



SUSTAINABLE BUILDINGS AND STRUCTURES

Editors: Stephen P. Wilkinson, Jun Xia & Bing Chen



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Sustainable Buildings and Structures

Editors

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Preface

The rapidly developing world, economic intensification and the evolution of urban development have caused a massive surge in environmental pollution, energy consumption and carbon emission. Construction plays a significant role in the emergence of ecological problems where specific attention has to be devoted to develop sustainable and green solutions in the design of buildings and structures. The main aim of the First International Conference on Sustainable Buildings and Structures (ICSBS 2015) is to cultivate research interaction between academics, researchers and practicing professionals from different communities in the multidisciplinary context of sustainability in built environment and infrastructure system. The main themes of the ICSBS 2015 centre on the state-of-the-art research in sustainable structural engineering (i.e. concrete, steel and composite structures), structural durability and hazard mitigation, sustainable building design and innovative technologies as well as building information modelling and innovative urban development or regeneration. The ICSBS 2015 publication consists of 37 high standard full length peer reviewed papers and I am grateful to the authors and co-authors for their contributions to this volume. I would like to take this opportunity to thank the Local Organising Committee and Technical Review Committee for helping make ICSBS 2015 a successful event. I would also like to thank Funding for Advanced Academic Exchange Activities of Suzhou sponsored by Suzhou Association For Science and Technology, Xi'an Jiaotong-Liverpool University and Suzhou University of Science and Technology for their financial supports.

Chee Chin
Chair

*Head of the Department of Civil Engineering,
Xi'an Jiaotong-Liverpool University*

On October 29–31, 2015, the First International Conference on Sustainable Buildings and Structures (ICSBS 2015) was held at the Xi'an Jiaotong Liverpool University, Suzhou, P.R. China. The conference was organized by Xi'an Jiaotong Liverpool University and Suzhou University of Science and Technology and sponsored by Suzhou municipal government.

A total of 47 abstracts and 39 manuscripts submitted to this conference, and 37 papers were finally accepted after a strict review process conducted by the International Scientific Committee of the conference. The topics of the conference covered a wide spectrum related to the sustainability and structural research areas, such as Sustainable construction materials and structural applications, sustainable building design, construction and maintenance, etc. This conference proceedings include 3 keynote presentations and 34 conference papers.

The Chairmen would like to thank all the participants and authors for their contributions. We also would like to warmly acknowledge the guidance and cooperation provided by the International Scientific Committee, the support provided by the Local Organising Committee and the financial support provided by the Suzhou municipal government. Particular appreciation is extended to Xi'an Jiaotong Liverpool University and Suzhou University of Science and Technology for hosting the conference.

Xiaoyong Mao
Co-Chair

*Vice Dean of School of Civil Engineering,
Suzhou University of Science and Technology*

Organization

This volume contains the Proceedings of the 1st International Conference on Sustainable Buildings & Structures—ICSBS 2015 held in Suzhou, China, from 29th to 31st October 2015. ICSBS 2015 has been organised by Xian Jiaotong-Liverpool University & the Suzhou University of Science and Technology.

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Keynotes

High-performance steel structures for buildings

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ABSTRACT: High-performance steel structures for buildings are defined as high capacity to support gravity and wind, high ductility to resist earthquakes and low steel consumption. Thus, high-strength steel is recommended. However, the ductility of high-strength steel cannot satisfy the ductility requirements of structural steel specified in the current codes for buildings in seismic zones. Several configurations of high-performance steel structures with high ductility are proposed. High-strength steel can be used for columns in the steel frameworks with simple or semi-rigid beam-to-column connections. The yielding of columns is prevented from limiting the moment transfer from beams. So the ductility of high-performance steel structures employing high-strength steel is guaranteed. To enhance lateral stiffness and ductility of steel frameworks with simple or semi-rigid connections, buckling-restrained braces or shear walls are inserted. The coupled shear walls using high-ductility steel link beams or steel link walls are also proposed for lateral supporting structures for high-performance steel structures. The high performance of the steel structures proposed is demonstrated with typical examples.

1 INTRODUCTION

The high-performance steel structures for buildings are defined, in this article, as that to satisfy three basic principles, which include high capacity to withstand gravity and winds, high ductility to resist earthquakes, and low steel consumption to be sustainable and environmentally friendly as well.

In order to reduce the steel consumption, high-strength steel can be used for steel structures, especially for columns to supporting gravity loads. However, the ductility of high-strength steel cannot satisfy the ductility requirement of structural steel specified in the current design codes for earthquake-resistance of structures for buildings in seismic zones (GB50011 2010). How to use high-strength steel for structures is a challenge for countries affected by earthquakes, such as China.

In order to enhance the structural lateral stiffness and strength to against wind and earthquakes, steel braces and concrete walls are frequently used all over the world. However, buckling may easily happen on normal steel braces especially under seismic effects. And traditional concrete walls are also easily damaged under earthquakes. Certainly, to restrain the buckling of steel braces and to improve the ductility of concrete walls can improve the performance of structures against lateral actions especially induced by earthquakes.

In this paper, several configurations of high-performance steel structures are proposed for utilizing high-strength steel for buildings in seismic zones and the performance of the structures proposed is fully demonstrated through examples of buildings located in seismic zones.

2 HIGH-STRENGTH STEEL STRUCTURES FOR EARTHQUAKE RESISTANCE

Due to the architectural and structural advantages, high-strength steel (yield strength ≥ 460 MPa) has been increasingly used in high-rise buildings, large span structures and bridges in the past two decades (Rosier et al. 1987, Pocock 2006, Collin et al. 2006). Compared with conventional mild carbon steel members, application of high strength steel not only reduces member size and saves building space, but also shows considerable economic benefits through alleviating workloads of welding and transportation, and shortening periods of construction as well. With the recent development of high strength steel and advancement in welding techniques, high-strength steel members can be produced at a reasonable cost and quality in China.

However, indicated by vast amounts of experimental study on high-strength steel material (Shi 2012, Wang et al. 2013), it is found to fall short of the requirements of elongation and

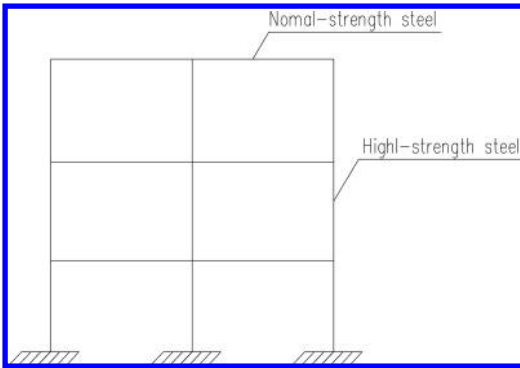


Figure 1. Moment-resisting frame using high-strength steel.

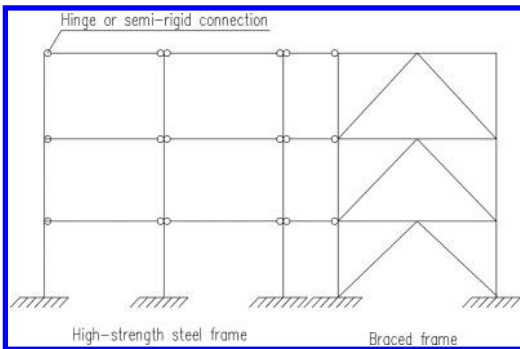


Figure 2. Hinged or semi-rigid high-strength steel frame with braces.

yield-to-tensile-strength ratio, both of which are the basic demands of structural steel specified in the current design codes for earthquake-resistance of structures for buildings in seismic zones that required to meet.

Thus, preventing the yielding of high-strength steel members turns out to be an efficient way of utilizing high-strength steel in structures for earthquake resistance. A case in point is the application of high-strength steel in the moment-resistant steel frames designed following the principle of “strong column and weak beam”, in which the columns are made from high-strength steel while the beams are fabricated in normal-strength steel, as shown in Figure 1.

Besides, by preventing the columns in high-strength steel from yielding through limiting the moment transferred from the beams, hinged steel frame and semi-rigid steel frame can also be adopted. To overcome the weakness of hinged and semi-rigid frames in resisting lateral actions, steel braces or concrete shear walls can be employed, as illustrated in Figure 2 and Figure 3, respectively.

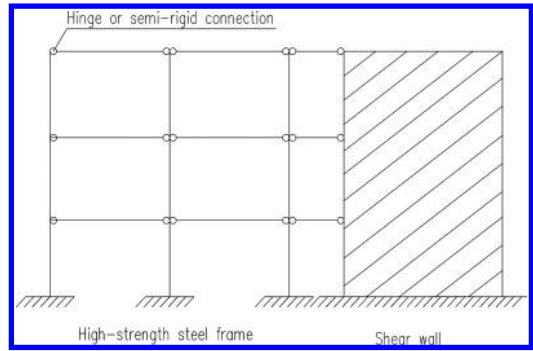


Figure 3. Hinged or semi-rigid high-strength steel frame with concrete shear wall.

3 DUCTILE BRACE-FRAMES

3.1 Configurations and characteristics

Literally, ductile brace-frames are those steel frames laterally enhanced with ductile brace elements such as Buckling-Restrained Brace (BRB) and buckling-restrained steel plate shear wall (BRSW), as shown in Figure 4, because both ordinary steel braces and steel plate shear walls tend to buckle at a early stage.

Ordinary steel braces that are used as diagonal members in framed structures are widely adopted for buildings in seismic zone. But their strength and deformation capacities turn out to deteriorate all of a sudden, as shown in Figure 5, because of its premature buckling when subjected to seismic load, such as 1995 Kobe earthquake, as shown in Figure 6.

Similarly, steel plate shear walls tend to buckle prematurely under seismic forces. And as long as it buckles, its load-carrying capacity and lateral stiffness deteriorate to a large extent, as illustrated in Figure 7.

Therefore, innovative ductile brace elements, such as BRBs and BRSWs are introduced herein, the hysteretic curves of which are ideal spindle-shape, as shown in Figure 8, indicating stable and superior energy-absorbing capability (Fujimoto 1988, Jin 2014).

Typically, brace elements or structural metal dampers are designed to yield first before any plastic hinge occurs in steel frame, such as plastic hinge at the bottom of columns at the first floor. And in cases that high-strength steel is used, the steel frame stays in elastic. By specifying that the brace elements undertake 50% of the lateral forces at least, high ductility and capacity of load carrying are thus assured and high strength of steel is fully taken advantage.

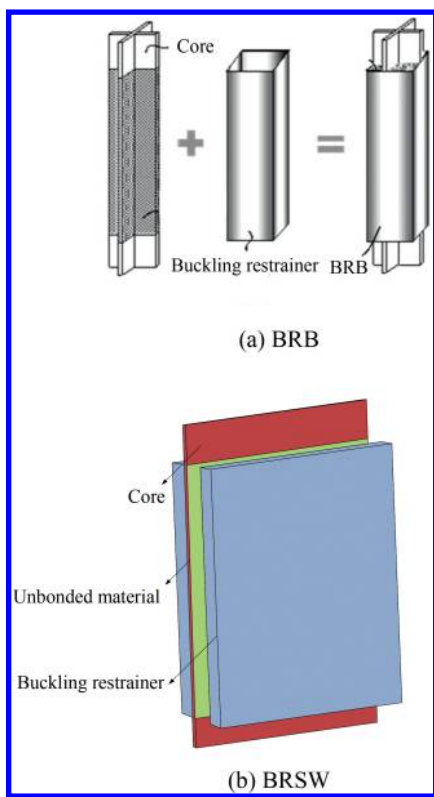


Figure 4. Commonly used ductile brace elements.

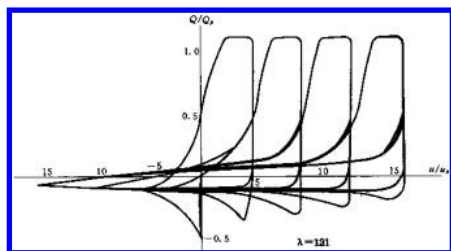


Figure 5. Typical hysteretic curve of ordinary steel braces.

Besides, rotational capacities of beam-to-column connections of ductile braced-frame are required to be beyond 0.02 rad, for which BRBs and BRSWs are ductile brace elements that required to meet the demands of maximum plastic drift ratio under rare earthquake specified in the current codes of China, which is 0.02 rad exactly.

3.2 Exemplification

One example is resorted to illustrate the high performance of ductile braced-frame, in which the

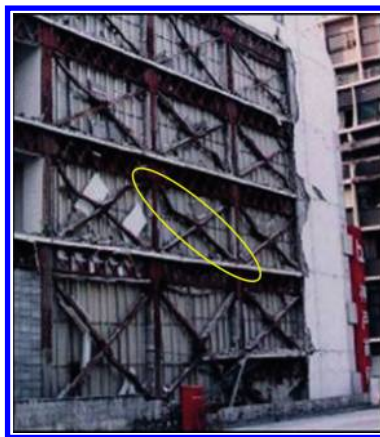


Figure 6. Buckled steel braces in 1995 Kobe earthquake.

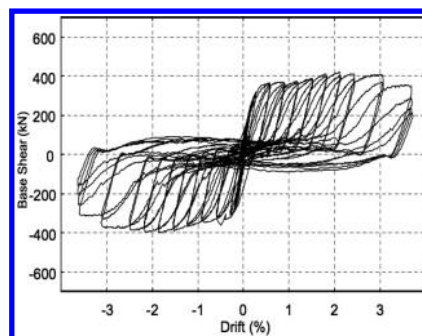


Figure 7. Typical hysteretic curve of steel plate shear walls.

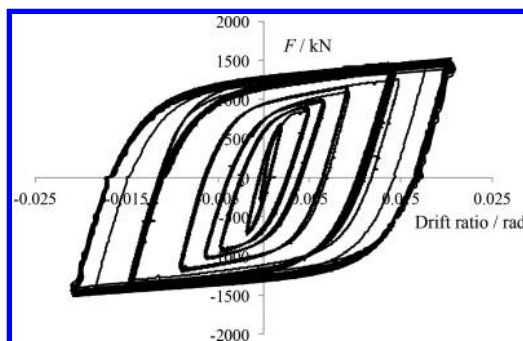


Figure 8. Typical hysteretic curve of BRB and BRSW.

exemplified ductile braced-frame is installed with typical chevron BRBs. By comparing to same steel frame with ordinary steel braces, the enhanced performance of ductile braced-frame such as ductility and strength is self-evident.

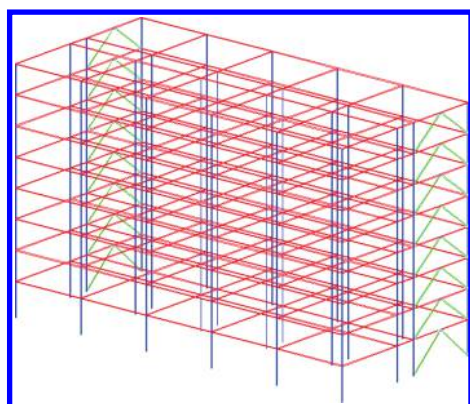


Figure 9. 3D view of analytical model.

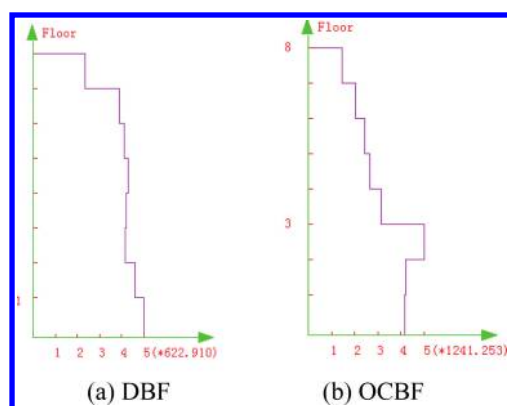


Figure 10. Distribution of maximum inter-story shear force.

One typical Ductile Braced-Frame (DBF) and one Ordinary Concentrically Braced-Frame (OCBF), as shown in Figure 9, located in a seismic zone with a seismic intensity of 8 degrees, are exemplified.

The result of analysis shows that base shear of DBF is only around 60% of OCBF, calculated in the most unfavorable load case, while the maximum inter-story shear force of the former is however much more uniform than the latter, as illustrated in Figure 10, which avoids the formation of weak floor in buildings and assures high ductility.

Also, the maximum inter-story drift ratio of ductile brace-frame is mitigated to a considerable extent of 66%, by contrast with that of OCBF, as can be concluded in Figure 11.

From all what can be concluded, ductility and capacity of load carrying of ductile brace-frame with BRBs and BRSWs installed are proved to be superb compared to steel frames with ordinary braces.

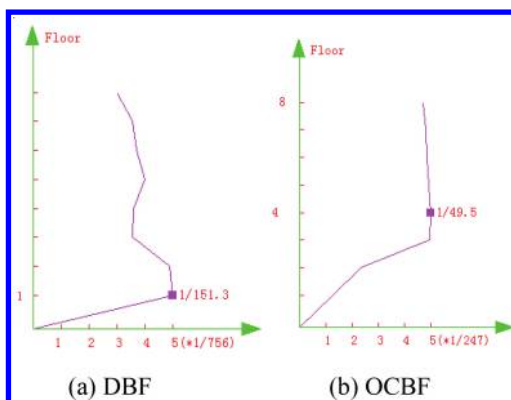


Figure 11. Distribution of maximum inter-story drift ratio.

4 DUCTILE SHEAR WALL STRUCTURES

Concrete shear wall is an effective structural element in providing high lateral stiffness, because of which a great proportion of seismic forces is resisted by itself. Consequently, concrete shear walls tend to be severe damaged in shear forces under earthquake, which leads to undesirable brittle failure. Besides, concrete shear walls are deliberately designed to possess the “plastic hinge” at the bottom in order to dissipate energy. However, extensive damages are observed in the bottom of wall piers and unfortunately, they are almost unable to be fully repaired.

Based on what has been concluded above, and in order to satisfy the architectural demands, such as openings and voids, concrete link beams are introduced, as shown in Figure 12. In the coupled shear walls, a majority of the input energy is dissipated by the link beams distributed over the structural height as opposed to the plastic zone concentrated at the base of flexural wall piers. In other words, the link beams are designed to provide the first line to resist the lateral forces and dissipate energy effectively as well. Wall piers are expected to function as the second line and prevent the total collapse of the structure. Since link beams serve as the first line to resist earthquake, their seismic performance is crucial to the overall structural behavior. However, concrete link beams, especially those with small span-to-depth ratio, are limited in shear capacity and poor in ductility, whose failure mode is indicated in Figure 13.

4.1 Coupled shear wall structure with steel link beams

Therefore, many researchers have turned to focus their research on steel link beam, as an alternative to reinforced concrete link beam, for steel link

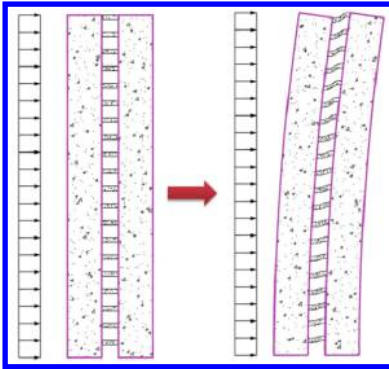


Figure 12. Coupled shear wall.

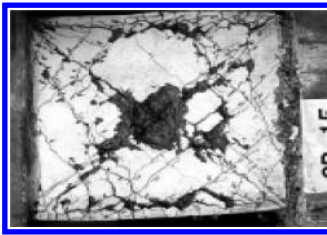


Figure 13. Typical failure of concrete link beam.

beam possesses remarkable stiffness, strength, and ductility, indicating its high strength without doubt (Ei-Tiwil et al. 2002, 2010). Besides, some researchers deliberately add steel damping devices to the steel link beam in order to dissipate more energy at an early stage of seismic load, such as “Fuse” steel link beam (Fortney et al. 2007), friction damper beam (Chung et al. 2009), SMA damper (Dong 2011), and viscoelastic coupling damper (Christopoulos 2013).

However, according to what has been founded in practical design, steel link beam hardly dissipates energy in frequent and moderate earthquake, but only play a great part in rare earthquake. And the previous method of deliberately adding damping devices cannot guarantee sufficient stiffness to ensure the coupling action, although it is capable of dissipating earthquake energy both in frequent and rare earthquake. Therefore, through combining those advantages of both methods, a new type of steel link beam, which is called double-step yielding steel link beam, is brought forward.

The double-step yielding steel link beam is well designed to possess two yielding points, as indicated in Figure 14, one part of which is to yield under frequent earthquake, while the other keeps elastic. Thus, it is able to provide both additional damping and sufficient stiffness under minor

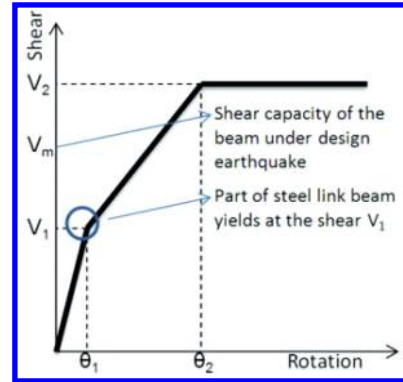


Figure 14. Double-step yielding of steel link beam.

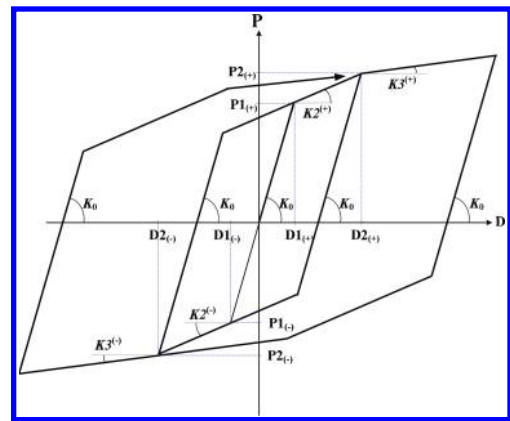


Figure 15. Hysteretic behavior of double-step yielding steel link beam.

earthquakes. Meanwhile, it is also capable of yielding to dissipate energy when subjects to moderate and rare earthquake, the hysteretic model of which is shown in Figure 15.

The application of this kind of steel link beam will lead to an economic structure for certain, because energy could be dissipated earlier even under minor earthquakes and base shear could be reduced to a certain extent. Thus, steel consumption is reduced remarkably.

The double-step yielding of steel link beam is composed of two separated parts. The inelastic part, as shown in Figure 16(a), which is made of low-yielding-point steel, is designed to yield first when subjected to frequent earthquake, while the other, as shown in Figure 16(b) stays in elastic, which is made of high-strength steel, in order to provide stiffness under minor seismic loads and also be capable achieving yielding when subjected to moderate and rare earthquakes.

