageing and deteriorating buildings and civil infrastructure, 387–406
 - combined progressive and shock-based deterioration, 403–6
 - deterioration mechanism, 388
 - graceful deterioration approximation, 399–403
 - shock-based damage accumulation models, 391–9
 - structural degradation, 389–91
- seismic safety, 256
- Seismic Source Database, 824–5
- seismic vulnerability, 235–7
 - approaches, 336–42
 - analytical methods, 337–9
 - assessment procedures of masonry structures, 340–1

- empirical methods, 336–7
- hybrid methods, 339–42
- collapse-mechanism approach, 342–54
- masonry buildings, 334–62
- stochastic and epistemic uncertainty
 - quantification, 354–60
 - epistemic uncertainty on fragility distribution for limit states, 361
 - fragility curve, 357–8
 - LS, DL and SD for three classes of collapse, 355–6
- seismic vulnerability analysis
 - application, 499–509
 - super-graph comprising all systems with their dependency edges, Plate I
 - complex interconnected civil infrastructure, 465–510
 - future trends, 509
 - main classes description of building, 472–88
 - model, analysis type and interactions, 468–70
 - object-oriented model (OOM) of infrastructure and hazards, 470–2
 - performance metrics, 488–93
 - probabilistic assessment of the model, 493–9
 - time, space and stake holder dimensions of the problem, 466–8
- single scenario event results, 503–5
 - functional consequences of physical damage and social impacts, 506
 - physical damage to buildings and lifelines, 504
- study area description, 499–503
 - log-normal fragility curves for two limit states and building typologies, 501
 - number of buildings per BCA, SCD, LUPA, 501
 - study region employed to exemplify the presented model, 500
- whole simulation results, 505, 507–9
 - complete simulation, 508
 - regional seismicity, 507
- seismological models, 41–2
- SeIEQ, 612
- semi-empirical model, 851–4
- semi-Markov process, 392
- semi-variogram, 61–2
- service life
 - deterioration and structure, 518–19
 - structural damage and resulting service life for various conditions, 519
- settlement, 123–4
- ShakeMap system, 843–4, 852
- shallow active crustal regions, 33–4
 - magnitude–distance distribution of a subset of the NGA database, 33
- shear demand model, 730–1
- Shelter Seeking Population (SSP), 493
- shock-based damage accumulation, 397–8
 - models, 391–9
- shock-based degradation, 391–9
 - accumulation models, 392–3
 - combined progressive degradation, 403–6
 - basic formulation, 403–4
 - combined damage results, 404
 - RV model parameter, 404
 - damage accumulation due to successive earthquake, 391–2
 - hazard rate-based stochastic model, 393–4
 - successive reconstruction, 397–9
 - degradation for regenerative process, 398
 - regenerative process, 399
 - systems abandoned after failure, 394–7
 - structural failure probability models comparison, 396
 - time to failure distribution, 395
- short-term recovery models, 285
- SIGE system, 841
- significant damage, 351, 353
- simulation
 - results, 654–6
 - deterioration effect on seismic retrofitting cost, 656
 - old-pipe replacement for seismic investment, 655
 - probability of system failure for retrofitting periods, 654
 - probability of system failure for various seismic investment, 655
 - techniques, 524
- single degree of freedom (SDOF), 344, 421, 854
- site seismic hazard, 325–6
- site-specific hazard (SSH), 181–7
 - BBN in liquefaction assessment, 183–7
 - assessment of different models for liquefaction classification, 189
 - Bayesian network for liquefaction assessment, 187
 - classification results for liquefaction, 188
 - correlation matrix of considered variables, 185
 - histogram of six variables, 184
 - regional and individual building damage estimation, 187
 - results through different classification technique, 186
 - search algorithms, 186
- soft computing application
 - lateral spreading, 182–3
 - liquefaction, 182
- Sklar theorem, 762, 765
- social/cultural capital dimension, 277
- social vulnerability index, 831, 833
- societal risk, 152–3
 - criteria, 153
- soft computing, 176
 - application for lateral spreading, 182–3
 - application for liquefaction, 182
- soft risks, 229–53