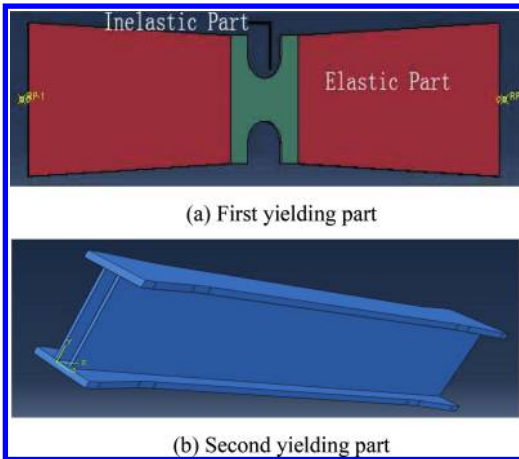


Figure 16. Components of double-step yielding steel link beam.

4.2 Coupled shear wall structure with buckling-restrained steel link walls

And out of the drawbacks of solid cantilever shear wall structures, coupled shear wall structure with buckling-restrained steel link wall is introduced and it is comprised of a pair of individual cantilever concrete shear walls, between which an array of buckling-restrained steel link walls is equally spaced along the structural height, as indicated in Figure 17.

Analogous to the coupling beams in traditional coupled shear walls, the link elements (buckling-restrained steel link wall) function as the energy-dissipation elements and also have considerable stiffness to strengthen the framing action of coupled shear walls for drift control. Since steel plates may be thin, their height should be large enough to provide considerable shear stiffness to connect the adjacent shear wall piers. Actually, in practical structures, there are substantial amount of solid cantilever shear walls where architectural openings are not necessary. Therefore, they are innovatively inserted with buckling-restrained steel link walls in order to increase the seismic performance of these cantilever wall structures. And it turns out to have equal stiffness and strength, compared to that of solid cantilever shear wall structure. The design process is explained in Figure 17.

Except for equal stiffness and strength the proposed ductile shear wall structure has exhibited, it still takes a few advantages over the latter. Firstly and most importantly, the energy-dissipating elements, buckling-restrained steel link wall, possesses excellent and stable energy-absorption capacities, which is able to build a desirable energy dissipating

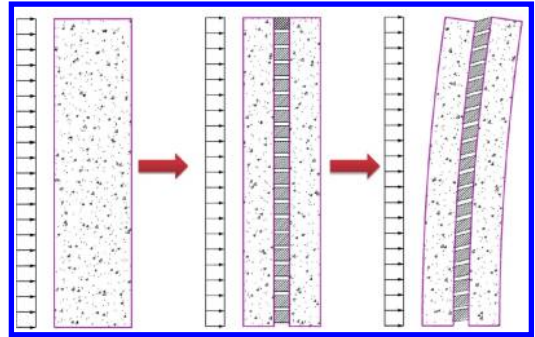


Figure 17. Ductile shear wall structure with buckling-restrained steel link walls.

mechanism as well as to protect the wall piers. The earthquake-induced energy could be mainly dissipated by the uniformly distributed steel link wall rather than concentrating in plastic hinge located at the bottom.

Besides, buckling-restrained steel link wall could be readily connected to the surrounding steel columns embedded in the boundary elements of concrete walls with fin plates. This simply connection details has brought a great deal of convenience to expedite the on-site installation of buckling-restrained steel link walls, as well as in retrofitting or replacement, in cases that they were damaged under rare earthquakes.

4.3 Exemplification

Two analytical examples are employed to elaborate high performance of proposed shear wall structures. One is coupled shear wall structure with steel link beams, while the other is coupled shear wall structure with buckling-restrained steel link wall.

A wall-frame structure (denoted as S-a) with typical floor plan indicated in Figure 18(a) is analyzed. A pair of channel wall piers constitutes the concrete core located at the center of plan. Concrete coupling beams are used in the x direction to connect the individual wall piers at each floor level. Thus, coupled shear walls generates in the x direction. While in the y direction, there is no coupled shear wall but solid cantilever shear wall to resist lateral loads.

As a comparison, in the 'S-b' structure, shown in Figure 18(b), double-step yielding steel link beam replace the concrete link beams in the x direction with the aim of encouraging more energy dissipated. By comparing seismic behaviors of above two structures, high performance of the proposed ductile shear walls structure with double-step yielding link beams is demonstrated self-evidently.

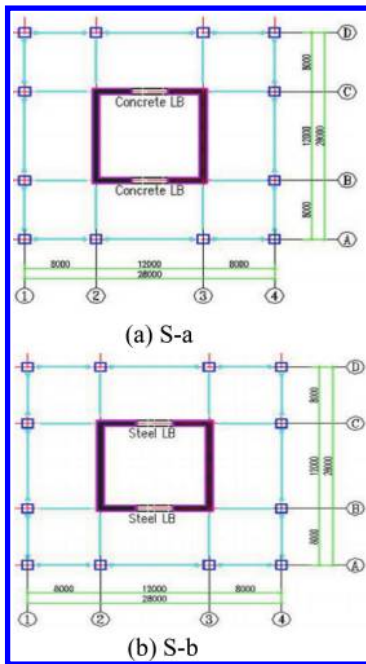


Figure 18. Plans of structure in Example I.

It is assumed that the regular building has 40 stories with equal story height of 4 m. The aspect ratio of the core structure in both directions is 13.3. The typical floor loads are presumably 4.0 KN/m^2 for dead loads and 3.0 KN/m^2 for live loads. The steel framing structure comprises of the moment-resisting perimeter steel frames and the floor beams pinned at each ends, which are connecting the perimeter columns and centered walls. Besides, the ratio of wall cross-section area to building floor area at the bottom in each direction varies from 2.0% to 2.4%.

Firstly, elastic analysis are carried out, from which it can be seen that the total weight and periods of “S-a”, “S-b” are close in each direction. Then, in order to verify the efficiency of proposed structures, elasto-plastic time-history analysis is conducted to investigate their structural behavior in earthquakes. Three groups of seismic records, which include SR1, SR2 and SR3, are selected based on Chinese seismic code speculations, and they are scaled with different coefficients for each group. For the “S-a” and “S-b” structure, the seismic records are scaled to 55 gal, in order to perform time-history analysis under minor earthquakes.

According to the result, comparisons on maximum base shear and inter-story drift ratio are shown in Figure 19.

Meanwhile, another typical shear wall structure is investigated. The structure is assumed to

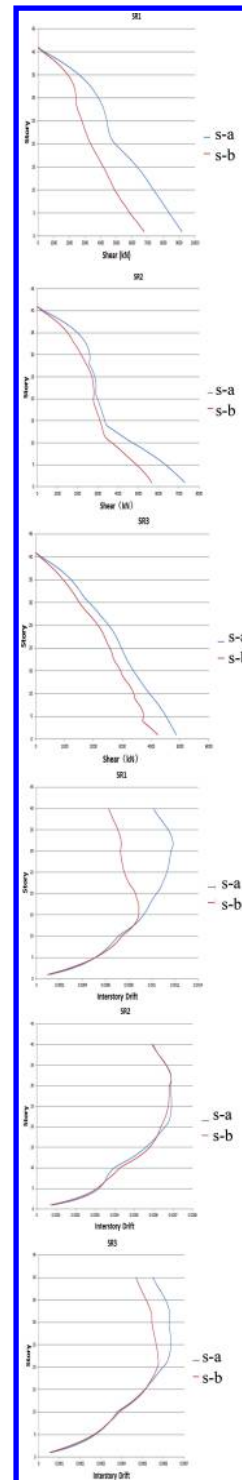


Figure 19. Maximum base shear and inter-story drift ratio.

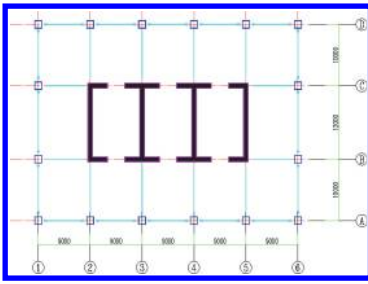


Figure 20. Layout of typical floor in Example II.

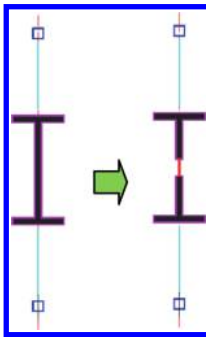


Figure 21. Utilization of steel link wall.

have a total height of 200 m and 50 stories with equal height for simplicity. The typical floor plan is shown in Figure 20. A noticeable characteristic of the plan is that, in the y-direction, cantilever shear walls are used to resist the lateral seismic loads, which is lack of sufficient energy-dissipation capacities. In order to enhance the seismic performance in this direction, double-step yielding steel link beam and buckling-restrained steel link wall are inserted to transform the previous cantilever shear wall structure to coupled shear wall structure, as shown in Figure 21. For simplicity, one bay of the structure in the y-direction is studied only, taking account of the wall flanges. Floor beams are end-pinned to concrete walls and steel columns. Steel reinforcement (including distributing rebar and longitudinal reinforcement in the boundary element) in the concrete walls is also taken into consideration.

It is also assumed that all structural members in the original frame-shear wall system are well designed to remain elastic under minor earthquake and wind loads as China Codes stipulates.

According to elastic analysis, the total weight and period of original shear wall structure, denoted as SW, and transformed structure, denoted as CSW, are close in each direction.

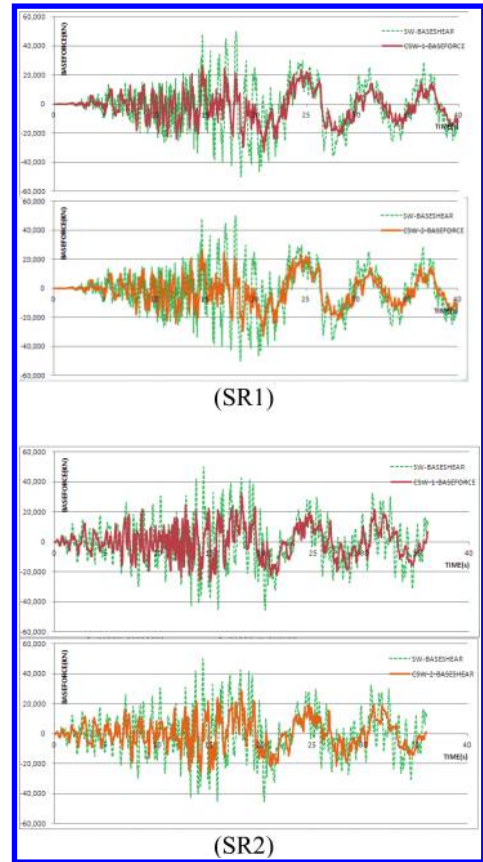


Figure 22. Comparison of base shear.

Then, a pair of strong ground motions including SR1 and SR2 is chosen for the nonlinear dynamic time-history analysis. Both records are scaled to 400 gal at the peak value, according to China code.

Figure 22 well demonstrates that the proposed ductile shear wall structure with buckling-restrained steel link wall could significantly reduce the base shear under the ground motions compared to the original cantilever shear walls. Elasto-plastic inter-story drift ratio as a crucial factor to reflect the structural performance under severe earthquake, is compared in Figure 23, indicating a significant decrease in inter-story drift ratio. It is the reason for which proposed structure is a sort of energy-dissipating system, so that the seismic response could be largely relieved. Besides, concrete damage factors are investigated to assess the structural seismic performance. As can be seen from the results of time history analysis, compared to the original cantilever walls, damage of concrete shear wall is much more mitigated.

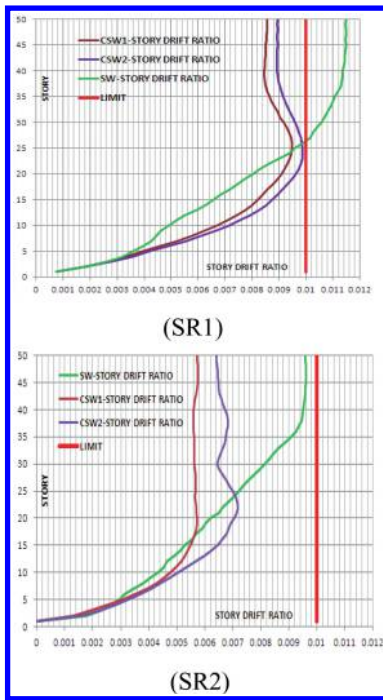


Figure 23. Comparison of inter-story drift ratio.

5 SUMMARY AND CONCLUSION

Configurations of high-performance structures using high-strength steel are elaborated in this paper. High-strength steel is recommended being used mainly for columns of frameworks, while beams are suggested in normal-strength steel. Besides, moment-resistant capacity of beams is proposed to be less than that of columns in high-strength steel structure, in order to limit the yielding of columns. Meanwhile, the yielding of columns in high-strength steel can also be prevented by using hinge or semi-rigid configuration for beam-to-column connections.

Ductile braced-frame and ductile shear wall structure with double-step yielding steel link beam or buckling-restrained steel link wall are recommended for high-performance structures against lateral actions especially induced by earthquakes.

In ductile braced-frame, brace elements such as BRBs and BRSWs are capable of not only providing stiffness and strength, but also taking a great part in dissipating earthquake energy, which is well proved by numerical analysis on comparison with the frame installed with ordinary steel braces in terms of maximum base shear and inter-story drift ratio.

It is also demonstrated that performance of concrete shear wall structures can be greatly improved by double-step yielding steel link beams and buckling-restrained steel link walls. Ductile shear wall structures proposed turn out to be a strong support for high performance of steel structures with high capacity, high ductility and low steel consumption.

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Computational analysis and design of composite steel and concrete structures

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ABSTRACT: This keynote paper presents the structural behaviour of composite structures under fire loading. It will provide an in-depth understanding of the performance of a whole structure with the interactions between each of its elements through a comprehensive finite element model by using the ANSYS software. Computer modelling was conducted to beam-to-column connections, composite floors and frames. The simulation presents the results of the time dependent temperature distribution within the structures, the structural responses under elevated temperature as well as their effects on the fire resistant capacity of the structure. Close agreement has been obtained between computing modelling results and fire test data. It has provided an important tool for this type of structure to be better designed under extreme loading conditions.

1 INTRODUCTION

Composite metal floor system is constructed by steel columns, steel beams with welded-through steel decking concrete floors. It has been widely used in the multi-storey construction due to its advantages of higher strength and stiffness, fast track construction on-site (Bailey & Ellobody 2009). However, with the presence of different thermal properties of steel and concrete components in the structural system, composite flooring system is very critical under the fire situation. Its structural has been much less investigated under fire compared with the bare steel structures.

The concept of fire safety design has been improved significantly and become more rational and acceptable to design and construction engineers in the last few years. It may not possible to prevent fires from occurring in buildings thus the aim of the fire safety engineer is to reduce the risk presented by fire to acceptable levels. With fast improved computer capability and software, finite element technique can be used to model the response of composite flooring system in the fire situation. The research presented here has been focused on investigating the behaviour of connections, floors and frames under fire conditions by using ANSYS software. The results obtained will provide a solid foundation for the Performance Based design method of steel structures in comparison with the traditional Prescriptive Based design method for steel structures design under fire.

2 MODELLING ON BEAM-TO-COLUMN CONNECTIONS

2.1 *3D finite element modeling on steel connections*

There are many types of steel connections being used in steel construction worldwide. Steel beam-to-column end-plate connections are commonly used for steel frame building construction as a standardised joint detailing in many countries. A typical flush end-plate connection consists of a rectangular steel plate welded to the full depth of beam-end. The assembly is connected to the flange of a column by structural steel bolts with or without friction force applied. In the traditional steel structure design, the behaviour of beam-to-column connections has been assumed to be either rigid or pinned. However, from a large volume of research conducted it has been proved that the structural behavior of this type of connections is in between these two extreme cases.

From full-scale fire tests and actual damaged structures it was confirmed that the behaviour of beam-to-column connections used in actual structures exhibit a range of characteristics under the fire loading (Al-Jabri et al. 1998). A lot of tests have been conducted in the past few decades to study the effect of dynamic load and static load on flush endplate connections. However, very little information on their behaviour in fire conditions had been disseminated.

In many cases, experimental work is either not feasible or too expensive to conduct. For fire

testing, the accurate measurement of the structural behaviour is sometimes very difficult to be obtained under fire. Additionally, the number of geometrical and mechanical parameters considered is limited in experimental work. Nowadays, with significant improvement of computational analysis capabilities, it is possible to simulate complicated structures under extreme loadings with finite element analysis for the much improved accuracy.

Finite Element Method (FEM), which is a reliable tool for investigating the effect of different parameters, can provide acceptable and accurate results once the model has been validated. The complicated phenomena such as local buckling, lateral buckling, detailed connection modelling and membranes actions can be accounted for in detail. Therefore, finite element modelling of connections in fires provides a good opportunity for wider parametric consideration and to eliminate the limitations associated with laboratory testing.

Morris and Liu, developed a finite element model to simulate the various types of connections under fire conditions (Liu and Morris 1994, Liu 1999). Sherbourne and Bahaari (1994, 1997) developed a three dimensional finite element model for simulating end-plate connections by using brick elements. Rahamn et al. (2004), investigated the behaviour of fin-plate connections in fire by ANSYS. The entire structures were modelled by four types of element: two types of three dimensional solid elements, pre-tensioning elements and contact elements. More recently, Xiao et al. (2010) created a three dimensional finite element to investigate composite connections and floors under fire. The results obtained have been well calibrated against test results.

2.2 Finite element scheme

A 3-D finite element model of a flush-end plate connection was developed in order to analyse and understand the behaviour of this type of structural connection at elevated temperatures. The commercial finite element software ANSYS was used to carry out the numerical simulation. In order to illustrate the behaviour of steel connections under fire loading, a cruciform bolted flush end-plate beam-to-column steel connection, tested by Al-Jabri et al (1998) was selected. In total, twenty tests (5 Group) were conduct by Al-Jabri et al, in this research paper, only the group 2 was selected as the benchmark for the computing validation.

The flush end-plate connection consists of two 356X171UB51 beams connected to a 254X254XUC89 column using 10 mm thick flush end-plate and eight Grade 8.8 M20 structural bolts. The connection details and 3-D FE mesh is shown in Figure 1. Since the geometry of the connections

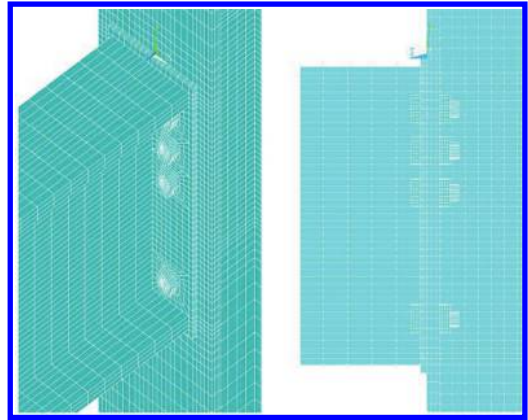


Figure 1. 3D endplate connections FE model.

Table 1. Material properties used for modelling.

Material	Yield stress (N/mm ²)	Ultimate stress (N/mm ²)	Young's modulus (N/mm ²)
S355	355	545	195
Bolts	480	600	195

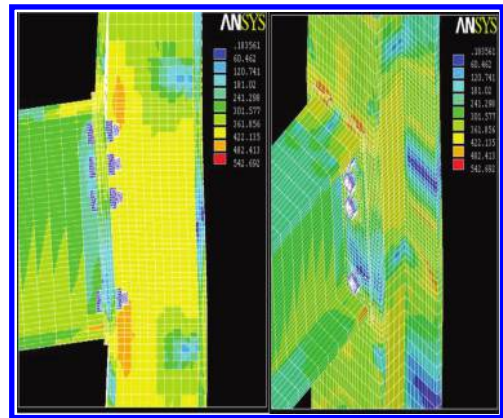


Figure 2. Response of the flush end-plate connection under rising temperature.

is symmetric, just half of the connection was modelled for efficient computing.

This is a comprehensive model accounting for all material non-linearities and contact interaction between each component of this type of connection.

The properties of connection components and the level of loading for tests as shown in Table 1.

The structural response of the flush endplate connections under fire is illustrated in Figure 2.

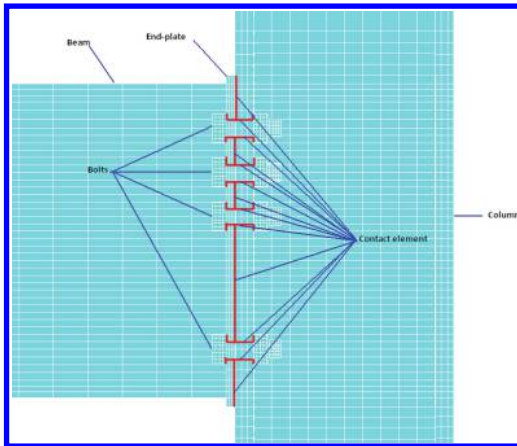


Figure 3. Nonlinear contact modelling of the bolts with endplate and the column flange.

It has been observed that the FE model has well captured the structural failure of the specimens in furnace testing.

This has been achieved by an explicit surface to surface contact element scheme which was developed for the contact surface to transfer the load from the beam to the column. The contact area include bolts shank and hole interface, bolt head and end-plate surface, nuts and column flange surface. Figure 3 shows the contact element distribution by Xiao et al (Xiao & Taufik 2007, Xiao et al. 2010, Taufik et al. 2011).

2.3 Fire loading and boundary conditions

The ISO 834 standard temperature-time curve has been considered in this analysis (ISO 1975). It is defined as the ability of a structure or its components to keep the bearing capacity during a standard fire exposure, for a specified standard period of time such as 30, 60 or 90 minutes. The standard fire exposure is described by an increasing temperature-time curve of the surrounding air as experienced in typical hydrocarbon fires. Further details about these temperature curves can be found in Eurocode (European Committee for Standardization).

$$T(t) = 20 + 345 \log_{10}(8t + 1) \quad (1)$$

where T [°C] is the gas temperature in the fire compartment at time t [min].

2.4 Computing verification

The finite element results were compared with experimental data generated by Al-Jabri

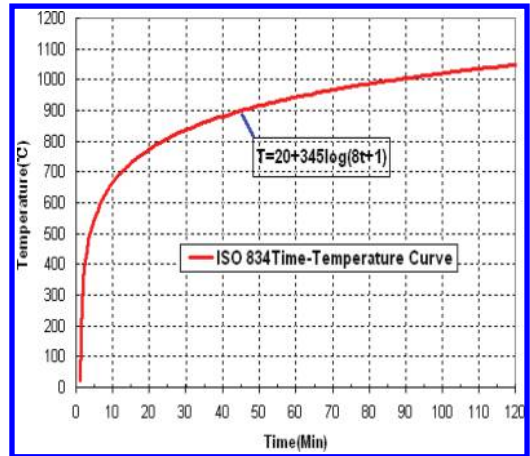


Figure 4. ISO834 standard temperature-time curve.

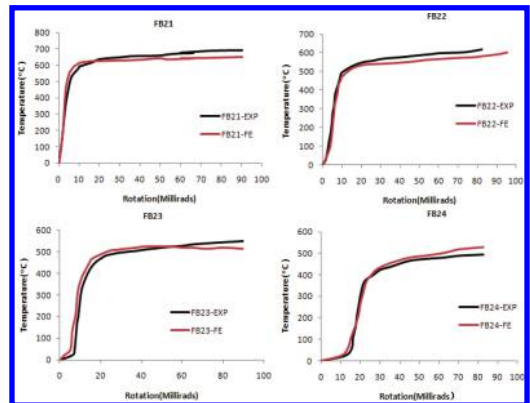


Figure 5. Comparison of finite element analysis and experimental results.

(Al-Jabri et al. 1998). Figure 2 has shown the deformed mode of the connection components. It can be seen there is a deformation at the middle of the column flange, where it is subjected to the highest tensile stresses. There are four elevated temperature tests were modelled, in comparison with of the temperature-rotation response of the connection are shown in Figure 5. The temperature-rotation response curves of the connection closely matched with tests at the elastic and plastic stages. However, the numerical simulations, for example, could easily allow contact surface interactions, frictional forces between each components of the connection and bolts pretension to be included.

This has also been proved by other researchers (Liu 1996). Therefore, this research could easily enhance the knowledge to the analysis of frame performance with the connection behaviour

fully understood. It will be very important for the understanding some of recent fire testing on composite structure.

Symmetric has been used for the efficient computing, only half of the structure was modelled. Therefore all the nodes along the planes of symmetry are restrained from moving in their corresponding perpendicular direction.

3 MODELLING OF COMPOSITE FLOORS

3.1 Composite floor detail

The response of composite floor in fire conditions have been validated by modelling metal decking floor system using finite element technique. The results of the fire test conducted by BRANZ (Building Research Association of New Zealand) by Lim et al 2002 are used for validation.

The composite slab used in the test is measured 3.3 m wide by 4.3 m long. The detailing of the composite slab is shown in Figure 6. The fracture strains of the steel, determined from tensile tests at ambient conditions, the yield stress of the mesh used in the slab was 565 MPa. The concrete compressive strengths are based on cylinder crushing tests carried out at ambient conditions.

Decking slab is 130 mm thick with concrete strength of 35 MPa and reinforcing steel area of 200 mm²/m which is a normal meshing reinforcement for the composite metal decking floor for shrinkage cracking control.

3.2 FE model validation

A 3-D finite element model has been created using ANSYS. Figure 7 shows the finite element mesh for the composite slab tested. The metal decking of the concrete slab was modelled by using Shell 57 element. The concrete slab elements are modelled with Solid 70 element. This element has eight nodes with a single degree of freedom, temperature, at each node. The element is applicable to a 3-D, steady-state or transient thermal analysis.

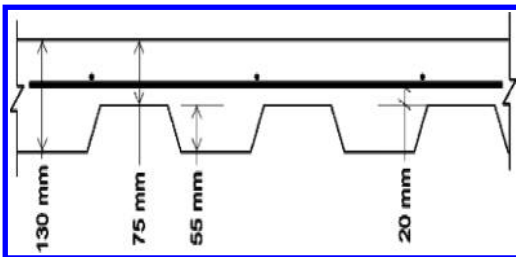


Figure 6. Composite floor detailing.

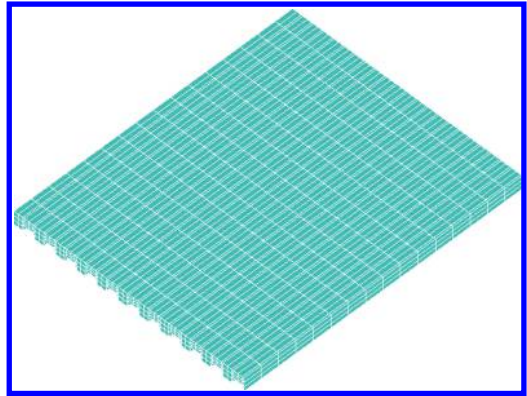


Figure 7. Finite element mesh for the composite floor.

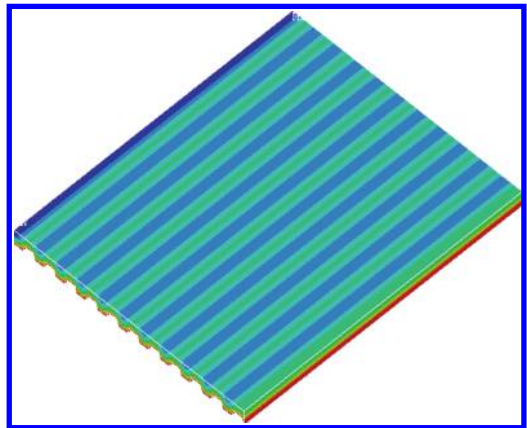


Figure 8. Thermal analysis of the composite floor under fire.

The reinforcing bars were modelled with Link 33 element. All these three elements could be replaced by equivalent structural elements for normal structural analysis.

In the real fire condition, there are two assumptions being taken into account on the structure. The temperature of inner atmosphere of the model will increase equally. So the temperature load will apply on the internal surfaces of the structure. And then, rather than using the natural fire curve, the temperature load will follow the ISO standard fire curve with the values of elevated temperature. Fire is usually represented by a temperature-time curve which gives the average temperature reached during fire in a small size compartment or in the furnaces used for fire resistance test. International standards are based on the standard fire defined by the heat exposure given by the ISO 834 curve (ISO 1975). It is presented in Figure 8 and Figure 9.

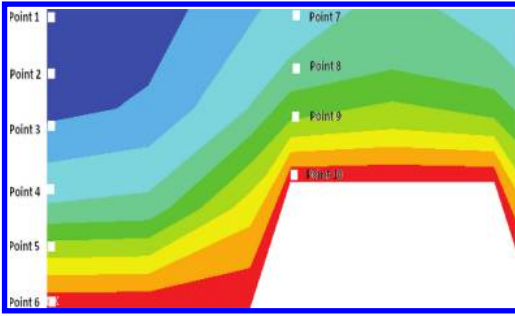


Figure 9. Selection of the key points for the temperature.

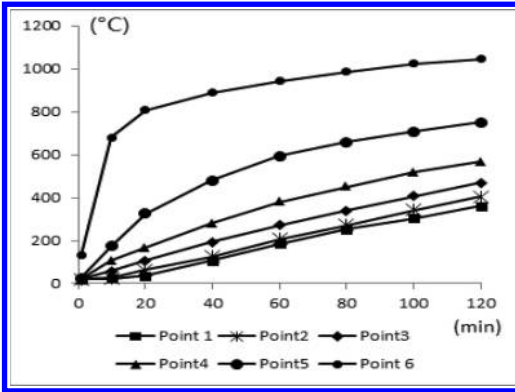


Figure 10. Temperature distribution along the full depth of the floor.

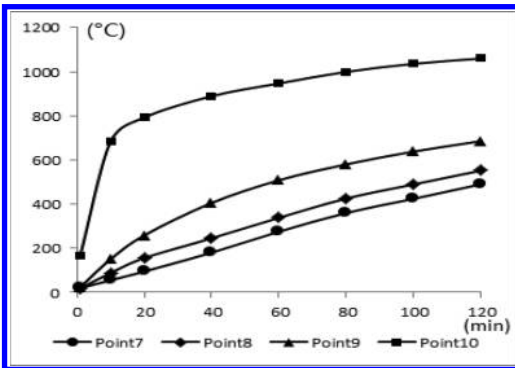


Figure 11. Temperature distribution in the thinner section.

Temperature distributions along the composite slab have been computed for the selected key sections in 2 hours of fire duration specified by ISO fire curve. The temperature distributions have been plotted in Figure 10 and Figure 11 along the critical sections for the key points selected in Figure 9.

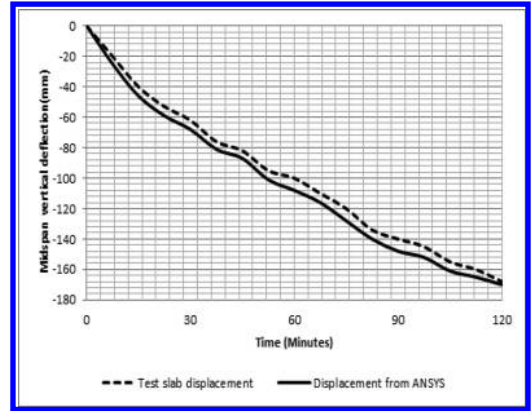


Figure 12. Comparison of the computing and testing results for the deflection in the composite slab.

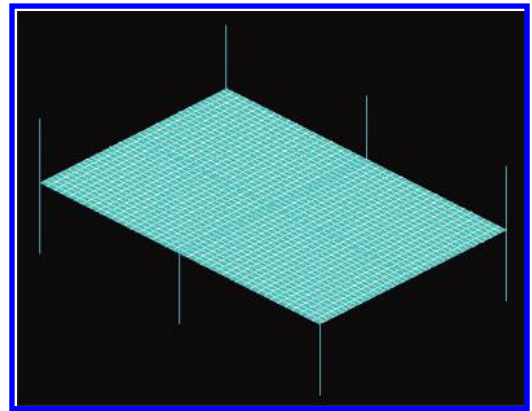


Figure 13. Finite element model of the composite frame.

It can be seen that steel decking reached the critical failure stress in a short period of time while the concrete top surface was much cooler due to low conductivity.

3.3 Coupled-field analysis

For the thermal-structural analysis, the first step is to conduct the thermal field analysis, to get the temperature distribution on the concrete slab, and then it is needed to change the element type and material properties from thermal field to structure field, base on the results of thermal analysis, to put the uniform pressure on the top surface of the concrete. Figure 13 shows displacement from the computing models in comparison with the test conducted in the testing time 7200 s (2 hours).

4 MODELLING OF THE COMPOSITE FRAME

It has drawn a heavy attention from the industry and the research community on the fire safety for steel structures since the collapse of the World Trade Centre (NIST 2005). Significant developments have been made in analysing the behaviour of steel-framed structures under fire conditions in the last 20 years. During the 1990s, a programme of fire tests was completed in the UK at the Building Research Establishment's Cardington Laboratory (Newman et al. 2004). The tests were carried out on an eight-storey composite steel-frame that had been designed and constructed as a typical multi-storey office building. The purpose of the tests was to investigate the behaviour of a real structure under real fire conditions and to collect data that would allow computer programs for the analysis of structures in fire to be verified (SFPE 2004).

In this paper the finite element models previously created for connections and floors discussed have been used to provide an understanding of the behaviour of the steel frame. The finite element modelling on a full structure allows the displacement and stresses of the structure to be predicted during the fire.

The steel frame model in this computer model is measured at 6 m wide by 12 m long. A three dimensional thermal analysis technique using an ISO834 temperature curve is utilised. Figure 13 shows the computer model of the frame.

The model takes into consideration the material properties and the effect of the boundary conditions. Figure 14 and Figure 15 show the displacement and the Von-Mises stress of the steel frame under fire conditions respectively. The validated connection model and floor model have worked efficiently in the full structure model.

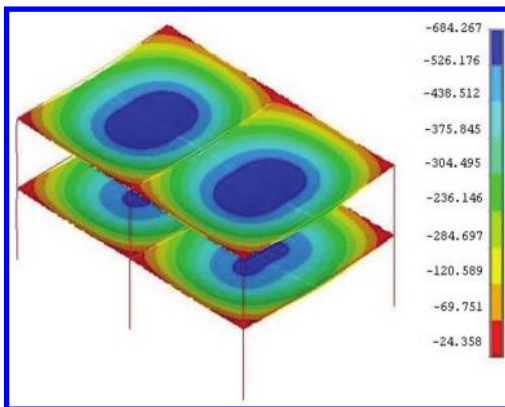


Figure 14. Displacement of the composite frame under fire.

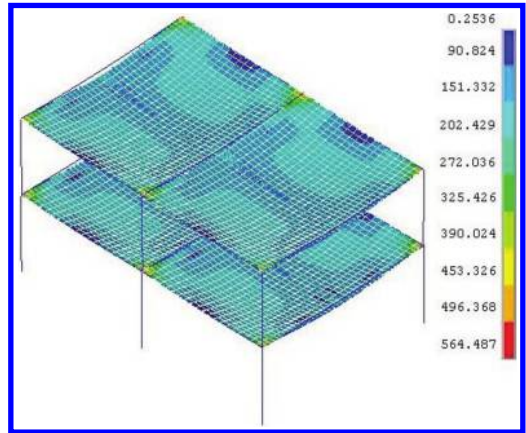


Figure 15. Thermal stresses in the composite floor.

5 CONCLUSIONS

This paper discusses the 3D nonlinear numerical analysis of steel beam-to-column end-plate connections, the composite flooring system and the composite frame under fire conditions using FE software ANSYS. Conclusions can be drawn from the results of finite element analysis with experimental validation under the fire situation:

Connections:

For the beam-to-column connections, all the components of the connection were modelled using solid elements, while contact elements have been used for the contact surface to transfer the load from the beam to the column. The finite element results have been compared with experimental data. The results obtained showed good agreement in both the elastic and plastic stages. The degradation of the connection strength and stiffness with increasing temperatures is well predicted by the model. This demonstrates that the finite element technique is capable of predicting the connection response at elevated temperatures.

Composite Floors:

For the composite flooring system, the thermal analysis indicates that the temperature distribution in the same section of the concrete slab is different along the same plane. The greater difference in temperature on the concrete slab surface will cause greater differences in local stresses distribution. After two hours the temperature of the metal decking increased to 1049°C, but the temperature towards the top of the concrete slab was just around 400°C. A large vertical displacement was found in the composite decking slab. At the beginning of the analysis, the vertical displacement in the midspan of the concrete slab developed slowly. However, it increased significantly by the end of

the two hours fire testing. The results from three dimensional analysis of the composite slab using the ANSYS software have shown good agreement with the fire test results.

Composite frame:

The full finite element models on connections and the composite floor have been successfully implemented to a full scale composite frame analysis. Finite element models have been developed to predict the performance of steel frame. Comparison between the finite element analysis and the test data were reported, which demonstrated the finite element models are able to predict the response of steel frame in the fire situation. It can be concluded that the finite element simulation is able to aid engineers to identify the weakest structural component in the fire situation; it is an effective solution for the complex behaviour prediction in the design of steel frames.

This has provided a robust tool for designing a safer and better composite structure subject to fire.

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Cities: Saviours of the planet?

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ABSTRACT: The drive towards urbanisation is continuing throughout the world. More than half of the global population, some 3.5 billion people, live in cities and this is expected to double by 2050. Cities will have to play some crucial roles. As well as meeting the physical and psychological needs of its citizens, cities will also need to minimise their impact on the environment. Cities will need to become highly efficient resource-users and develop in ways that minimise waste and optimise the use of clean and renewable resources. We do know that cities can be very efficient but a step change in the way cities function is needed. In short they will have to be transformed to meet human needs. Further they will need to reduce their ecological footprint in order to free space for other species to thrive. A diverse planet is a healthy planet. Is this achievable? There are some indications that cities in some aspects are heading towards a transformative path. This article will evaluate these trends and discuss what more needs to be done to transform cities into places that enhance the sustainability of the planet.

1 CITIES

From their beginning cities were places of commerce and manufacture, often developing in locations suited to an economic activity, such as on trade routes or near useful resources. With their concentrated population, diversity of skills and demands cities stimulated growth in the amount and diversity of economic activity and knowledge. This was often reinforced by the consumption patterns of the privileged layers (Taylor 2013). As well as dense population and networks of criss-crossing activities, cities have public buildings and spaces for government, organised religion, education, commerce, social interaction, cultural events and public services. These buildings and spaces play an important role in providing a focus for citizens and communities. They provide emotional attraction for both citizens and visitors, embodying political and cultural activities and significance, and landmarks in time and space. They link the past, present and future, become reassuringly familiar to local people and stimulating for visitors. Lozano (1990) argues that the city is a realm with a high level of culture linked with the most civilised expression of social behaviour. Lewis Mumford in *Culture of Cities* (1970) described the city as humanity's "greatest work of art" while Levi Strauss (1955) stated, "it is the human invention, par excellence." Despite the many problems that cities face such as urban sprawl, inappropriate development and the scarring of major road networks, many city residents identify strongly with their place of residence and the cultural landmarks it contains (Giddings et al 2005).

Though cities play a positive role in human development, they can also be places of fear and isolation. Crime, violence, exploitation and inequality are also features of cities. Cities were often (and still are) places where disease was rampant. Cities are places that can be highly polluted. Though we cannot always see pollution we do know that cities consume vast amount of resources and produce high levels of waste and pollution. According to UN-Habitat (2013) cities now house more than half of the world's population. Cities consume some 80 per cent of global energy and are responsible for the emission of more than 70 per cent of greenhouse gases. Cities also produce more than 70 per cent of the world's waste. Between 30 to 40 per cent of urban dwellers in developing countries do not have access to modern forms of energy such as electricity and clean cooking fuels. Though there has been an increase in sanitation coverage in the developing world, some 2.5 billion people still lack access to improved sanitation facilities.

One of the most important changes in cities began during the Industrial Revolution. Prior to the Industrial Revolution more people lived in rural areas than cities which reflected the agrarian nature of the global economy. The Industrial Revolution brought about a complete shift in the economic underpinnings of the global economy. Fossil fuels replaced biomass as the primary source of fuel. Technological innovation meant that humans began to change their relationship with nature. We moved from stewardship to mastery of nature. Exploitation of minerals, metals and chemicals led to new and innovative industrial process to satisfy

growing consumers demand. Improvements in public health led to decline in mortality and increases in fertility. Populations began to expand. There was a growing move from the rural to the urban as agricultural technological innovations meant that less agricultural labour was needed, whereas as new industrial processes required more. In the industrialising nations rural populations began to migrate to the city.

Prior to World War II little consideration was given to the environmental impacts of cities and the growing number of industrial processes. But this began to change after World War II. The Great Smog of London in 1952 led to the introduction of legislation, the Clean Air Act, which set standards for air quality. Scientists such as Rachel Carson (1962) pointed out the dangers of the unregulated use of pesticides. Her book, *Silent Spring*, proved to be very influential and she is credited with being the founder of the American Environmental Movement and for laying the foundations for the Environmental Protection Agency. In 1966, Barbara Ward (1966) published *Spaceship Earth* and is sometimes said to have coined the term Sustainable Development. She and Rene Dubas (1972) were co-authors of *Only One Earth: The Care and Maintenance of a Small Planet*. This was written for the 1972 UN Stockholm conference on the Human Environment. The Stockholm Conference made the link between development and environmental degradation and introduced environmental concerns into the formal political sphere.

In 1987 the Brundtland Report or *Our Common Future* was published by the United Nations World Conference on Environment and Development. This report defines Sustainable Development as development that meets the needs of the present without compromising the ability of future generations to meet their own needs (Brundtland 1987).

There has been considerable international debate on how to interpret the term Sustainable Development. There are two positions that have emerged on the meaning of Sustainable Development. One position has an anthropogenic focus and believes that technological processes can be a substitute for natural processes. The other has an ecocentric focus and argues that we must work with natural processes. What is common between these positions is that we must protect the environment, have an efficient and clean economy and cohesive societies. How this will be realised is contested, with some believing small incremental changes are needed, whilst others believe that either reform or transformation is needed if we are to realise sustainable development (Hopwood et al. 2005). In the definition for Sustainable Development one of the important words used is 'needs'. What seems

to be the case is that the concept of Sustainable Development should be focused on promote human well-being. In doing so it should also promote environmental well-being and sustainable livelihoods. Whilst there is no absolute model of needs for human well-being, work done by Maslow (1954) and Max-Neef (1991) have identified two broad areas that influence human well-being; physiological and psychological. Physiological needs include shelter, comfort, security and safety. Psychological needs include community interaction, attachment, identity, status, self-esteem and attractiveness of surroundings. Sustainable cities must therefore address both physiological and psychological needs.

1.1 *Physiological needs*

A sustainable city must have clean water and air and access to secure high quality food supplies. Good quality and affordable accommodation is also required. It is important that citizens feel that their physical needs are met where there are physical places for leisure and interaction. This will contribute to a sense of security. Also connectivity through transport and multi-media communications systems is needed along with a variety of sustainable livelihood options. In short a sustainable city has to meet our physical needs.

1.2 *Psychological needs*

Meeting human needs starts at birth where the relationship with parents is so important. Throughout life relationships will be important and the sustainable city will need to devise ways of ensuring we have a range of relationships that build a sense of belonging and togetherness. Underpinning this will be education and life-long learning and leisure time that will help people to express in different ways.

2 REPRESENTING SUSTAINABLE DEVELOPMENT

Sustainable Development is typically represented as the area where 3 circles, each representing environment, economy and society overlap as shown in [Figure 1](#). In reality there is constant interaction between these sectors. For example, food is fundamental to sustaining us. Food is produced in the environment and its production will have environmental impacts. Some argue that we should develop food production methods that minimise environmental damage, for example, organic agricultural methods. On the other hand some argue that we need to techniques such as genetic modification in

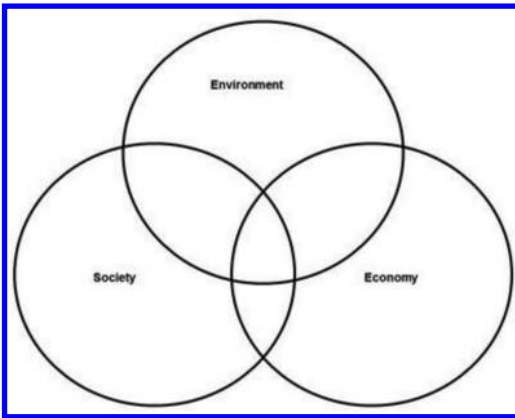


Figure 1. A model for sustainable development.
Source: Author.

order to feed the growing world population and to have crops that are more tolerant of changing climate conditions. Others argue that we should fundamentally change our diet, for example, by not eating meat as meat production is a very inefficient way of producing protein. Further, there are ethical issues that need to be addressed. The production of food has an impact on the economy as products produce receipts and wages. Throughout the food chain economic transactions take place right from food preparation to consumption either in the home or restaurants. There is also a very strong social dimension to food. Meals are a common vehicle for families and friends to get together either in the home or a restaurant.