- soft system uncertainty, 238, 241
- soil–foundation–pier–superstructure interaction, 599–607
 - piers founded on pile shafts (single or groups), 599–604
 - dynamic loading, 603–4
 - static loading, 599–603
 - soil liquefaction effect on dynamic bridge response, 606–7
 - soil nonlinearity, 605–6
 - soil–structure interaction and characteristic of ground motion, 604–5

- soil–structure interaction
 - earthquake excitation scenarios for
 - interactive soil–bridge systems, 610–19
 - asynchronous excitation, 614–19
 - excitation direction, 613–14
 - finite element model of Krystallopigi Bridge, 614
 - rotational component of foundation response, 612–13
 - selection and generation of earthquake ground motion scenarios, 611–12
 - seismic risk assessment of bridges, 597–619
 - embankment–backfill–abutment–superstructure interaction, 607–9
 - soil–foundation–pier–superstructure interaction, 599–607
- spatial variability
 - ground motion on seismic risk assessment of bridges, 597–619
 - earthquake excitation scenarios, 610–19
 - embankment–backfill–abutment–superstructure, 607–9
 - soil–foundation–pier–superstructure, 599–607
- spectral acceleration (SA), 3
- spectral models, 39–41
- spectral shape factor (SSF), 578
- stable continental regions, 34–6
 - magnitude–distance distribution, 35
- Standard Penetration Test (SPT), 18, 83, 117–18
- static loading, 599–603
 - finite element modeling of pile groups using uniaxial soil springs, 600
- statistical modeling, 763–7
 - dependence modeling, 765–7
 - extreme value theory and marginal distribution modeling, 763–5
- statistically independent and identically distributed (SIID), 414
- stochastic models, 39–41
 - building and civil infrastructure deterioration, 410–32
 - general deterioration process, 411–14
 - schematic diagram, 412
 - reinforced concrete (RC) bridges deterioration, 418–30
 - stochastic semi-analytical approach (SSA), 414–18
- stochastic semi-analytical approach (SSA), 411, 414–18
 - failure time analysis, 416–18
 - formulation of failure, 415–16
 - modeling the effect of deterioration on capacity, 415
 - modeling the effect of deterioration on demands, 414–15
- stochastic uncertainty, 354–60
- stress-based liquefaction potential analysis, 6
- structural damage, 344
- structural degradation
 - definition, 389–90
 - general model, 391
- structural deterioration, 389–90
 - general process, 411–14
 - schematic diagram, 412
 - stochastic modeling in building and civil infrastructure, 410–32
 - stochastic modeling in reinforced concrete (RC) bridges, 418–30
 - stochastic semi-analytical (SSA) approach, 414–18
- structural performance targets
 - performance-based earthquake engineering (PBEE) approach for bridges, 536–58
 - future trends, 556–8
 - performance-based seismic evaluation framework (PEER approach), 537–42
 - performance-based seismic evaluation of ordinary highway bridges, 546–56
 - probabilistic seismic demand analysis (PSDA), 542–4
 - vector-valued probabilistic seismic hazard assessment (PHSA), 544–5
- structural rings, 212–14
 - schematic diagram, 213
- structural rounds, 212–14
 - schematic diagram, 213
- structural vulnerability analysis
 - advantages, 226–7
 - civil infrastructure facing seismic hazards, 209–27
 - infrastructure networks, 223, 225–6
 - risk analysis, 218–23
 - vulnerability, hazard and risk, 210–12
 - vulnerability identification, 212–18
- subduction zones, 36–7
 - magnitude–distance distribution, 37
- sufficient IM, 543
- sum of squared errors, 572
- system-level metrics, 489–92
- system responses, 672–6
 - LADWP, 673–4
 - spatial distribution of water outage, Plate IV
 - risk curves, 674–6
 - schematic diagram, 675
- system serviceability index (SSI), 673
- systems thinking, 238–43, 438–40
 - complexity attenuation, 440
- taxonomy, 827–8
- test data-related (TD) uncertainty, 568
- time-dependent functionality map, 277
- total probability theorem, 7, 589
- total repair cost
 - computation, 316–17
 - cost function, 317
 - different representation, 317–19
 - distribution of repair cost, 318
 - loss curve, 319
- transient ground deformation (TGD), 668–9
- Transmission and Distribution system, 481
- transport networks, 226
- trend-dependent, 401–2