The food example shows the interrelationships between the different sectors of sustainable development. A decision made in one sector can have unforeseen consequences in another, for example, the introduction of inorganic fertilisers led to the contamination of water courses and aquifers that impacted both human and aquatic health. Rachel Carson vividly pointed out in her book, *Silent Spring*, the impact of DDT, a synthetic pesticide used on farms, on wildlife.

3 IMPLICATION OF SUSTAINABLE DEVELOPMENT FOR CITIES

So what does Sustainable Development mean for cities and those that live in them? First, cities are very demanding and have a large influence on the environment. Cities impact their immediate environment as well as the global environment. The demand for fuel, food and water just to feed and power our cities is growing rapidly and this will continue as both the human population grows and

we continue to urbanise. The UN estimates that the human population could be near to 10 billion by 2050. Steven Emmott (2013) in his book *10 Billion* paints a bleak picture of resource needs and impacts, for example, some 3 000 litres of water are required to produce one hamburger, and argues that the demands of a growing population may outstrip the capacity of the ecosystem. There is considerable resource use and waste production in our consumerist lifestyles, but it is possible to minimise impacts. One major consideration of a sustainable city is precisely that—minimising impacts. For example, in Europe, buildings account for some 40 per cent of energy use with most of that use aimed at thermal comfort. The Passive House Institute has produced a building specification that means little energy is needed for thermal comfort down to temperatures as low as minus 20 degrees Celsius. Good building design can help to promote effective passive cooling. The energy system comprises two parts; the supply and demand sides. The major focus of energy policy is the supply side. However, the sustainable city should address the demand side through more efficient technologies. Further the city should become an energy producer. For example, advances in photovoltaics and battery technology can enable the city to generate power (O’Keefe et al. 2010).

Other ways in which the city can reduce its impact is through vertical farming. Paul Marks reports in *The New Scientist* on the growth of vertical farming (Marks 2014). Vertical farming could improve food security as production can continue even when extreme weather strikes. And as long as farmers are careful to protect their indoor “fields” from pests, vertical farming needs no herbicides or insecticides. They also conserve water far better than conventional farming. Vertical farming can reduce the amount of land needed for food production significantly. A thirty story building with a 2 hectare base would yield an annual crop equivalent to that of 1000 hectares of conventional farming (Despommier 2009). It is also possible to combine fish farming with salad and herb production where the fish provide the nutrients for the plants and both can be consumed (GrowUp Urban Farms 2014).

4 GOVERNANCE AND THE SUSTAINABLE CITY

There are many challenges in developing a sustainable city. One challenge is getting the balance between sectors. There may be areas where compromises are needed. Arriving at compromises will require forms of governance that are transparent and understood. The other challenge is to

transform existing cities on a more sustainable trajectory. Though there are examples of new cities that are being developed from a sustainable perspective, for example, Masdar in Abu Dhabi. However, it must be recognised that we will have to refurbish existing cities along sustainable lines. We cannot build everything from new!

This is an enormous challenge. Many of our cities are sprawling monstrosities that are places of production and consumption. They require enormous resources and produce huge amounts of waste and pollution. In short they act in a linear fashion drawing resources in and pushing wastes out as shown in Figure 2. This is not the way that nature functions.

In reality the impact of cities spreads far beyond their boundaries. This is termed the ecological footprint. An ecological footprint measures the land and sea area required to produce the resources we consume. This includes food, clothing, energy and materials for buildings and industry. It also measures how much land and water is required to deal with the waste products of our consumption, such as carbon dioxide and agricultural pollution. Research on the ecological footprints of cities, regions and nations show that around 1985 the biological capacity of the earth was exceeded (Wackernagel et al. 2006). According to the Global Footprint Network we currently require 1.5 planets to meet our needs and by 2050 we will require 3 planets (Global Footprint Network 2015).

There are some who argue that the consumerist model that so dominates the global economy is part of the problem. Many countries have exceeded their ecological footprint and in thinking about a sustainable city then thought will need to be given to the type of economic model it should have. Since the global financial crisis in 2008 the focus of many OECD nations has been on the economy. A report by Credit Suisse shows the growing wealth inequality across the world with the richest 1 per cent of the world's population are getting wealthier, owning more than 48 per cent of global wealth (Credit Suisse 2014). Such stark distinctions do little to foster social cohesion. Harvey (2014) argues that

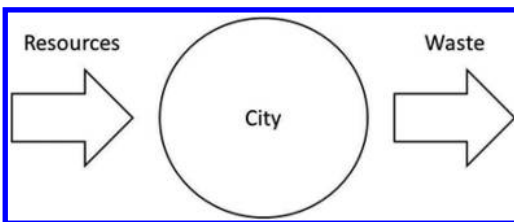


Figure 2. City linear metabolism. Source: Author.

we need a fundamental change in the type of capitalism we use as well as changes in lifestyles by simply slowing down. He cites a number of changes that are needed if we are to move towards a fairer and more sustainable future:

“15. The economy converges on zero growth (though with room for uneven geographical developments) in a world in which the greatest possible development of both individual and collective human capacities and powers and the perpetual search for novelty prevail as social norms to displace the mania for perpetual compound growth.

16. The appropriation and production of natural forces for human needs should proceed apace but with the maximum regard for the protection of ecosystems, maximum attention paid to the recycling of nutrients, energy and physical matter to the sites from whence they came, and an overwhelming sense of re-enchantment with the beauty of the natural world, of which we are a part and to which we can and do contribute through our works.” (Harvey 2014:296)

Not only do we need to re-think the economy of the sustainable city we need to think about the way in which businesses operate. Pavan Sudkhev is an environmental economist who was, amongst other roles, the Special Adviser and Head of UNEP's Green Economy Initiative, a major UN project suite to demonstrate that greening of economies is not a burden on growth but rather a new engine for growing wealth, increasing decent employment, and reducing persistent poverty. In 2012 Sudkhev stated that:

“The rules of business urgently need to be changed, so corporations compete on the basis of innovation, resource conservation and satisfaction of multiple stakeholder demands, rather than on the basis of who is most effective in influencing government regulation, avoiding taxes and obtaining subsidies for harmful activities to maximise the return for shareholders.” (Sudkhev 2012:27).

A report by the Cambridge Institute for Sustainability Leadership (CISL) argues that the world's current course has revealed a monumental market failure and that there appears to be a strategic chasm between where the world agrees it should be headed and the direction of the economy. The report makes a series of recommendations that would affect business, include raising green taxes, requiring companies to reveal their environmental impacts and ending the damage caused by short-term profit-seeking (CISL 2015).

This article does not argue that the precursor to the sustainable city is a radical change in governance and how its economy is organised. Cities will have to operate within the rules and regulations that govern the place where they are located. But it is worth noting that new trends are

beginning to emerge. For example there has been growth in ethical/organic shopping and sales of Fair Trade products as well as growth in cooperative ventures. Many companies have adopted policies of Corporate Social Responsibility (CSR) and environmental management systems. Some European governments have actively encouraged the deployment of renewable energy technologies with the offer of generous tariffs. But there is one clear message there is no single solution or model for a sustainable city. What we can reflect on is the determinants of the sustainable city.

5 PHYSIOLOGICAL AND PSYCHOLOGICAL NEEDS

Earlier we looked at ways providing energy and food to meet some of our physiological needs. This raised two issues; embedding energy technologies in the city fabric and vertical farming. Meeting other needs such as accommodation, leisure area, meeting places and so on will have a profound impact on the design of buildings. As rule of thumb the more compact a city is the more efficient it is. This implies that, as with farming, a more vertical approach is needed. Tall buildings are common throughout our cities. However, there is very tendency towards a mono-culture in the design of buildings; that is each building is built for a single function—residential, office and so on. Buildings can be multi-functional and include residential, commercial, leisure functions as well as green areas, meeting places and communal gardens for residents to manage. Building design can draw upon traditional and vernacular architecture.

Though each city will be shaped by its location, environment, topology, economy, politics and culture there are a number of determinants/characteristics that sustainable cities should have in common. In terms of resources they should be able to capture energy and water. They should be designed to maximise recycling, including human waste, they should be energy efficient and, wherever possible, the building materials should be sustainable and recyclable. Buildings should be designed to be easily repairable and maintainable. They should be designed to be flexible so that changes can easily be made. It is important to recognise that sustainable development is not an outcome but a process. Likewise sustainable cities are also a process and need to be inherently flexible so that they can respond to new challenges. Issues such as accessibility, walkability and comfort are important needs to be factored into the design as well as the appropriateness of both public and private spaces. Consideration will also be needed of the relationship to the natural world as this appears to

have a significant impact on well-being. A recent US study found that being close to nature might soothe the mind by reducing rumination—when negative thoughts get stuck on repeat, playing over and over in the mind (Bratman et al. 2015).

A key issue in the design of the sustainable city is the ability to deal with the threat of accelerated climate change and increasing variability. Broadly there are two categories of the threat. First, there are rapid onset events such as storms, floods, wild fires, heat waves and epidemics. Second, there slow onset events such as sea level rise, drought and loss of biodiversity (IPPC 2014). If, for example, a new city is being planned then careful consideration of the location and the prevailing environmental conditions will be needed. For existing cities then refurbishment work will need to factor in climate events. If, for example, records show an area of an existing city is prone to hazards then it could be abandoned or a radical new approach be adopted. For example, about one third of Holland is below sea level. Three major rivers, the Rhine, the Meuse and the Scheldt, pass through Holland on their way to the North Sea. About one sixth of Holland is covered by water. In 2006 the Dutch authorities signalled a new approach to flood management entitled “Room for the River” programme that addresses flood protection, landscaping and the improvement of environmental conditions in the areas surrounding Holland’s rivers. In some cases this has meant people have had to relocate to raised areas so that flooding can occur (McVeigh 2014). Other innovative ideas include floating homes. These are allowed to rise or fall on a mooring. Utilities are supplied to a dwelling by a flexible connector. Of course it should be recognized that there are a number of other hazards that can threaten a city such as earthquakes, volcanoes and tsunamis and these will also need to be considered.

A further issue to consider is compactness. A compact city reduces the need for transport within the city. Where transport is needed then it is likely that autonomous non-polluting vehicles will be available in the near future. Some argue that private vehicles will not be needed and a fleet of autonomous taxi-like vehicles could meet needs. Transport connections between cities are important. Multi-media communication systems provide the opportunity to reduce the need to travel and effective communications systems will be an integral part of the sustainable city. Where inter-connection between cities is needed then the most efficient means should be used. At present High Speed Trains (HST) are proving to be efficient and can lower greenhouse gas emissions if the electricity used is generated from fossil fuels. However, the infrastructure impacts are likely to be higher than conventional trains. But there is agreement that

HST has a much lower impact than automobiles and aircraft (Tutton 2011). That still leaves the issue of aircraft. For long haul journeys aircraft are the only viable option. But in reality we need to encourage the use of communications system to minimize the amount of air travel. Though aircraft manufacturers have made considerable progress in improving aircraft efficiency the impact of air travel still remains high (O'Keefe et al. 2010).

5.1 Psychological

Psychological needs are as equally important as physiological needs. As stated earlier a livelihood can be considered as a physiological need. However, it is difficult to be prescriptive as earlier we saw that a major shift in the ways in which we live is needed. In thinking about sustainable livelihoods Harvey (2014) cautions us to slow down. There are many who believe that the pace of modern life is too fast and despite the development of so called labour saving devices we seem to spend longer each day at work. And there is concern about the relentless to grow economies and increase consumption in what is a finite world. And there is little evidence to suggest that a consumerist society promotes wellbeing. Kasser (2003) identified a connection between an excessively materialistic outlook and increased levels of anxiety and depression, while Putman argues we're paying the ultimate price for our consumeristic tendencies with the loss of friendships, neighbourly support and robust communities. Together they suggest we are witnessing nothing short of the collapse of social fabric across society. Pieters (2013) conducted a 6 year study of 2 500 consumers which found that materialism formed a self-perpetuating vicious and virtuous cycles depending on the materialism sub-type; valuing materialism as a measure of happiness or success increased loneliness whilst materialism for mirth decreased loneliness.

The impact of this relentless push for more consumerism is having a significant impact on global ecosystems. The Worldwide Fund for Nature WWF (2014) paints a bleak picture of the state of the planet at present stating that at current consumption levels we would need 1.5 planets to regenerate current demands. If everyone on the planet had the same lifestyle of the average American then we would need 3.9 planets!

In terms of the sustainable city much thought will need to be given to how it functions. In terms of the shape of business Sudkhev (2012) argues that innovation and resource conservation are really important. The ways in which business is conducted is likely to be very different in the sustainable city. This has very strong implications for governance. Again it is difficult to be prescriptive

over the model of governance for the sustainable city and it is certain that there not be one model. It is likely that local custom and practice along with existing laws and regulation will be the main determinants as was discussed earlier. However the thrust of policy and governance must be to reduce impacts if our cities are going to be sustainable.

In 1968 Paul Ehrlich published a book entitled *The Population Bomb* (Ehrlich, 1968). Though this mainly focused on population growth Ehrlich produced an equation to show the impact of a growing population that established the relationship among four factors: Population (P), Affluence (A), Technology (T), and environmental Impact (I). The relationships are expressed in the Ehrlich impact equation: $I = P \times A \times T$. Though there are some limitations to the equation it is generally recognised that it is a robust way of looking at impacts (Fischer-Kowalski & Amman 2001). At first sight it does appear that with increasing population and affluence then impacts will continue to grow. But how can this equation help us in moving to a sustainable future? First as said earlier it is expected that the global population will stabilize in the future. We will then know the size of population. It is estimated that some 70 per cent of the global population will live in cities. The sustainable city is about promoting human well-being by addressing both physiological and psychological needs. The term A for Affluence can be re-thought as a means to an end, and as opposed to an end in itself. In short we can use affluence to build sustainable cities where our needs are not focused on consumerism. Through societal changes and priorities that are implied in the concept of the sustainable city we can increase happiness and well-being without more and more affluence.

The next challenge is the term T for Technology. Technology means how we gather and transform resources. As argued earlier many of our technologies are very inefficient. But for a sustainable city a major goal is to reduce resource consumption through such methods as embedding renewable energy capacity in buildings, reuse and recycling, closed loop industrial process and clean production methods.

By reducing the needs to live affluent consumerist lifestyles and radically changing technologies then we can reduce impact of our cities. Our history tells us that things can change very radically and rapidly. The Industrial Revolution brought about huge changes. More recently the internet and smart phones have completely changed the ways in we work, live and communicate. Tony Seba in his recent book argues that the fossil fuels will be obsolete by 2030 and that all cars will be autonomous and the size of the fleet will shrink dramatically (Seba 2014). Seba shows that

technological changes can quickly diffuse through society and that by creating the right of policy and regulatory frameworks then transformation becomes the norm.

6 CONCLUDING THOUGHTS

Realizing the sustainable city is a considerable challenge. Whether it is moving existing cities to a more sustainable trajectory, as is the case in Frankfurt and Oslo, or developing new cities, for example, Songdo and Masdar, then transformational changes will be needed. By transformational change is meant the process of altering the basic elements of social culture, including the norms, values, and assumptions under which the city functions. It is also about transforming the economic function to a fairer and more equitable model that takes into account the externalities of products and processes. It will also mean transforming technological processes to be more innovative and less damaging to the environment. In short we will have to hugely reduce the environmental impact of cities. This is not a technological challenge but a political and social challenge. A sustainable approach to cities means that we must improve the well-being of people and that will require transforming the ways in which we live.

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Sustainable concrete: Material and structures

Application of acoustic emission testing to crack detection in Sprayed Fibre Reinforced Concrete tunnel linings samples

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ABSTRACT: The application of sprayed Steel Fibre-Reinforced Concrete (SFRC) in tunnel lining design is a growing trend owing to its great advantages. However, the performance characteristics of this material as well as the design procedures are not properly served by design codes, hence, potentially limit the development of SFRC in the tunneling industry. A project is in progress at the University of Southampton, UK, which mainly aims at investigating the early-age crack behaviour of SFRC. Short-term four-point bending tests have been carried out on four SFRC beam samples. A non-destructive testing, Acoustic Emission (AE) technique has been used to observe the first cracks, crack locations and crack widths. Damage derived from AE results show very high consistency with the actual damage observed by human eyes which further proves the feasibility of applying the AE technique in determining concrete damage.

1 INTRODUCTION

Nowadays, concrete is one of the most common and reliable materials in the construction industry. Concrete usually performs well under compression. Nevertheless, there are some typical weaknesses. For example, concrete is usually very brittle in tension under low confining pressure (Arnon, 2007). Therefore, Fibre-Reinforced Concrete (FRC) which contains fibre reinforcing material is utilised to increase the structural integrity.

Although Sprayed Fibre-Reinforced Concrete (SFRC) makes a great progress in improving the strength and durability of concrete (CHANH 2011), there is another shortcoming of almost all types of concrete materials, which is that they experience micro-cracks before they can be observed by human eyes directly, and it is always too late to discover the observable cracks. Consequently, the micro-cracks can cause potential damage and it is preferable to fix the problem before it becomes too severe.

Acoustic emission is defined as transient elastic waves within a material (Kaphle 2012). As the result of the external forces, the internal stress distribution of a material changes suddenly. The mechanical energy is converted to sound energy which produces elastic waves which are known as Acoustic Emission (AE). The frequency of AE is usually between 1 kHz and 1 MHz (Hellier 2001). Because of the versatility of AE, it has been extensively utilized in various situations such as examining the defects and fatigue of a component, monitoring the welding and corrosion process, and

some metal manufacturing process including hardening, forging and extrusion (Hellier 2001).

2 EXPERIMENTAL METHODOLOGY

2.1 Testing equipment

The beam samples were cut from panels built from a sprayed primary layer, a sprayed waterproofing membrane layer and a sprayed second layer (Su & Bloodworth 2014). Steel fibre dosage of 35 kg/m³ was used in the concrete mix. The dimensions of the beams are 150 mm deep, 150 mm wide and 900 mm long with a 9 mm thick membrane near the top surface (see Fig. 1).

All three beams were tested under flexural loading profiles through a short-term four-point bending test as revealed in Figure 2. Two roller supports were placed 50 mm away from each end of the beam to give an 800 mm span. Loadings were applied at 133.5 mm distance from the centre of the beam to leave a 267 mm pure bending area. All beams were designed to fail in the bending mode.

The AE data acquisition system was a 15-channel system produced by Vallen Systeme GmbH used



Figure 1. Beam sample.

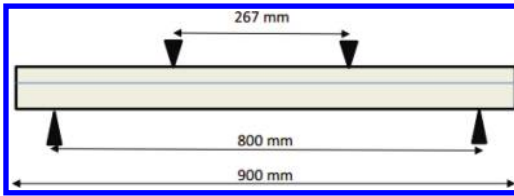


Figure 2. Configuration of four-point bending test.



Figure 3. Four AE sensors and preamplifiers used in the experiment.

with four Pancom P15 acoustic sensors of flat frequency ranges from 110 kHz to 1 MHz (see Fig. 3). Visual VallenAE software was used to display the data. The upper limit of amplitude this software can deal with is 95 dB.

Rough beam surfaces were polished first. Then hot liquid glue pressurized from a glue gun was used to adhere the sensors against beam surfaces, with the sensors held tightly for 30 seconds to squeeze out the air trapped at the interface, therefore ensuring good acoustic continuity.

To determine the crack location in the horizontal plane, the sensors were placed on two side surfaces in test 1 and 2 (see Figs. 4 and 5). Since this experiment only involved a 2D analysis, the height of the sensors was not accurately measured but was generally on the same level, approximately 10 mm below the membrane.

Test 3 was designed to orientate the vertical crack location and examine the degree of composite action between the membrane and concrete. Although the beam contains two layers of concrete and one layer of membrane, these three layers are expected to work together as one object, and the cracks grow from the bottom to the top. The beam is not designed to work in two separate parts; if so, the cracks should happen almost simultaneously on both the upper and lower parts. Thus, four sensors were placed with two on the top surface and two on the bottom surface as showed in Figure 6. Locations of sensors in all three tests are summarized in Table 1.

A digital deflection gauge (also called potentiometer) as well as a mechanical dial gauge was

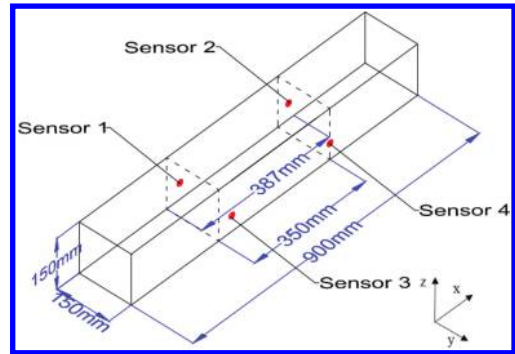


Figure 4. Location of sensors of beam 1.

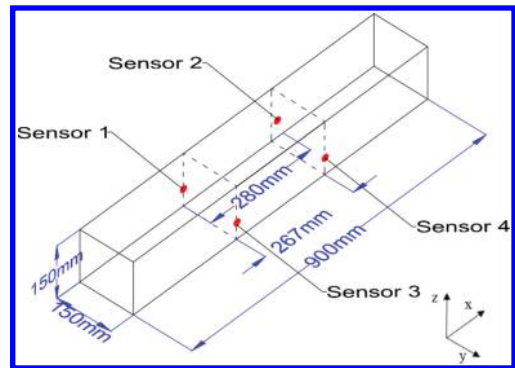


Figure 5. Location of sensors of beam 2.

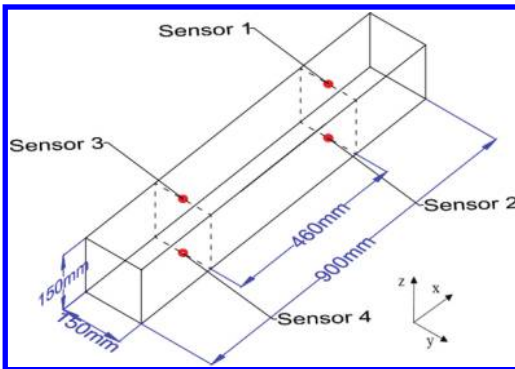


Figure 6. Location of sensors (beam 3).

placed on both the left and right hand side of the beam to measure the mid-span deflection of the beam (see Figs. 7 and 8).

2.2 Loading profile

After the sensors were stuck on the beam, they were connected to the preamplifiers (see Fig. 3 right).

Table 1. Sensor locations.

Sensor no.	X (mm)	Y (mm)
1	0	0
2	387	0
3	0	150
4	350	150

Sensor no.	X (mm)	Y (mm)
1	0	0
2	280	0
3	0	150
4	267	150

Sensor no.	X (mm)	Z (mm)
1	460	150
2	460	0
3	0	150
4	0	0



Figure 7. Testing equipment of beam 1 and 2.



Figure 8. Testing equipment of beam 3.

It was very important to set proper parameters for the data acquisition system. Acquisition threshold was a trigger value which when the output signal exceeds, the AE system began to record the data as a hit. This value was set to reduce the effect of

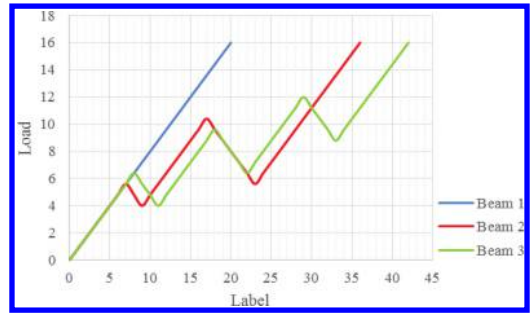


Figure 9. Ideal loading profiles for three beam tests.

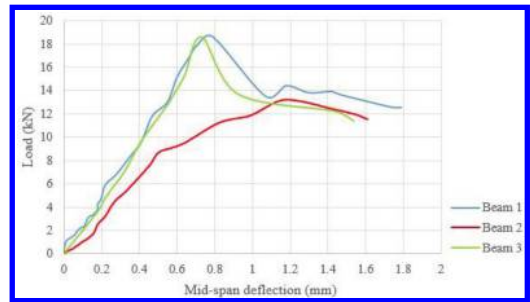


Figure 10. Simplified load-displacement graph.

noise in the surrounding environment. In these beam tests, all threshold values were set quite low, as 34 dB to avoid missing weak signals.

Stroke control was used for loading in this experiment. Ideal load profiles are presented in Figure 9.

SFRC deformed under sustained loading. This deformation involved the instantaneous elastic strains when the load was applied and also the time-dependent strains caused by creep (relaxation). Therefore, the applied load under “stroke control” tended to drop in when the beam was held in the exact same location. Thus, each deflection step could be treated as a small cyclic load process. Since this experiment mainly focused on the very short-term response of the specimen, peak loads at every load increment were picked out to produce the following simplified load-displacement diagram, Figure 10.

It can be seen that beam 1 and 3 have similar peak loads of 18.6 kN and 18.57 kN with corresponding mid-span deflections 0.78 mm and 0.73 mm. Beam 2 has a comparatively smaller peak load of 13.2 kN but with a larger deflection of 1.14 mm.

3 AE SIGNAL PROCESSING

Figures 11 and 12 represent the amplitudes of all AE signals received by the four sensors during the beam tests 2 and 3 experiments.

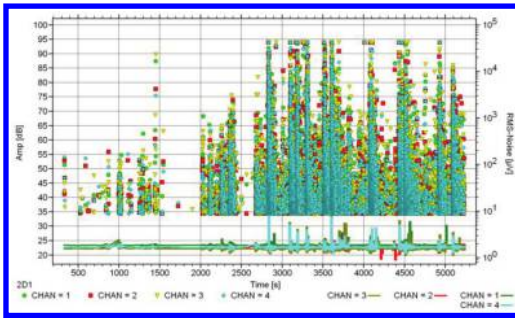


Figure 11. AE amplitude versus time graph of beam test 2.

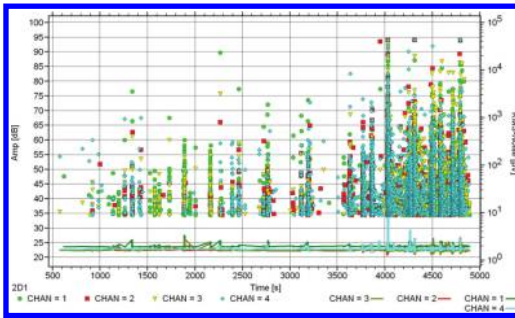


Figure 12. AE amplitude versus time graph of beam test 3.

In Figure 11 there are two periods with virtually zero AE activity, in time ranges 1500s–2000s and 2400–2500s. Figure 9 shows that the beam test 2 load profile contained two cycles before peak load. Based on the Kaiser effect (Hellier 2001), AE resulting from plastic deformation is irreversible. Facts in Figure 11 and 12 both prove that when first decreasing then increasing the load, there was few AE signals until the load almost exceeded the previous maximum load.

It is also apparent from Figure 11 and 12 that only a few AE signals are generated at the beginning. With the increase of load (time), AE signals gradually become denser and stronger. A large number of high amplitude signals imply the generation of a large crack or a large number of minor cracks. In this analysis, it is suggested that the first occurrence of a Located Event (LE) represents generation of the first crack in the beam. On the basis of the data output (see Table 2), the first located AE events in the three beam tests happened at 90%, 66% and 65% of peak load respectively.

However, the first beam test used larger load steps than the others (even +0.16 mm) and as a result the data is not persuasive as there were not enough data points when approaching the

Table 2. Comparisons between load of the first Located Event (LE) and peak load.

Beam number	Load of first LE (kN)	Peak load (kN)	% of peak load
1	16.7	18.6	90%
2	8.7	13.20	66%
3	12	18.57	65%

Table 3. No. of hits before and after the peak load.

Beam no.	No. of hits		
	Before peak load	After peak load	Total
1	2123	17632	19755
2	37825	41674	79499
3	2230	14117	16347

Table 4. Energy released before and after the peak load.

Beam no.	Energy released (eu)			Energy released per hit (eu)
	Before peak load	After peak load	Total	
1	43197	649927	693124	35
2	1874626	1601088	3475714	43
3	62595	562613	625208	38

peak load. The figures from beams 2 and 3 lead to the conclusion that the first crack appears when the load reaches around 66% (2/3) of the peak load.

Table 3 illustrates the number of hits and energy released before and after the peak load. The total number of hits and energy released during beam test 2 is far greater than for beams 1 and 3. The reason for this may be the presence of pre-existing cracks in the beam. Thus, when load was applied, cracks grew on the original cracks. This also explains why the peak load of test 2 is only 13.2 kN.

Table 4 shows the energy released per hits. There is no significant difference between the tests. This proves the reliability of the tests. Although test 2 has more hits and energy released, the average crack event size in beam test 2 almost equals that in beam tests 1 and 3.

4 CRACK LOCATION

The crack could not be visualized by human eyes until it reached the peak load or one deflection

step before. Once the FRC beam cracked, the crack noise can be heard. The AE output graph clearly shows the crack location along the beam (the x -direction). Taking beam 2 as an example, as can be seen from Figure 13, the maximum number of hits takes place at 8–11 cm from sensor 1 and 3. Comparing to the crack location observed visually (see Fig. 14), the major crack location is 10 cm distance from sensor 3 which is consistent with the AE diagram.

Figure 15 represents the distribution of hits over the cross section of the beam (y -direction).

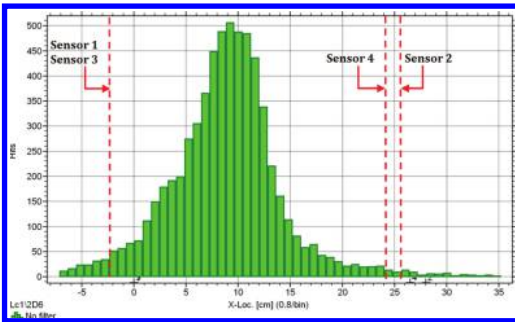


Figure 13. Crack locations (x direction) of beam 2 detected by AE system.

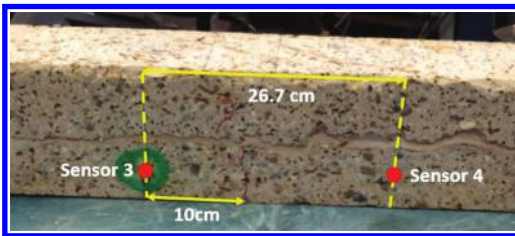


Figure 14. Crack locations (x direction) of beam 2 observed visually.

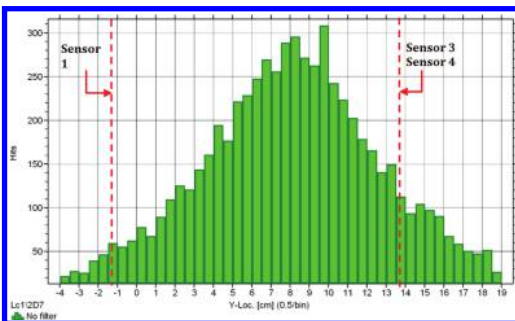


Figure 15. Crack locations (y direction) of beam 2 detected by AE system.

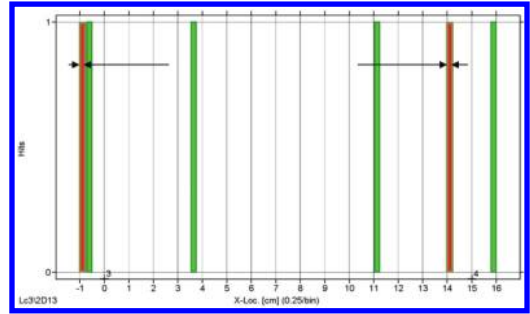


Figure 16. Hit locations diagram (z -direction) during pencil calibration of beam test 3.

It can be seen that most hits happen in the middle of the beam while fewer hits happen at each surface.

In beam test 3, the vertical position of a crack (*i.e.* z -direction position) can be located. The hit locations were supposed to be at 0 (top surface) and 15 cm (bottom surface) where the red bars stand. However, the actual locations detected, the green bars, were a little bit off the scale (see Fig. 16).

Since this beam is a composite structure which includes the FRC and a layer of the polymer membrane and sound travels at different speed in these two materials, it was difficult to adjust the location to the exact position. However, the indication of location is reasonable.

5 CRACK DEPTH

Figure 17 indicates the crack locations of beam 3 detected by AE system sensors 3 and 4. X-Loc at 0 represents the top surface of the beam while 15 cm is the bottom surface.

The first located event on the top part of the beam was after peak load, when the load had dropped from 18.57 kN to 12.23 kN. It is also when the crack at the bottom surface became visible.

When the beam started to crack, the bottom surface was in tension condition while the top surface was in compression. It is certain that some hits were generated at the top surface of the beam. In an AE test, breaking or squeezing can both cause an AE signal. To be more specific, rubbing between any two surfaces can release sound waves. Therefore, it is difficult to conclude whether the hits at the top region resulted from friction between the roller support and the beam or compression cracks during the experiment.

In order to demonstrate the existence of compression cracks, a further check was carried out after finishing the beam test. Once the yellow steel

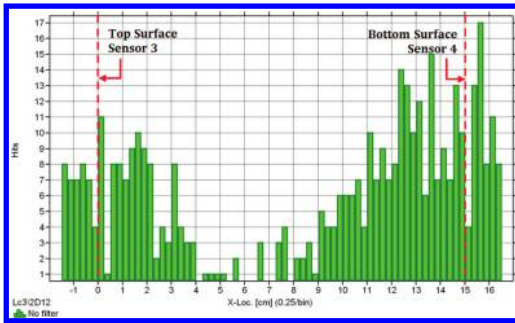


Figure 17. Crack locations of beam 3 detected by AE system sensor 3 and 4.

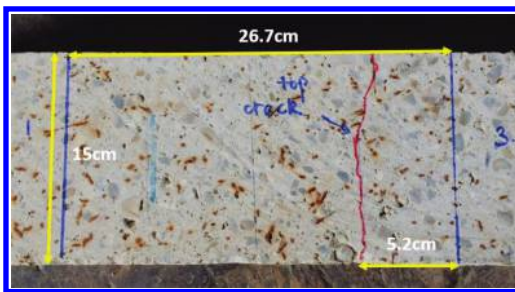


Figure 18. Top surface of beam 3 after finishing the experiment.

beam and all roller bearing supports were removed, one compression crack could be seen clearly on the top surface of the beam (see Fig. 18) which further proves the feasibility of the AE technique in testing for compression cracks.

6 CONCLUSIONS

Short-term four point bending tests was carried out on three FRC samples cut from panels built up from a sprayed primary layer, a spray-applied waterproofing membrane and a sprayed secondary layer. The acoustic emission technique was applied to detect the crack development during the experiment.

In general, the experiment went on very well. The initial objectives including determining the load at which first cracking occurred, the crack locations, the crack widths and providing recommendations for tunnel design. However, as the experiment went on, the objective of measuring crack depth was added.

There are some significant findings which can be summarized as:

- After testing different load steps on beam 1, a ± 0.1 mm deflection step is found to be reasonable in a 'stroke-control' test on an FRC beam of this size.
- First crack of an FRC beam is considered to be at 66% of the failure load with corresponding bending stress of 5.79 N/mm^2 .
- Beam 2 was thought to have an existing crack before the experiment started, due to two pieces of evidence. One is the smaller peak load (13.2 kN) compared to the other two beams (18 kN). The other one is the large number of AE hits produced during the experiment. The total number of AE hits of beam 2 is five times the amount of beam 1 or 3.
- Although beam 2 didn't reach the predicted peak load, the residual load after passing the peak load was similar to beam 1 and 3 at around 72% of peak load (12 kN). Thus the residual load of FRC with pre-existing cracks is the same as for un-cracked beams.
- Based on the analysis of AE data, it is certain that AE technique performed really well in determining the crack locations and crack depths. The corresponding crack details derived from AE data and observed by eyes are consistent. This further proves the feasibility of using AE technology in monitoring the structure health in daily life.

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Numerical simulation of the through-thickness cracking of concrete cover due to steel reinforcing bars corrosion

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ABSTRACT: Concrete cover cracking due to the corrosion of steel reinforcing bars is one of the main causes of deterioration in Reinforced Concrete (RC) structures. The oxidation level of the bars causes varying levels of expansion. The rebar expansions could lead to through-thickness cracking of the concrete cover, where depending on the cracking characteristics, the service life of the structures would be affected. In this paper, the parameters which influence the required pressure for concrete cover cracking induced by the rebar expansion due to corrosion have been investigated through detailed numerical simulations. ABAQUS finite element software is employed as a modelling platform where the concrete cracking is simulated by means of eXtended Finite Element Method (XFEM). The accuracy of the numerical simulations is achieved by comparison of the numerical results with experimental data obtained from the literature. The results reveal that the most effective parameter in the cracking process is the ratio of concrete cover depth to the rebar diameter. Using a previously proposed empirical equation and the numerical model, the time from corrosion initiation to the cover cracking is calculated and then compared to the respective experimental data.

1 INTRODUCTION

Due to the high alkalinity of the concrete pores solution, a thin layer is formed surrounding steel reinforcing bars which protects them against corrosion (Broomfield 2007). In this case, the bars are called “Passive”. This layer is unstable and disappears with decrease of pH. Carbonation of the concrete cover and the presence of sufficient chloride ions on the steel bars surfaces are the main reasons for the protective layer disappearance and corrosion initiation (Zhao & Jin 2011). The corrosion products diffuse into capillary voids in concrete. If the total amount of corrosion products is less than the porous zone volume, no additional stress is developed in concrete. This stage is known as free expansion. When the products volume exceeds that of the porous zone, expansive stresses are developed in concrete surrounding the reinforcing bars; the more corrosion products, the higher stresses. Finally, once the stresses exceed tensile strength of concrete, cracking initiates. Therefore, RC

structures which are exposed to aggressive environments are prone to fail before reaching the end of their service lives (Zhao et al. 2014).

Considerable research has been undertaken on corrosion-induced cracking of concrete cover, effective parameters on the process, and the time to cracking of the cover. The research can be classified into three main categories, experimental tests, analytical studies, and numerical simulations (Clark & Saifullah 1993, Andrade et al. 1996, Oh et al. 2009, Zhao et al. 2012, Morinaga 1988, Williamson & Clark 2000, Pantazopoulou & Papoulia 2001, Bhargava et al. 2006, Suda et al. 1993, Du et al. 2006, Chen & Mahadaven 2008, Ozbolt et al. 2011, Cao et al. 2013).

There are two main test methods, in practice, to conduct the experiments, accelerated and simulated corrosion test methods (Clark & Saifullah 1993, Andrade et al. 1996, Oh et al. 2009, Zhao et al. 2012, Morinaga 1988, Williamson & Clark 2000). Andrade et al. (1993) and Rodriguez et al. (1996) employed the accelerated method for

investigating the corrosion-induced cracking and influential parameters to develop equations for the time of concrete cover cracking. Localized corrosion effect of steel reinforcement on the concrete cover cracking was experimentally accelerated by Torres-Acosta & Sagüés (2004) where an empirical equation was also proposed for the time of cracking. McLeish (1986) experimentally simulated the pressure induced by reinforcing bars corrosion to examine the effective parameters on the cracking process. Allan & Cherry (1992) examined local corrosion of reinforcement through the simulated corrosion test method. Williamson & Clark (2000) carried out experimental tests by means of the simulated method to investigate the pressure magnitude induced by the corrosion products which would result in surface cracking of the concrete cover.

A variety of analytical models dealing with corrosion induced concrete cover cracking can be found in the literature (Pantazopoulou & Papoulia 2001, Bhargava et al. 2006, Bazant 1979, Liu & Weyers 1998, Maaddawy & Soudki 2007, Lu et al. 2011, Yu 2013).

Bazant (1979) proposed an analytical model to predict the time of concrete cover cracking caused by corrosion of embedded reinforcing steel. Another model was developed by Bhargava et al. (2006) to predict the time required for cover cracking and the weight loss of reinforcement. Yu (2013) used the damage mechanics with consideration of two distinct stages, i.e. the non-cracking stage and the partial cracking stage, to develop an analytical model for predicting the time of the concrete cracking.

Dagher & Kulendran (1992) simulated the corrosion-induced fracture of concrete via finite element analysis where smeared crack approach was employed for fracture analysis. The model considered only the radial expansive deformation around the reinforcing bar induced by corrosion while other loadings, such as dead and live loads, were not included. Du et al. (2006) presented a two-dimensional finite element model under a plane strain assumption to idealise three-dimensional physical specimens, tested by the simulated method and investigate the development mechanism of the concrete cracking. The model was then employed to predict test results from reinforced concrete accelerated corrosion. An integrated computational methodology for chloride-induced degradation assessment of RC structures by considering all three phases of the deterioration process, i.e. chloride penetration process, reinforcement corrosion, and rust expansion was developed by Chen & Mahadaven (2008). Finite element analysis with a smeared cracking approach is implemented to simulate the rust expansion and the associated concrete cracking process.

In this paper, the effect of various parameters including concrete tensile strength, f_t , bar diameter, d , concrete cover thickness, C , and ratio of the cover thickness to the bar diameter, C/d , on the pressure required for cracking is investigated via eXtended Finite Element Method (XFEM) in ABAQUS software. Furthermore, using an empirical equation proposed by other researchers and also taking advantage of the developed numerical model, the required times for cover cracking are calculated and then compared to the experimental data.

2 FINITE ELEMENT MODELLING

The concrete specimens have been modelled and analysed using ABAQUS (Standard User's Manual 2010) finite element software. The eXtended Finite Element Method (XFEM) within the Linear Elastic Fracture Mechanics (LEFM) framework is incorporated in the nonlinear analysis of the models to capture concrete cover cracking. Taking advantage of XFEM feature, i.e. enriching degrees of freedom, crack initiation and propagation along an arbitrary and solution-dependent path with no requirement of re-meshing the models are achieved (Abdelaziz & Hamouine 2008, Asadpour & Mohammadi 2007, Areias & Belytschko 2005, Sukumar et al. 2001, Moes et al. 1999).

To reduce the computational cost of 3D simulation, a two-dimensional (2D) Finite Element (FE) model is developed. The developed 2D model is based on a plane strain finite element formulation. The 4-noded bilinear shell element, i.e. CAX4R, is used for such plane stress simulations.

The FE model is verified through comparison of numerical results with respective experimental data reported by Williamson & Clark (2000). The test specimens were cubes with edge length 150 mm and a cylindrical hole located at one corner with diameter of 8 mm, as seen in Figure 1. The ratios of concrete cover thicknesses to the hole diameter, i.e. C/d , were 0.5, 1.0, and 2.0. A uniform and gradually increasing outward pressure is applied into the holes to simulate the expansion induced by corrosion of steel bars.

The mechanical properties of the concrete material are provided in Table 1 (Williamson & Clark 2000).

A mesh sensitivity analysis is undertaken to determine the maximum element size for such simplified models where a value of 28 GPa is considered for Young's modulus according to ACI 318 (2008).

$$E_c = 4730\sqrt{f'_c} \quad (GPa) \quad (1)$$

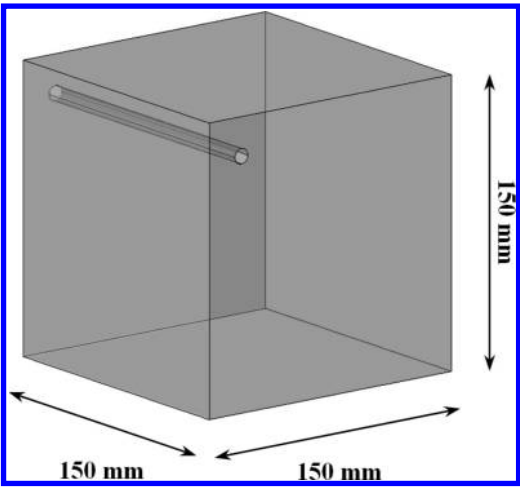


Figure 1. Specimens' dimensions tested by Williamson & Clark (2000).

Table 1. Mechanical properties of concrete Williamson & Clark (2000).

Compressive strength (<i>MPa</i>)	35
Tensile strength (<i>MPa</i>)	2.8
Poisson's ratio	0.2

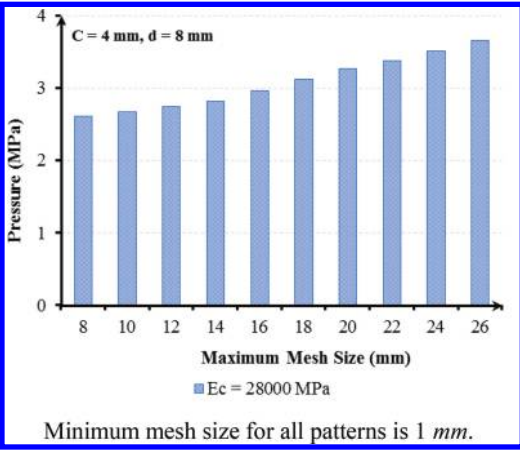


Figure 2. Mesh sensitivity analysis.