- triangular fuzzy numbers (TFN), 146
- TurbSim, 719–20, 725
- Turkish Catastrophe Insurance Pool (TCIP)
 - basic properties of compulsory earthquake insurance scheme, 804–5
 - statistical information on implementation of compulsory earthquake insurance, 805

- earthquake insurance implementation, 803–8
 - background information, 803–4
 - future actions, 807–8
 - major problems, 806–7
 - objectives, 805–6
 - organisational structure, 806
- UBC-SAWS models, 771
- Unified Modeling Language (UML), 470
- uniform hazard spectra (UHS), 108
- United States Geological Survey (USGS), 664
- unreinforced masonry (URM), 345–6
- upgrade benefit index (UBI), 492
- user-access cost, 458
- vector-valued probabilistic seismic hazard
 - assessment (PHSA), 544–5
- verification testing, 704
- vibro-replacement, 703
- virtual experiment data, 723–7
 - analytical modeling, 723–7
 - bins from which ground motions are selected, 724
 - finite element model of sample wind turbine support structure in ABAQUS, 726
 - offshore wind turbine model in FAST and ABAQUS, 725
 - equality and lower bound data, 727
 - Latin hypercube sampling, 723
 - geometrical and mechanical properties used in experiment design, 724
- virtual seismotectonic source zones, 797
- volcanic hazards, 691
- vulnerability
 - assessment, 449–52
 - detection strategy, 451
 - principle of maximum sparsity between clusters, 450
 - future trends, 383–5
 - hazard and risk, 210–12
 - identification, 212–18
 - clusters and hierarchy, 214–15
 - deteriorating events and failure scenarios, 215–17
 - rings and rounds, 212–14
 - samples, 217–18
 - structure in sample 1 and its hierarchical representation, 217
 - infrastructure networks, 223, 225–6
 - lateral load resistance of fire-damaged column using hybrid method, 383
 - reinforced concrete assessment for fire and earthquake risk, 366–85
 - reinforced concrete building fire performance following an earthquake, 370–9
 - residual seismic resistance of fire-damaged columns, 379–83
 - structure response to fire, 368–9
 - structure seismic response, 369–70
- vulnerability index method (VIM), 336
- waste water system (WWS), 471
- water pipeline networks, 223, 225
- sample and hierarchical representation, 225
- seismic risk management, 626–57
 - future trends, 656–7
 - risk assessment, 641–50
 - seismic failure of lifeline system, 627–41
 - seismic risk mitigation, 650–6
- water supply system (WSS), 470–1, 478–81
 - model, 478–81
 - elementary WSS and EPN and corresponding flow equations, 480
 - seismic risk assessment, 659–78
 - component responses, 669–72
 - economic and social consequences, 676–7
 - future trends, 677–8
 - general framework for evaluating seismic risk, 660
 - seismic hazards, 664–9
 - system responses, 672–6
 - system characteristics, 660–4
 - operational characteristics of LADWP system, 663–4
 - physical and operational characteristics, 661
 - physical characteristics of LADWP system, 662–3
- wave propagation, 689, 693–4
- weight vector
 - effect on semi-automated ground motion selection, 94–9
 - amplitude scale factors vs magnitude distance distribution of ground motions, 98
 - nine *SA* ordinates, *CAV*, *Ds575* and *Ds595*, 97
 - scale factors, *SF*, mean magnitude and distance of ground motion, 98
 - uniform weights for nine *SA* ordinates, 96
- weighted connectivity loss (WCL), 490
- wind turbine support structures
 - demand models for offshore, 719–32
 - deterministic demand model, 719–21
 - model correction, 721–3
 - probabilistic demand models, 727–32
 - virtual experiment data, 723–7
 - fragility estimates for offshore, 733–5
 - properties of NREL offshore 5MW baseline wind turbine, 733
 - proposed damage states, 734
 - spectral acceleration function, 735
 - wind speed function at different spectral acceleration, 736
 - seismic risk analysis, 716–37
 - future trends, 737
 - probabilistic demand models, 718–19
- Winkler model, 600, 717
- within-earthquake correlation, 59–62, 63, 66, 73, 74
- within-earthquake residual, 61
- within-earthquake variability, 73
- World Agency of Planetary Monitoring and Earthquake Risk Reduction (WAPMERR), 840
- World Housing Encyclopaedia (WHE), 854–5
- WSSedge, 478
- WSSnode, 478