As seen in Figure 2, the maximum element size of 10 mm results in reasonable accuracy in the simulation results. The 2D model is validated through comparison of the numerical results with experimental counterpart (Williamson & Clark 2000).

A typical 2D finite element model is shown in Figure 3.

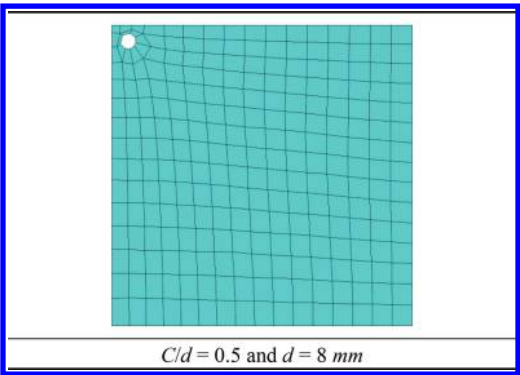


Figure 3. A typical FE model of the concrete specimens.

Table 2. The selected experimental specimens for the validation.

Specimen ID	Cover (<i>C</i>)	Bar size (<i>d</i>)	Tensile strength (<i>f_t</i>)
M-4-8-2.8	4 mm	8 mm	2.8 MPa
M-8-8-2.8	8 mm	8 mm	2.8 MPa
M-16-8-2.8	16 mm	8 mm	2.8 MPa

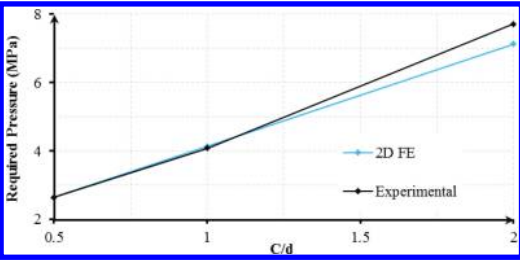


Figure 4. Validation of 2D numerical model.

The experimental specimens considered for the numerical validation purpose listed in Table 2. The numerical models are labelled such that the concrete cover thickness (*C*), bar diameter (*d*), and tensile strength of concrete (*f_t*) of each model can be identified from the label. For example, the label “M-4-8-2” indicates that the model has a cover thickness of 4 mm, a hole with diameter of 8 mm, and concrete with tensile strength of 2 MPa.

The numerical results along with the respective experimental data are provided in Figure 4. As seen, the 2D model can accurately predict the required pressure for initiation of concrete cover cracking.

3 DISCUSSION OF FE RESULTS

The effects of parameters including concrete cover thickness (C), steel bar diameter (d), and tensile strength of concrete (f_t) on the required pressure for concrete cover cracking are investigated in this section.

3.1 Effect of concrete cover thickness

To evaluate the effect of the cover thickness variation on the required expansive pressure, six models are developed with varying cover thicknesses (C), from 5 mm to 30 mm while all other geometrical and material properties are kept constant ($d = 8$ mm, $f_t = 2.8$ MPa). Therefore, the models' labels are "M-C-8-2.8".

The results are provided in Figure 5. As can be seen, by increasing the cover thickness, the required pressure is increased, i.e. the thicker the cover, the higher the pressure.

Figure 6 shows the variation of strain energy against pressure of the models up to the respective failure points. The increasing of the cover thickness increases the failure pressure and consequently the ultimate strain energy. However, at the same pressure

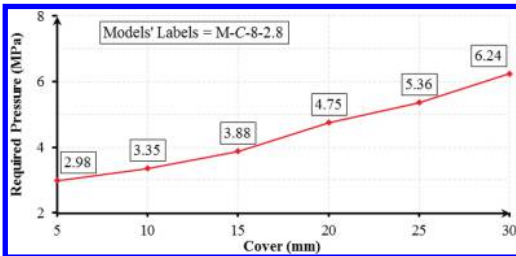


Figure 5. Effect of concrete cover thickness on the required pressure for cracking.

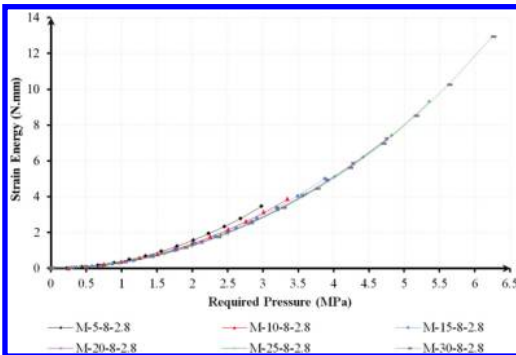


Figure 6. Strain energy vs. the required pressure for cracking.

values, cover thickness increase leads to decrease in strain energy. It is likely attributable to the deformation pattern of the models. The deformation is localized at the vicinity of the areas with lower stiffness, i.e. concrete cover, where the cracking initiates. By increasing of the cover the surrounding areas of the rebar become more symmetric in stiffness which results in higher required values for deformation localization and crack initiation.

3.2 Effect of steel bar diameter

Six finite element models are developed with varying hole diameters (d), from 10 mm to 20 mm, where all other geometrical and material properties are kept constant ($C = 25$ mm, $f_t = 2.8$ MPa), hence the models' labels are "M-25-d-2.8".

The pressure required for cracking of concrete cover for each model is shown in Figure 7. As seen in the figure, the expansive pressure decreases as the hole diameter increases.

3.3 Effect of tensile strength of concrete

In this section, the effect of tensile strength of concrete (f_t) is investigated. To this end, four values for

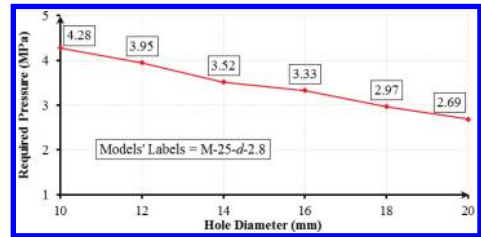


Figure 7. Effect of hole diameter on the required pressure for cracking.

Table 3. Effect of concrete tensile strength on the required expansive pressure.

f_t (MPa)	C (mm)	d (mm)	C/d	Model ID	P_{req} (MPa)
1.5	4	8	0.5	M-4-8-1.5	1.28
		8	1	M-8-8-1.5	2.39
		12	1.5	M-12-8-1.5	3.41
1.7	4	8	0.5	M-4-8-1.7	1.56
		8	1	M-8-8-1.7	2.67
		12	1.5	M-12-8-1.7	3.63
1.9	4	8	0.5	M-4-8-1.9	1.70
		8	1	M-8-8-1.9	3.05
		12	1.5	M-12-8-1.9	4.09
2.1	4	8	0.5	M-4-8-2.1	1.92
		8	1	M-8-8-2.1	3.35
		12	1.5	M-12-8-2.1	4.77

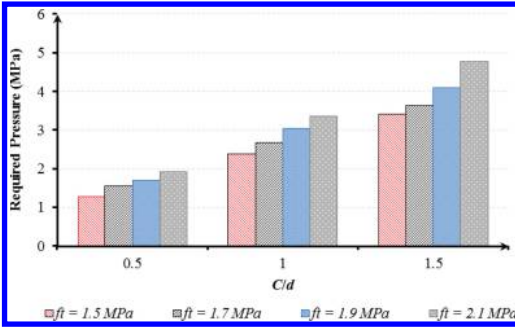


Figure 8. Effect of concrete tensile strength on the required expansive pressure.

the tensile strength are examined (1.5 MPa, 1.7 MPa, 1.9 MPa, and 2.1 MPa) where for each value, three different cover thicknesses are considered.

As expected, the results show that the tensile strength has a considerable effect on the required pressure for concrete cover cracking. By increasing of the tensile strength, the pressure increases, as shown in Table 3 and Figure 8.

4 TIME PREDICTION OF CONCRETE COVER CRACKING

4.1 Concept and equations

To determine the time from corrosion initiation to concrete cover cracking, the empirical equations proposed by Martin-perez (1999) is employed as Equations 2 and 3.

$$\Delta V_s = 3.6466 \times 10^{-11} \pi d_b i_{corr} \Delta t \quad (2)$$

$$\Delta M_s = 2.862 \times 10^{-7} \pi d_b i_{corr} \Delta t$$

$$(r_o^2 - r_c^2) \gamma = (r_r^2 - r_c^2) \quad (3)$$

where i_{corr} is the rate of corrosion ($\mu A/cm^2$), ΔV_s is volume change of steel, ΔM_s is mass change of steel, d_b is bar diameter (m), Δt is time (s), r_o is original bar radius (m), r_c is reduced radius of corroded bar (m), r_r is the total bar radius including rust (m), and γ is volumetric ratio of a rust product.

The risk of corrosion is divided into four levels as provided in Table 4. Depending on the iron oxidation level, corrosion leads to varying corrosion current density (i_{corr}) which directly influences the volume increase. The steel reinforcement may have up to six times increase in volume.

Figure 9 shows the procedure how to employ the numerical model to predict the time from corrosion initiation to cover cracking using the empirical equations.

Table 4. Risk of corrosion.

i_{corr} ($\mu A/cm^2$)	Risk of corrosion
$i_{corr} < 0.1$	Negligible
$0.1 < i_{corr} < 0.5$	Low
$0.5 < i_{corr} < 1.0$	Middle
$i_{corr} > 1.0$	High

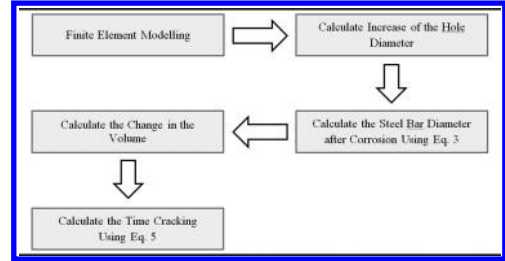


Figure 9. Calculation procedure of the cracking time.

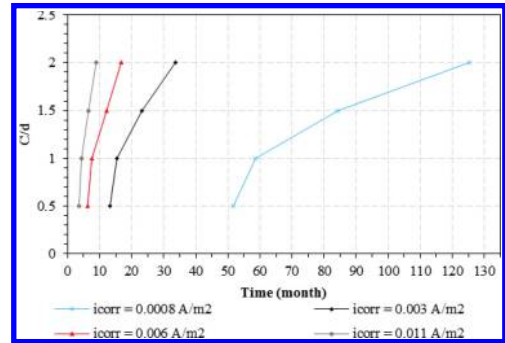


Figure 10. Risk of corrosion.

Figure 10 shows the concrete cover to bar diameter ratio against time obtained from numerical simulation and the empirical equations for four different values of i_{corr} .

4.2 Accuracy of the predicted time

To assess the accuracy of the time estimation, the results from experiments in literature and the estimated data are compared. The experimental specimens' properties (C , d , i_{corr}) are provided in Table 5. In some cases where the actual tensile strength (f_t) and the modulus of elasticity (E_c) were not available, they were estimated by using Eqs. 1 and 2 in accordance with ACI 318 (2008).

$$f_t = 0.33 \times \sqrt{f'_c} \quad (MPa) \quad (4)$$

$$E_c = 4730 \sqrt{f'_c} \quad (GPa) \quad (5)$$

Table 5. Experimental specimens' characteristics.

Specimen	d (mm)	C (mm)	i_{corr} ($\mu A/cm^2$)	f_t (MPa)	E_c (GPa)
Andrade et al. (1996)	16	20	100	3.55	22
Rodriguez et al. (1996)	16	20	100	3.85	22
Maaddawy & Soudki (2007)	16	33	150	4.9	28

Table 6. The estimated and experimental time for cracking.

Specimen	Experimental time (h)	Estimated time (h)	Dif. (%)
Andrade et al. (1996)	96	117	17.95
Rodriguez et al. (1996)	113	153	26.14
Maaddawy & Soudki (2007)	95	121	21.48

The estimated times from corrosion initiation to concrete cover cracking are compared to the respective experimental data in Table 6.

As can be seen from the table, the experimental data to some extent are lower than the predicted values using the model. The difference between the cracking times obtained from numerical simulations and experiments could be attributed to the fact that experimental specimens generally have initial cracks mainly due to shrinkage and creep while no imperfection, i.e. pre-cracking, is assumed for the numerical models. Therefore, cracking times in the models are greater than those of the specimens.

5 CONCLUSIONS

A two-dimensional (2D) numerical model developed in this paper to estimate the required pressure for concrete cover cracking. The effect of material and geometrical parameters including tensile strength of concrete (f_t), cover thickness (C), and rebar diameter (d) were examined on the pressure. The model can be used along with other empirical equation to estimate the time of cover cracking.

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Experimental characterization of ferrocement wall panels for sustainable low cost housing in Pakistan

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ABSTRACT: According to data from the Government of Pakistan, around 25% of the population in the country lives in one room houses. Many become homeless due to natural disasters. To provide them with affordable housing, research is needed to examine low cost materials and quick construction techniques. The objective of this research is to characterize ferrocement wall panels to examine the possibility of using them as a low cost housing material in Pakistan. For this purpose, a total of 18 ferrocement wall panels were tested, both in flexure and axial compression. Comparison of cost was also carried out for a 675 ft² (3 Marla) housing unit. Results show that ferrocement panels are capable of bearing the load of a single story housing unit. Due to their high energy absorption and ductile behavior they are also suitable for earthquake prone areas. Comparison of ferrocement construction with traditional brick masonry shows that the material cost of both systems is comparable; however the ferrocement house can be assembled in 1/3rd of the time in comparison to brick houses.

1 INTRODUCTION

Due to high cost of construction, building a house has become a challenge for low income group. The need of low cost and affordable houses is much discussed subject especially in the wake of rapid growth of population. According to Sumadi and Ramli (2008), constructing own house is not an affordable proposition for the large fraction of common people. In Pakistan, 24.83% of population is living in one room housing units (Government of Pakistan 2011). Further, in recent years, thousands of people become homeless due to natural disasters and for them the immediate need is to have a shelter.

Low cost housing is a relative term and is more related with budgeting and better management to reduce the cost through appropriate use of resources, without sacrificing the performance and structural life of buildings. Low cost housing is considered affordable if a household can acquire a house for an amount less than 30% of their income (Miles 2013). It must be understood that low cost houses are not the ones which are constructed using material of substandard quality (Tam 2011). Mainly there are two approaches to achieve cost effectiveness: use of local and cost effective materials or reduction of labor cost through accelerated construction methods and innovative techniques.

For reducing construction time, prefabricated steel construction is in practice, however, this is only limited to industrial buildings. For housing units, steel prefabricated construction may not be a good option due to low serviceability features and high cost of material. Similar concept can be adopted by using some cementitious material. Prefabricated ferrocement panels may be a potential candidate for low cost housing. These panels can be prefabricated using locally available materials and can be easily assembled in very less time.

According to ACI 549R-97 ferrocement is a type of thin wall reinforced concrete constructed with cement mortar having layers of wire mesh embedded as reinforcement. Ferrocement has higher tensile strength-to-weight ratio and better cracking behavior as compared to conventional reinforced concrete. Therefore, light weight and water tight structures can be constructed using ferrocement. Ferrocement can be used as an alternative material of construction for prefabricated housing units (ACI 549R-97 1997). Since 1884, Ferrocement has been used in the making of boats, plant pots, housing etc. (Aboul-Anen et al. 2009).

Close distribution of reinforcement in ferrocement helps it to act as a high performance material. In future it may be an economical option for constructing temporary and permanent buildings (Saleem and Ashraf 2008). Hundreds of roofs with

ferrocement are constructed in poor communities and success of ferrocement for houses is well documented (Ibrahim 2011). Major advantage of ferrocement is that almost all type of structures made of conventional materials can also be constructed using ferrocement. Ferrocement has its own limitations and within its limitations ferrocement is considered to be one of the best composite materials.

All constituent materials of ferrocement are easily available in Pakistan. This is one of the reasons of selecting ferrocement as an alternative solution of low cost housing. The panels of ferrocement are relatively light-weight, easy to cast, transport and assemble, therefore prefabrication of these panels is easily possible. A do-it-yourself concept of a small house using ferrocement wall panels has previously been proposed by Saleem and Ashraf in 2008. The proposed house model may be used as temporary or permanent residence.

The primary objective of this research was to investigate the suitability of ferrocement as an alternative material for low cost housing and also to decide suitable cross sectional dimensions and volume fraction in terms of numbers of mesh layers for ferrocement panels. A secondary objective was to compare the cost of ferrocement construction with conventional brick and mortar construction.

2 EXPERIMENTAL WORK

To fulfill the objectives of the research, a comprehensive test matrix was developed which is presented in Table 1. Total 19 wall panels of size 450 mm × 1500 mm each were cast including one trial specimen. Thickness of panels and number of mesh layers were variables in specimens as shown in Table 1. Each specimen was designated with a unique identification number which provides information of thickness, number of mesh layers, testing in axial of flexure and count of the unique specimen. For example in case a panel has tag number 50-4-F2, '50' is thickness of mm, '4' denotes numbers of mesh layers, F means the specimen was tested in flexure and '2' shows that it is 2nd specimen with the same specifications. Further it must be noted that for specimens tested in axial, 'A' is used instead of 'F' in identification tag.

Ordinary portland cement was used for the mortar of ferrocement panels. To make consistent batches of mortar, cement from same source was used for each batch. Fresh bags of cements were purchased every time on the day of casting. Aggregate size must be compatible with thickness of panels, mesh opening and mesh spacing. Due to low thickness of ferrocement and closely spaced reinforcement, ACI 549 recommends the use of

Table 1. Test matrix.

No.	Panel ID	Reinforcement	
		Mesh layers	Volume fraction
1	30-2-F1	2	1.67
2	30-2-F2	2	1.67
3	30-3-F1	3	2.51
4	30-3-F2	3	2.51
5	40-3-F1	3	1.88
6	40-3-F2	3	1.88
7	40-4-F1	4	2.51
8	40-4-F2	4	2.51
9	50-4-F1	4	2.01
10	50-4-F2	4	2.01
11	50-5-F1	5	2.51
12	50-5-F2	5	2.51
13	30-2-A1	2	1.67
14	30-2-A2	2	1.67
15	40-3-A1	3	1.88
16	40-3-A2	3	1.88
17	50-4-A1	4	2.01
18	50-4-A2	4	2.01

aggregates passing through sieve # 8 (2.36 mm) i.e. only fine aggregates. For this purpose lawrancepur sand was used. Reinforcement in ferrocement may be in form of mesh, one dimensional fiber mats, three dimensional reinforcement or fibers. In the current research galvanized iron wire mesh with square openings was used because it was easily available in the local market. Mesh opening size was 12.5 mm and wire diameter was 1.4 mm. Skeletal steel was used to keep reinforcement in position during pouring and to maintain spacing between mesh layers. For this purpose steel bars of 6 mm diameter were used, which is maximum allowed by ACI. Rich cement to sand ratio of 1:2 was used to obtain a dense matrix with high compressive strength. Water to cement ratio range was from 0.40 to 0.43. Reinforcement ratio was controlled by changing the number of mesh layers from 2 to 5.

Smooth plywood formwork was used for casting the panels. Three different formworks of thickness 30 mm, 40 mm and 50 mm were used. For each sample new batch of mortar was used. Cement and sand were mixed with water in pan type mixer. Mixing was completed in two phases, dry mixing and wet mixing. In dry mixing cement and sand were mixed for 2 to 3 minutes. When cement and sand appeared to be homogeneously, water was added slowly in the running mixer. After adding water, mixer was run for another 2 to 3 minutes to ensure a homogeneous mixture. The formwork was placed on a table type vibrator and a thin layer of mortar equal to the clear cover was poured in

the formwork and vibrated. Mesh layers fixed with skeletal steel was placed on this thin mortar layer. Now the formwork was filled with the mortar to approximately 3/4th of the height and second layer of the mesh was then placed and the formwork was vibrated again. Then mortar was then filled to the top with another round of vibration. Specimens with 3, 4 and 5 mesh layers were also cast in similar fashion. In the end, the top surface was leveled using steel trowel and specimens were moved



Figure 1. Setup for flexural test.



Figure 2. Setup for axial test.

to the curing area. Formwork was removed after 48 hours and curing continued for 28 days using wet jute bags.

Panels were tested under four point bending as shown in Figure 1. Point loads were applied to the panels at distance of 450 mm from each support i.e. one third of the clear span. A clearance of 75 mm from the edge of the panel to the support was provided for both supports. Load and mid-span deflection data were recorded using a high speed data acquisition system at a frequency of 1 Hz. Deflection controlled loading was applied at a rate of 0.5 mm/min. Two Specimens of each thickness were tested in axial compression. Figure 2 presents the test setup for the axial load test setup. To measure horizontal deflection at midpoint, a displacement transducer was installed with the test setup. Axial test was also conducted with displacement controlled loading with a displacement rate of 0.5 mm/min.

The expected axial load and bending moment is calculated for a single storey housing unit with one bed room of size 5 m × 5 m. The factored axial load for the most critical wall comes out to be 120.4 kN. For a wind speed of 160 km/hr the maximum factored bending moment developed in the wall panel is 0.1 kN-m.

3 RESULTS AND DISCUSSION

3.1 Compressive strength of mortar

Compressive strength of mortar was determined at 7, 14 and 28 days. For this purpose, 100 mm diameter, 200 mm height cylinders were cast from each batch. Table 2 shows 28 days compressive strength of mortar. Maximum compressive strength achieved was 49.6 MPa, minimum compressive strength was 33.1 and average value of all 18 batches was 41.5 MPa with standard deviation of 5.0.

3.2 Load-deflection responses of flexural test

Load-deflection responses are presented in Figure 3. Two specimens of 50 mm thickness with 5 mesh layers achieved maximum deflection of 30.6 and 30.7 mm and the ultimate load of 8.17 and 5.9 kN, respectively. Load deflection curves of both samples having 4 mesh layers and 50 mm thickness exhibited similar trend. Absolute maximum failure load was of specimen 50-4-F1 i.e. 16.59 kN. Specimens with 50 mm thickness having 4 mesh layers showed more ductile behavior than samples with 5 mesh layers. Specimens with 40 mm thickness showed increasing trend of maximum load and maximum deflection with increase in numbers of mesh layer. Two specimens with

Table 2. Compressive strengths at 28 days.

S. no.	Panel ID	28 days compressive strength (MPa)
1	50-5-F1	42.6
2	50-5-F2	44.7
3	50-4-F1	40.9
4	50-4-F2	37.1
5	40-4-F1	47.1
6	40-4-F2	39.1
7	40-3-F1	35.1
8	40-3-F2	47.2
9	30-3-F1	44.1
10	30-3-F2	43.1
11	30-2-F1	33.1
12	30-2-F2	48.1
13	50-4-A1	38.3
14	50-4-A2	42.1
15	40-3-A1	38.1
16	40-3-A2	49.6
17	30-2-A1	33.1
18	30-2-A2	43.1

4 mesh layers reached the deflection of 38.6 and 42.2 mm and ultimate load of 7.85 and 12.84 kN. Whereas, the two specimens with 3 mesh layers showed different behavior reaching maximum deflection of 36.8 mm and 16.5 mm. The 30 mm thick samples achieved more deflection and failed at lesser load as compared to 40 mm and 50 mm thick panels. Maximum deflection achieved by specimen of 30 mm thickness with 3 mesh layers was 80.8 mm. For the same volume fraction and different thicknesses of the specimens, load carrying capacity increases with increase in thickness. However, maximum deflection increases with the decrease in thickness of the panels.

Under flexural loading first crack appeared at bottom closed to mid span of the panels. With increase in load, crack width increases and load is transferred to bottom most layer of reinforcement mesh. After yielding of mesh layer load is transferred to mortar and new cracks appears. This phenomenon is repeated until failure of the specimen. Crack in all panels appeared in similar manner and shown in Figure 4.

3.3 Load deflection response under axial load

Load deflection responses for axial load tests are presented in Figure 5. Specimens of 50 mm, 40 mm and 30 mm thickness had slenderness ratio of 104, 130 and 173 respectively. With the increase in thickness of ferrocement panels, ultimate load increased and more vertical displacement occurred at the failure. Maximum load at failure was 666,

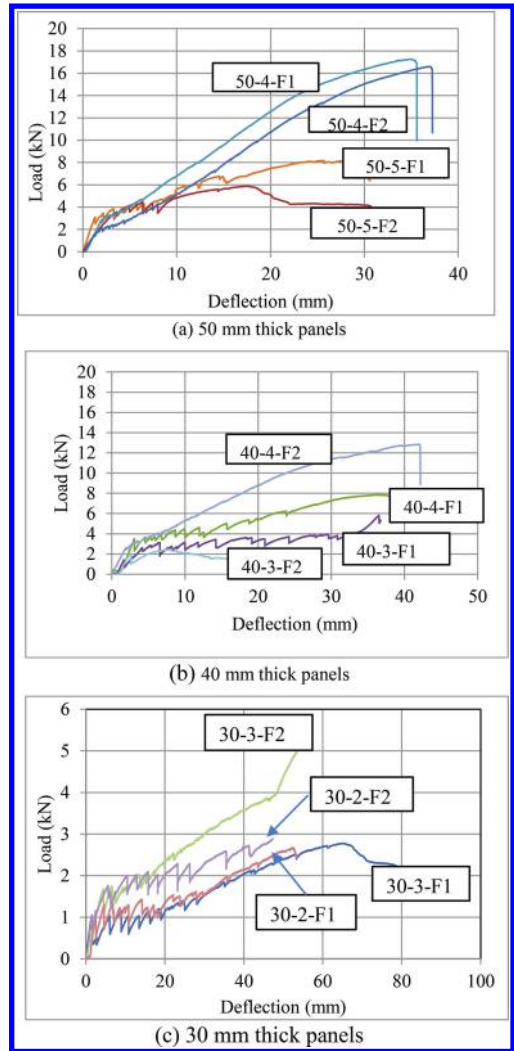


Figure 3. Load deflection responses for flexural tests.



Figure 4. Flexural cracks in specimen 40-4-F1.

494 and 284 kN for 50, 40 and 30 mm thick panels, respectively. Panels having 50 and 40 mm thickness failed in compression near the supports while 30 mm thick panel failed in buckling with the cracks appearing at the mid height.

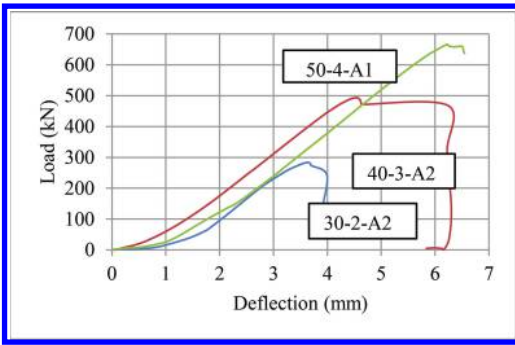


Figure 5. Load deflection responses for axial load tests.

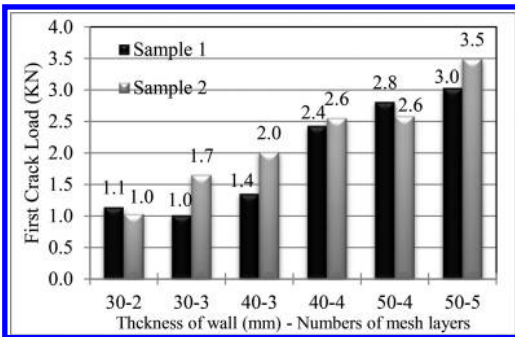


Figure 6. First crack comparison for flexural test.

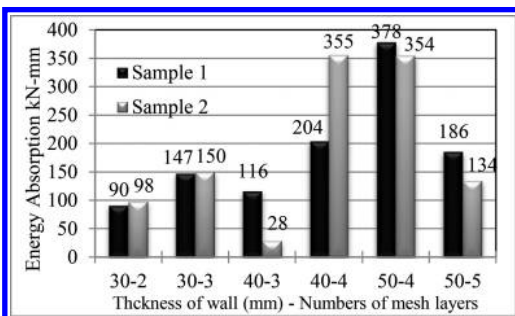


Figure 7. Energy absorption for the flexural tests.

3.4 First crack and ultimate load

First crack load in the flexural test showed increasing trend with the increase in thickness and numbers of mesh layers. Figure 6 shows comparison of first crack load for all 12 samples tested in flexure. Ultimate load achieved by panels also shows increasing trend with the increase in thickness and numbers of mesh layers. Both specimens of 50 mm thickness with 4 mesh layers showed inconsistent behavior by reaching ultimate load greater than

50 mm thick panels with 5 mesh layers. Under axial loading, load carrying capacity increases with increase in thickness and volume fraction.

3.5 Energy absorption

Energy absorbed by panels in flexure is calculated by computing area under the load deflection curve up to ultimate load. Samples with 30 and 40 mm thickness shows increase in energy absorption with the increase in volume fraction while 50 mm thick panel showed opposite trend (see Fig. 7).

4 COST ANALYSIS

Cost analysis is carried out to compare ferrocement construction of a single story housing unit with brick and mortar construction. Direct cost is distributed in two components i.e. direct material cost and direct labor cost. Direct material cost is calculated for panel of size 1 m × 2.5 m. For this purpose only structural elements are considered for both ferrocement and brick masonry. Considering 50 mm thick panel with 5 mesh layers, cost of ferrocement is almost same to that of brick masonry. Direct labor cost is compared for a small housing unit of 675 ft² (3 Marla). Man hour estimation for both type of construction is shown in Table 3. Man hours requirement for ferrocement construction is 1/3rd as compared to the brick masonry construction.

Table 3. Break up of man hours.

No.	Activity	Brick construction		Ferrocement construction	
		Days	Man hours	Days	Man hours
1	Excavation/back filling	3	96	3	96
2	Strip foundation	5	120	–	–
3	Block foundation	–	–	2	48
4	Damp proffing	1	24	–	–
5	Joint sealing	–	–	1	16
6	Brick wall	10	400	–	–
7	Ferrocement panels erection	–	–	3	96
8	Roof shuttering	3	120	–	–
9	Concreting	1	80	–	–
10	Curing of roof slab	14	–	–	–
11	Formwork removal	2	48	–	–
12	Wooden truss installation	–	–	2	64
13	Roof sheeting installation	–	–	2	48
Total		39	888	13	368

5 SUMMARY AND CONCLUSIONS

The primary objective of this research was to evaluate ferrocement as a potential alternative for the construction of walls of a single story housing units. For this purpose, ferrocement wall panels with various volume fraction and thicknesses were tested. Strength characteristics under flexural and axial loads were determined and cost of construction was calculated. Load deflection responses were plotted based on data gathered from flexural and axial testing. In order to perform cost analysis of ferrocement wall panels with brick walls, a single story small housing unit was considered.

Ferrocement panels have demonstrated adequate structural strength to bear the load in a typical housing unit. In axial compression load, maximum load achieved by 50 mm thick panel having slenderness ratio of 104 was 666 kN. Minimum axial failure load was 284 kN for 30 mm thick wall having slenderness ratio of 173. In a single story housing unit a wall of room having dimensions of $5\text{ m} \times 5\text{ m}$ has to bear a factored load of 120.4 kN which only 42% of the minimum failure load of the ferrocement panel.

Maximum bending moment capacity of 50 mm thick panel is 3.88 kN-m while lowest capacity is 0.60 kN-m of 30 mm thick panels. Demand of bending moment is 0.1 kN-m for a wind speed of 160 km/hr. Although this demand is fulfilled by all the panels, however 40 mm thick panel may be a good choice because it does not fail in buckling in the axial test, unlike the 30 mm panels.

Cost of a 50 mm thick ferrocement wall having height of 2.5 meters is compared with 230 mm thick brick masonry wall. Material cost for structure is almost same equal for both materials, however, less than half man hours are required for ferrocement construction. Therefore, it can be concluded that ferrocement provides low cost solution in terms of material and man power for temporary and permanent housing facilities. While considering ferrocement walls houses for long term use, these

will require additional cost on materials to make it weather resistant and meet serviceability requirements. In this scenario, the real cost saving will be from the man hours of labor.

Behavior of ferrocement in punching shear and under impact loading is still needed to be evaluated before its application in real life. Galvanized Iron wire mesh was used for this research because it is easily available at hardware stores in Pakistan. However, study on ferrocement with some other type of reinforcement like polypropylene should also be carried out to make it a rust free solution. It is also important to study the durability characteristics of ferrocement. A full scale test of ferrocement house on shaking table is also needed before its application in earthquake prone regions.

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Compressive and flexural strength of Ultra-High Performance Fibre Reinforced Concrete containing recycled rubber crumb

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ABSTRACT: Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) has excellent mechanical properties and good durability. However, the embodied energy of UHPFRC is much higher than that of ordinary concrete due to the high usage of cement. In order to reduce the adverse environmental impact of utilizing UHPFRC, recycled rubber crumb is used as an additional component or as a replacement for fine aggregates. Cube specimens with two different mix design schemes were investigated during this study. The reduction of compressive strength was investigated. UHPFRC prisms were also cast and loaded in four point bending tests to check the reduction of flexural strength. The mixing procedure and temperature variation during the casting process was recorded. Some cube specimens were heat cured in hot water at 90 degree Celsius for forty-eight hours. Those cubes were tested following curing at around 7 days to obtain the early age strength, while other cubes are normally cured in water at around 20 degree Celsius with the prisms until 28 days.

1 INTRODUCTION

Large amounts of used tyres are accumulated in the world because most rubber is difficult to degrade in natural process. The treatment of waste tyre rubber generally causes considerable impact on the environment (Xie et al. 2015). A popular approach utilizing rubber from discarded tyres is to use them as replacement of aggregate in concrete. Comparing with traditional concrete, concrete with recycled rubber has higher energy dissipation, ductility, durability, damping ratio, impact resistance, and toughness. However, adding rubber also causes a series of problems. Rubber aggregates result in compressive strength losses up to 85% depending on the rubber size and content and lower tensile strength and modulus of elasticity of rubber will also cause adverse influence on the concrete properties (Yousf et al. 2014). It seems to maintain a relatively low capillary water absorption by using 15% rubber waste (F. Azevedo 2012). Mixing with crumb rubber can make a significant influence on the fracture toughness, in a certain range, rubber content can increase the fracture toughness with increasing rubber content. However, beyond certain value, toughness will decrease with increase of rubber content. In addition, appropriate rubber content can improve the ductility of concrete but there is a negative influence with excess rubber (Yousf et al. 2014). There are some similar study indicating the same results, the compressive strength, flexural

tensile strength, pull-off strength and sulphate attack of concrete with crumb rubber decrease obviously comparing with common concrete. After the content of rubber reach a certain percentage, some properties such as resistant to water absorption and carbonation will improve (Holmes et al. 2014). Temperature will influence the performance of rubber concrete, appropriate rubber content can reduce the degradation rate of compressive strength and toughness effectively after exposure to elevated temperature, but too much rubber will make a negative effect on resistance to high temperature (Guo et al. 2014b). Some study also show that the influence of crumb rubber on steel fibre concrete, comparing with normal rubber concrete, with steel fibre the reduction of compressive strength and modulus of elasticity is smaller. However, for fibre reinforced concrete, the reduction of flexural strength caused by higher rubber content is more significant than compressive strength. An optimal content of rubber is reported to be around 2% volumetric quantity in concrete (Guo et al. 2014a).

Ultra-High Performance Concrete (UHPC) and Ultra-High Performance Fibre Reinforced Concrete mixed with steel fibre (UHPFRC) have relatively high compressive strength which can exceed 200 MPa, high flexural strength, workability and durability (Yu et al. 2014). This study will investigate the compressive and flexural strength of UHPFRC containing recycled rubber crumb (Watanabe et al. 2014).

2 RAW MATERIALS AND MIX DESIGN

2.1 Raw materials

Raw materials for producing UHPFRC include Portland Cement 52.5 grade (Chinese Type II), silica fume, ground quartz, fine sand, straight steel fibre, and superplasticizer. The steel fibre used in this research is high strength straight steel fibre approximately 13 mm long and 0.2 mm in diameter, with aspect ratio around 64. These raw materials were obtained from local Chinese market. Specially, two types of rubber crumb were obtained through a regional recycling plant as shown in Figure 1.

2.2 Density measurement

As important parameters, the density of rubber crumb and fine sand were measured according to Method B in ISO 2781:2007 (ISO/IEC, 2008).



Figure 1. Recycled rubber crumb.

Weights taken in the measurement were rounded accurately to 1 mg.

The formulas used to calculate the density is shown in Equation 1:

$$\delta = \rho_w \frac{m_2 - m_1}{m_4 - m_3 + m_2 - m_1} \quad (1)$$

where ρ_w = density of water in a unit of kg/m^3 ; m_1 = mass of density bottle in a unit of g; m_2 = mass of density bottle and specimen in a unit of g; m_3 is the mass of density bottle and specimen filled with water in a unit of g; m_4 is the mass of density bottle filled with water in a unit of g.

Based on Equation 1, the measured density of sand is about $\delta_{\text{sand}} = 2400 \text{ kg/m}^3$, and the density of rubber is $\delta_{\text{rubber}} = 536 \text{ kg/m}^3$ for fine rubber crumb. Those density measurements will be utilized in the mix design for equivalent volume replacement method.

2.3 Mix design

The mix design for normal UHPFRC can be found in Table 1. The adjustment due to replacement will be imposed on this mix design. Two strategies were imposed on modification of mix design. Mix A keep the original mix design of UHPFRC and add additional rubber particles up to 20.4 kg per cubic meter, while mix B use a 40% equivalent volume replacement to reduce the usage of fine sand. The calculations were shown in Equation 2. Detailed mix design can be found in Table 1. In order to compare the effectiveness of two different designs, only fine rubber crumb were used as their particle size is comparable to that of fine sand.

$$\begin{cases} m_{\text{sand_RE}} = m_{\text{sand}} \times (1 - \lambda) \\ m_{\text{rubber}} = \frac{m_{\text{sand}} \times (\lambda)}{\delta_{\text{sand}}} \delta_{\text{rubber}} \end{cases} \quad (2)$$

where, m_{sand} and $m_{\text{sand_RE}}$ are the weight of fine sand used per cubic meter before and after the

Table 1. Summary of mix design (kg/m^3).

Material	Base mix	Mix A	Mix B
Cement	712	712	712
Silica fume	231	231	231
Water	140	140	140
Fine sand	1020	1020	612
Ground quartz	211	211	211
Superplasticizer	40	40	40
Fibre (2% vol.)	156	156	156
Rubber crumb	N/A	20.4	91.1

replacement, respectively; $\lambda = 40\%$ is the volume replacement ratio; m_{rubber} is the weight of rubber crumb used in cubic meter of concrete; δ_{rubber} and δ_{sand} are densities of rubber crumb and find sand, respectively.

3 EXPERIMENT RESULTS

3.1 Mixing procedures

The mixing process followed the general UHPFRC mixing method and ice cubes were used to control the temperature during the mixing. The temperature variation versus time plot is shown in Figure 2 for Mix design A. It was observed that use of ice significantly brought down the temperature and elongated the allowable mixing time, which is beneficial to achieve more homogenous distribution of rubber crumb with other ingredients. However, the low temperature and retarded release of water also slow down the hydration process and the whole mixing procedure took around 35 minutes. In casting mix design B, half ice half water scheme was used, and the mixing time was brought down to around 22 minutes.

3.2 Specimen list

Cube specimens were cast to obtain compressive strength while prism specimens were used in four point bending test to obtain the flexural strength. Detailed specimen list can be found in the following Table 2.

Part of specimens were heat treated in 90 degree Celsius hot water bath for 48 hours, excluding the time for heating and cooling (noted as “heat treated”) while others were kept in the water tank for 28 days (noted as “normal treated”).

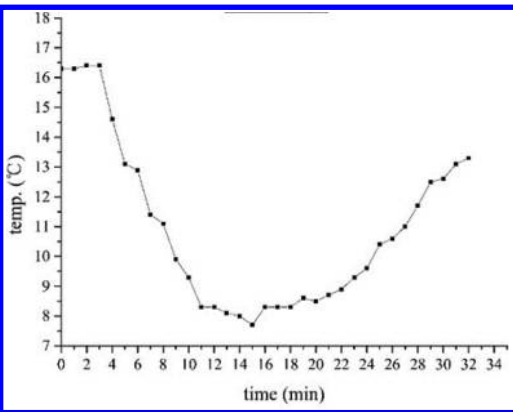


Figure 2. History of temptation variation during casting.

Table 2. Summary of mix design (kg/m³).

Mix	Cubes	Prisms
Base	6 @ 50 mm 3 @ 100 mm	70 mm × 70 mm × 220 mm 2 specimens
Mix A	6 @ 50 mm	70 mm × 70 mm × 220 mm 2 specimens
Mix B	3 @ 100 mm	70 mm × 70 mm × 220 mm 3 specimens



Figure 3. Buoyance balance for density measurement.

3.3 Density measurement of the cubic specimen

According to BS EN12390-7:2009 (British-Standard-Institution 2009), the volume of specimen can be determined by water displacement method. First determine the mass of the specimen in the water and then determine the mass of the specimen in air. As Figure 3 shown, buoyance balance was used. The formula that was used to calculate the volume of specimen is shown in Equation 3.

$$V = \frac{m_a - [(m_{st} + m_w) - m_{st}]}{\rho_w} \quad (3)$$

where, V = volume of the specimen, in m³; m_a = mass of the specimen in air, in kg; m_{st} = apparent mass of the immersed stirrup, in kg; m_w = apparent mass of the immersed specimen, in kg; ρ_w = density of water, at 20°C.

Density results of three specimens from Mix design B and one specimen from base mix design are listed in Table 3. It was observed that with 40% volume replacement ratio, the density of R-UHPFRC reduced 7.3% when compared to specimen made based on original mix design.

Table 3. Density measurement results.

	Mass (kg)	Volume (m ³)	Density (kg/m ³)
Mix B: 1	2.3566	0.0010378	2270.8
Mix B: 2	2.3344	0.0010263	2274.6
Mix B: 3	2.2818	0.001004	2272.7
Base mix	11.7267	0.0047831	2451.7

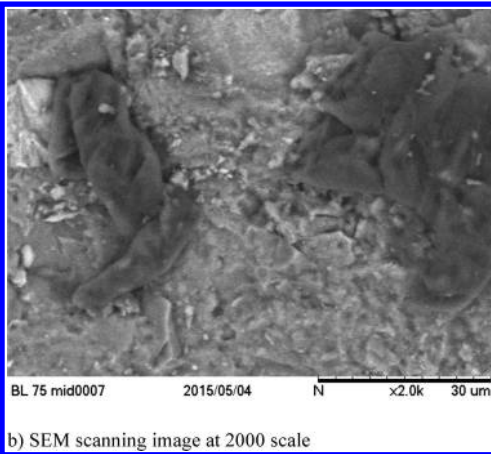
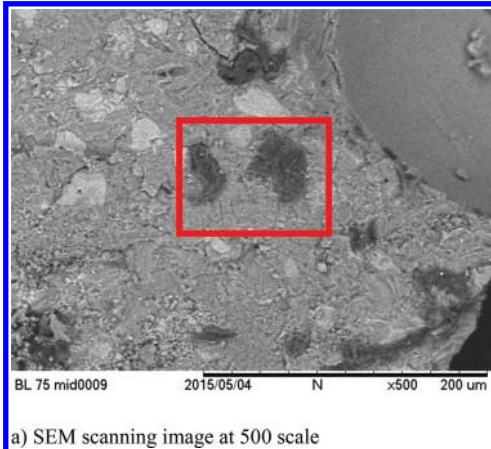


Figure 4. SEM scanning image for specimen in Mix A.

3.4 SEM scanning of rubber particle in the cement matrix

SEM scanning machine was used to examine the integration of rubber crumb particle with the cement matrix. As seen in [Figure 4a](#)), the dark regions are the location of rubber material. The high scaling [Figure 4b](#)) give a closer look at the interface between rubber and cement matrix and no obvious gap was found in the image.

3.5 Compressive strength

The compressive strength were performed using uniaxial compressive machine with capacity of 3000 kN. Loading rate of about 0.8 MPa per second was adopted for all specimens. The compressive strength results and average values were presented in [Table 4](#). The corresponding compressive strength reduction are around 10% for mix design A and almost 50% for mix design B. A typical failure model of the 100 mm cube is shown in [Figure 5](#) for specimens for mix design B.

3.6 Flexural strength

The experiment apparatus are shown in [Figure 6](#) accordingly to ASTM C1609 (American Society for Testing and Materials, 2004). Linear Variable Differential Transformer (LVDT) were installed on both sides of the test specimen, which was used to record the net mid-span deflections. They can also help minimize effects due to seating or twisting of the specimens.

Two side surfaces during casting were chose for loading and supporting purposes to assure the evenness. Specimens were loaded at the third points with displacement control increment of 0.25 mm/min. After load drop to 80% of the peak load, the loading speed was doubled. Flexural hardening effects were observed in all specimens, which means after first flexural cracking, specimens can keep loading with significant higher level of load. Weak sound could be noticed after the specimen cracked, especially toward the failure. The sound was believed to be caused by fibre being pulled out

Table 4. Compressive strength test results.

Mix	Cubes	Compressive strength (MPa)	Average (MPa)
Base	3 @ 50 mm	163.2	165.3
	Normal treated	159.8	
		173.0	
	3 @ 50 mm	215.8	208.3
	Heat treated	213.0	
		196.1	
Mix A	3 @ 100 mm	172.0	182.5
	Heat treated	188.0	
		187.6	
	3 @ 50 mm	184.6	184.1
	Heat treated	185.2	
		182.5	
Mix B	3 @ 50 mm	145.4	141.9
	Normal treated	142.0	
		138.2	
	3 @ 100 mm	95.4	96.6
	Heat treated	108.3	
		86.0	



Figure 5. Failure mode of cube specimens.

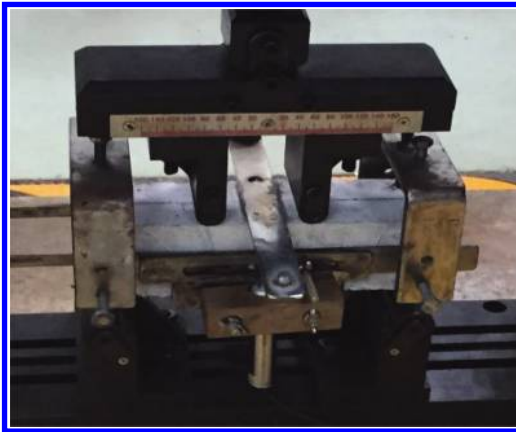


Figure 6. Four point bending test setup.

from cement matrix. In the end, the prism would be failure with one continuously enlarged crack and small cracks at nearby locations.

The load versus displacement curves for all specimens were plotted in following Figure 7. It was observed that while mix design A only show small degradation on flexural strength when compared to the base mix design group, the mix design B with 40% fine sand replacement ratio show a significant

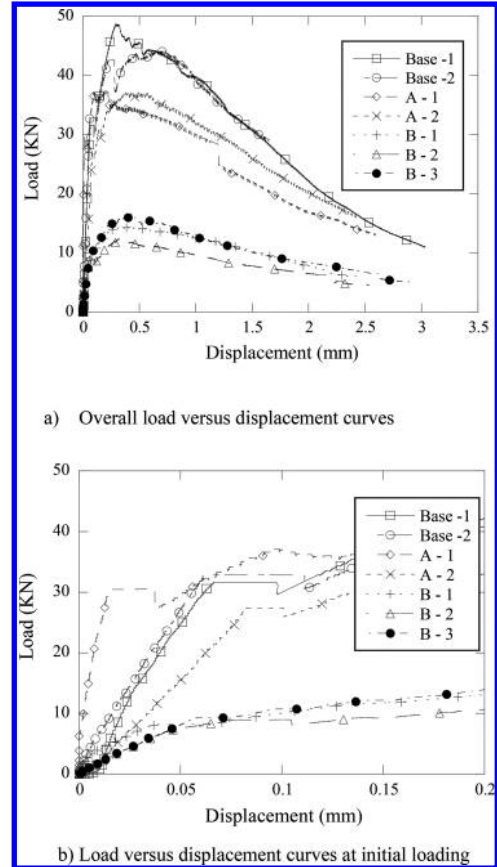


Figure 7. Flexural test results.

reduction in flexural strength. Both the first crack strength and ultimate strength decreased around 60%. The failure mode of specimens from Mix design B can be found in Figure 8.

3.7 Conclusion and discussion

This research investigated the physical and mechanical properties of ultra-high performance fibre reinforced concrete containing recycled rubber crumb. Two different replacement mix designs were investigated. The density of hardened concrete decrease with the inclusion of rubber particles when they are used to replace same volume of fine sand. The compressive strength decrease for both mix designs. The flexural strength of UHP-FRC containing high percentage of rubber crumb decrease significantly, which is different from previous conclusions found in corresponding research conducted on normal strength concrete. From the test results, it was found that for utilizing recycled rubber crumb in UHPFRC, volume replacement



Figure 8. Failure mode of specimens (side view).

might not be a better solution because it break the optimized microstructure of UHPFRC by altering constitutive percentages. In the future, material with different additional amount of rubber crumb will be investigated.

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Literature review on experiment study on RC element with corroded rebar

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ABSTRACT: Corrosion of steel rebars threatens the sustainability of Reinforced Concrete (RC) structures by decreasing the cross sections of the steel bars, causing cover cracking and further undermining the bond between the rebar and concrete. Recent years has witnessed that corrosion has become an urgent issue for RC structures, especially for those in corrosion-inducing environments. Corrosion of steel rebars fundamentally changes the integrity of RC and impairs the designed mechanical properties of RC components. This paper had focused on the review of major experimental studies on corrosion damaged reinforced concrete beams, columns, and joints. The literature review on corrosion-damaged RC components has suggested that the corrosion of steel rebars seriously decreases the strength, deformation capacity, energy dissipation capacity and ductility of RC components. The review on the experimental research has indicated that future research efforts should be put into finding out how to quantify local corrosion and pit corrosion and how they are related to the performance of RC components. Moreover, research efforts could focus on studying the differences between longitudinal rebar corrosion and transverse rebar corrosion. In addition, even though there are many well-proved models to calculate the stress strain relationship of confined concrete, none have taken into account the effect of corrosion. Future studies could focus on making up for the lack of experimental research on confinement behavior of corroded transverse rebars.

1 INTRODUCTION

Corrosion of steel rebar is an electro-chemical process initialized by neutralization of the alkaline in concrete by acidity ingressive chemicals, including sodium chloride, carbon dioxide, sulfur dioxide, etc. The surface of original steel bar is closely adhered by a thin passivation layer, which is generally formed by condensed corrosion products (mainly FeO and Fe_2O_3). This passivation layer could prevent the oxygen from accessing the iron element. The existence of passivation layer critically depends on the high PH value alkalinity. But the acidity from the environment infiltrates into the concrete through pores and touches the surface of the reinforcement, lowering the PH value of the surrounding of the reinforcement and thus neutralizing the alkaline. As a result, acidity damages the passivation layer on reinforcement gradually. Then steel metal is dissolved into metal ion, forming hydrated ferric oxide, generally known as rust. The occurrence of corrosion also relies on the presence of water and oxygen. The unavoidable pores in the concrete would let the water and oxygen penetrate into RC structure from the environment.

The substances potentially causing and accelerating corrosion includes the chloride ion (Cl^-), carbon dioxide (CO_2), sulfur dioxide (SO_2), etc.

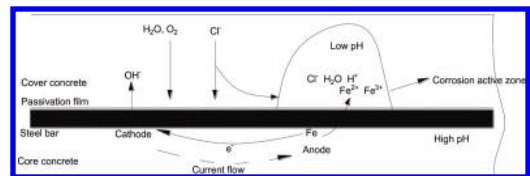


Figure 1. Illustration of chloride attack on steel bar.

The chloride comes from the de-icing salt, coastal vapour, etc. (Li et al., 2009). Chloride is highly active and capable to damage the passivation layer. Figure 1 demonstrates that chloride acts as catalyst in the corrosion process (Neville, 1995).

The physical deterioration of concrete caused by corrosion is divided into two phases, before cover cracking (pre-cracking phase) and after cover cracking (post-cracking phase). The time of cover-cracking is deeply related to corrosion rate, cover depth, corrosion products type, concrete properties, etc. Pre-cracking phase is the process of accumulation corrosion products to the extent of causing concrete cover cracking. Figure 2 demonstrates the relative volumes of each corrosion iron compound to iron metal (Nielsen & Geiker, 2003).

Corrosion products are a mix of different kinds of compounds. And the average volume of

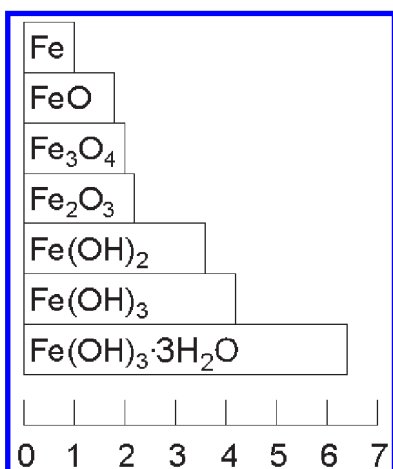


Figure 2. Relative volume of each corrosion iron compound to iron metal.

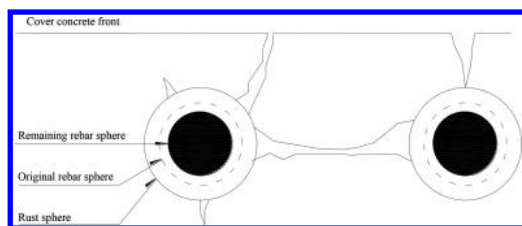


Figure 3. Corrosion induced concrete cracking.

corrosion iron compound is five times the volume of the original metal iron (Bertolini et al., 2004). Thus the expansion of the corrosion products will generate circular stress in the concrete, which could eventually cause cover concrete to crack, as demonstrated in Figure 3.

The bond ensures the concrete and reinforcement will work coherently. When the bond strength is not enough to maintain the connectivity between concrete and reinforcement, the bond-slip failure could occur. The bond between the reinforcement and the concrete is composed of three different kinds of action, namely the chemical adhesiveness, the friction and the mechanical interlocking. Many researchers had found the different trend of bond strength changing in different phases of corrosion (Wang et al., 2011, Wang & Liu, 2004, Lee et al., 2002, Castel et al., 2000b, Ou et al., 2013). In the primary phase, the expansion of the corrosion products enhances the confinement behaviour of the concrete to the rebar. The expansion of corrosion products exerts pressure on the interface between the surrounding concrete and bar, consequently increasing the friction. Moreover, the particle of corrosion products raises the friction

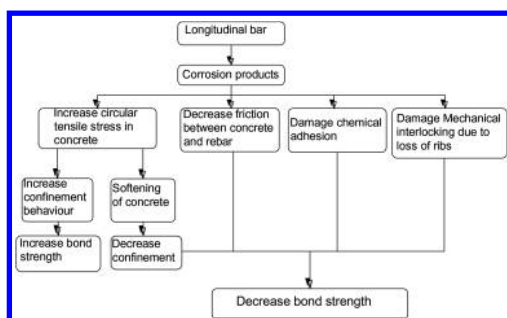


Figure 4. Bond degradation mechanism of rebar corrosion.

coefficient, which is more obvious for smooth bars. The two factors confinement pressure and friction coefficient both contribute to the increase of bond strength in the primary phase. But as more corrosion products accumulate, the chemical cohesion will be deteriorated. It may even lead to the reduction of the friction coefficient. For deformed bar, the ribs could be eradicated, which deduces the mechanical interlocking. And the expansion effect of the corrosion products can lead to the cracking of the concrete cover, conversely resulting in the decrease of the confinement and friction force. Thus the bond strength declines as the corrosion level exceeds the boundary value (generally it is 5%). The following graph better demonstrate the process of bond degradation.

Fang et al. (2004) had conducted experiments on how the transverse reinforcement and the configuration of the reinforcement bar will affect the bond on different level. It has been found that the bond strength of deformed bars without transverse reinforcement declined considerably as the corrosion levels increase (Fang et al., 2004). However, corrosion levels do not affect the bond strength of deformed bars with transverse reinforcement significantly. Compared with uncorroded bars, the bond strength of corroded bar went through a post-peak plateau or residual bond strength. As for smooth bars, at the low to medium levels the bond strength surprisingly increased when the corrosion level increased. This is due to the increase of confinement pressure between concrete and bars when the corrosion levels increase. Another reason could be that the corrosion products between the bar and concrete enhance the friction coefficient, which eventually increase the bond strength. Generally the bond strength of smooth bar without transverse reinforcement dropped faster than smooth bar with transverse reinforcement, as is the case with deformed bar. Therefore, how the bond strength will be affected by the corrosion level generally depends on the confinement effect

of transverse reinforcement and surrounding concrete after being damaged by corrosion.

Ou et al. (2013) proposed a modification parameter based on the corrosion length of reinforcement bars to consider the reduction of bond strength resulted from corrosion (Ou et al., 2013). The modification factor Φ shows the change of L_{eu} , where L_{eu} is referred to equivalent unbounded length of the bar with the strain distribution due to slip elongation S_E from the joint. Equivalent unbounded length L_{eu} can be formed with equation for each steel strain value. The modifier factor to modify strain of the steel is shown in Equations:

$$L_{eu} = \frac{S_E}{\epsilon_s} \quad (1)$$

$$\Phi = \frac{L_{eu}(\text{uncorroded bar})}{L_{eu}(\text{corroded bar})} \quad (2)$$

Two principle methods could be used to obtain corrosion damaged RC component to conduct experimental investigation of mechanical properties of corroded RC structure. One is through natural corrosion; the other is through electro-accelerated corrosion. Natural corrosion generally takes a comparative longer time to expose RC structure to natural corrosion-inducing environment until the corrosion level reaches the designated level (Allam et al., 1994). Coastal environment is often selected because of its high chloride concentration. Another method is to carry out electro-accelerated corrosion, which was favoured by researchers, like (Shi & Niu, 2000, Ma et al., 2012, Gong et al., 2004, Castel et al., 2000b, Dai & Yuan, 2005). The electrochemical nature of corrosion process makes it possible to apply external high voltage current to accelerate corrosion. It could help reach the corrosion level in months, which normally takes years. The Faraday law is used to predict the corrosion rate. This method has proven to be an effective and efficient way to corrode RC manually. Generally researchers tested these corroded RC component under static loading or pseudo-dynamic loading. The following will demonstrate experiments on electric-corroded RC beam, column and joint and the general conclusions.

2 EXPERIMENT RESEARCH ON CORRODED RC BEAM

Castel et al. (2000a) conducted a long-term corrosion experimental study program on corroded RC beams which were 3000 mm in length and with a rectangular cross section of 150 mm by 280 mm. The columns were reinforced with 6 mm transverse stirrup at a spacing of 220 mm and four

12 mm diameter longitudinal rebar. The program was started in the year of 1984 and continued for 13 years. In the experiment two beams were kept in man-made confined salty mist in the and two beams were stored in lab conditions (Castel et al., 2000b, Castel et al., 2000a). All beams were subject to service loading in order to imitate the corrosion environment of RC structures in service. Experimental results indicated that when the compressive zone of RC structural component was not seriously corrosion damaged, the residual bearing capacity was only affected by the cross section area loss of tensile rebars. Comparative analysis between the two groups of beams revealed that corroded beam yielded larger deflection and correspondingly 35% of stiffness loss. Furthermore, the ductility of corroded beams exhibited significantly drop because of local corrosion of tensile rebars. Castel A. also studied the coupling effect of rebar cross section area reduction and bond degradation (Castel et al., 2000a). It was found that the two factors were not working alone and had exceedingly strong coupling effect on the global behaviour of corroded beam. The loss of bond can result in less tensile contribution from concrete and more tensile demand on steel rebar which could greatly contribute to the increase of tensile stress in steel rebar at the notch point, which can lead to post yield when other parts are still in elastic range.

El Maaddawy et al. (2005a) studied the coupling effect of corrosion attack and service loading. Nine beams with a size of $152 \times 254 \times 3200$ mm were included in the experiments. Concrete mixture contains 2.25 percent of chloride ion by weight of cement to simulate chloride penetration. Eight of them are stored in misty confined environment containing sufficient oxygen and moisture, and further exerted with direct current to speed up corrosion reactions. Four of the corroded beams are imposed with 60 percent of their yield load during their corrosion exposure time, which ranges from 50 days to 310 days. In the experiment it has been found that flexural cracking caused by service load can increase the corrosion rate especially at the early stage, promote the generation of corrosion cracking, and raise the maximum corrosion crack width. Also the flexural capacity of beams is almost proportionally reduced by the corrosion level of steel rebar. It has been concluded that the effect of flexural cracking on improving the corrosion level becomes lessened when the corrosion exposure time increases. The test results indicate that beams with great rebar weight loss exhibit appearance of pit corrosion, which seriously reduces the flexural capacity. However, comparison analysis concludes that the influence of pit corrosion on flexural capacity could be weakened by sustained loading on the beams.

Xia et al. (2011) had imposed constant current on stirrup and thus had achieved different corrosion levels in stirrup and longitudinal rebar to investigate the shear capacity of corroded reinforced concrete beam was investigated. Eighteen beams with the dimension of $120 \times 230 \times 1200$ mm were manufactured, among which fifteen beams were corroded using accelerated wet-dry cycle corrosion method. In the cycle, ends of beams are wrapped with sponge which holds 5% sodium chloride solution and applied with current density of 100 cm^2 for 4 days, after the beams are dried for 3 days. Corrosion-induced cracking was recorded. Four point monotonic loading tests were carried out on the all beams. Experiment results demonstrate that average corrosion-induced crack width increase as corrosion level goes up and maximum crack width increases faster than average crack width. Loading test indicates that corroded beam has reduced stiffness when being applied with 20-30% of its maximum load. Shear capacity could be reduced by the cross section area loss of steel rebar, the average crack width and maximum crack width of concrete cover, among which the average crack width has the biggest impact. In the loading test it has been observed that highly corroded beam fail by the fracture of stirrup. This experiment had realistically simulated the corrosion of beam, cover cracking was achieved, and the influence factors of corroded beam on shear capacity have been investigated.

Ou & Chen (2014) had studied the seismic performance of RC beam with corroded transverse reinforcement (Ou & Chen, 2014). Specimens are designed according to ACI 318 Code. Each specimen has one anchorage block end which is used for fixture, while the other end is free end used for exerting cyclic loading. The beam has a dimension of $300 \times 500 \times 1850$ mm. And it has 1.5% of longitudinal reinforcement ratio and 1.8% of volumetric transverse reinforcement ratio. The transverse hoops at the plastic hinge region are linked with direct DC power supplies so as to confine corrosion at the most critical area. In the accelerated corrosion experiment, corrosion-induced cracking is constrained to occur in transverse direction since the longitudinal rebars are isolated from corrosion. Average weight loss of transverse hoop ranges from 0% to 35%. For the most highly corroded beam, pit corrosion is so severe that transverse hoop breaks at several points. Cyclic loading is exerted on the free end of beams with drift ratio from 0% to 6%. The envelope curves of the tests indicate that corrosion has insignificant effect on maximum force. But it also reveals that 6% corrosion level is a threshold value to cause detrimental decrease of seismic resistance. Deformation capacity and ductility could be reduced dramatically by increasing corrosion level over 6% due to the undermined confinement to

concrete core. Corroded hoop also lose the ability to constrain the development of shear cracking. Corrosion-induced crack and loading-induced cracking could work together to cause severe concrete spalling, which leads to decreasing of shear resistance. Furthermore, analysis has concluded that shear resistance should be estimated by considering both the average corrosion level and the minimum cross section area of transverse rebar.

3 EXPERIMENTAL STUDY ON CORRODED RC COLUMN

Shi et al. carried out cyclic loading test on corroded column, and investigated how the axial loading and the corrosion level would affect the seismic performance (Shi & Niu, 2000, Niu et al., 2005, Chen et al., 2005). Ten identical specimens were constructed. The cross section of column was 200 mm by 200 mm, and height was 1100 mm, resulting in an aspect ratio of 5.5. The boundary for the column was prismatic with the dimension of $250 \text{ mm} \times 300 \text{ mm} \times 1100 \text{ mm}$. The columns were reinforced by six longitudinal bars with diameter of 14 mm and stirrup with diameter of 6 mm at the spacing of 80 mm. The boundary element, which represented the discontinuity, was reinforced by six longitudinal bars with diameter of 16 mm and stirrup which was the same as that of the column. The compressive strength of the concrete was 20.1 MPa. The yield strength of the longitudinal bar was 415.6 MPa and the stirrup bar would yield at the strength of 327.0 MPa. In order to best simulate the corrosion process in the real environment and carry out load test afterward, and prevent the bond failure at the discontinuities, the boundary element and the column top were covered with Paraffin. And the specimens were exposed to outdoor environment for two and a half years. And then they were immersed in NaCl Solution with the concentration of 5%. The current was charged for seven days, subsequently drawn out the liquid. Then the specimens were kept in outdoor natural environment without accelerated corrosion for seven days. Then repeated the procedure until the calculated corrosion levels had been reached. The actually corrosion level was calculated by weighing the bars which had been cleared of corrosion products after the loading test. The columns were exerted on reversed cyclic loading while a constant axial load was applied on the top. The axial load ratios included 0, 0.2 and 0.4 to research on how axial load affected the response. More detailing of the specimen is shown in Appendix A. The following conclusions were drawn from the test:

1. The extent of damage on the column depends on the duration of accelerated corrosion. The

three major causes of corrosion damage include the deterioration of mechanical properties of rebar, the concrete cover cracking, and the degradation of the bond strength.

2. The increase of corrosion level would exacerbate the degradation of the yield strength, the stiffness, the ductility, the energy dissipation.
3. The corrosion of rebar could change the failure characteristic of the RC columns. Compared with the sound RC column, corroded column demonstrated more apparent signs of brittle failure.
4. The impact of corrosion on the strength, stiffness, ductility, energy dissipation of corroded RC columns was similar to that of sound columns.

Gong et al. (2004) had conducted experimental research on lateral cyclic behaviour of sixteen corroded columns subjected to constant eccentric compressive loading (Gong et al., 2004, Li et al., 2009). The experiment produced performance results of the columns, with regard to ultimate capacity, stiffness, ductility, and energy dissipation capacity. It had been found out that the accumulative energy dissipation capacity would decrease from 11.5 kN.m to 1.5 kN.m and ductility ratio decline from 6.29 to 3.96 as the reinforcement corrosion level increased from 8% to 19.8%. The hysteretic loops had an apparent form of fusiform and asymmetry. Moreover the transition of failure mode from ductile failure to brittle mode had been observed. The phenomenon that stirrup underwent more severe corrosion attack than longitudinal reinforcement conformed to the real life inspection results of the engineering structures. More details about the specimens were tabulated in Appendix A. Li and Gong had done future researches on using FRP and steel jacket to retrofitting the corrode specimens. The following general conclusions were derived from the analysis of test results:

1. The corrosion of reinforcement had contributed to the change of failure modes of columns under constant eccentric axial load and lateral cyclic loadings. The reduction ratio of ultimate strength, stiffness, ductility, and energy dissipation of corroded columns relies on the corrosion level of reinforcement.
2. The non-uniformity of the reinforcement corrosion had brought about the asymmetry of hysteretic loop. The severity of stirrup corrosion decreased confinement behavior and shear capacity, thus exacerbating the degeneration of the ductility of the specimens.

Wang and Liang (2008) researched on the mechanical behaviour of reinforced concrete columns with partial length corrosion (Wang & Liang, 2008). Twelve columns were partially corroded in

tensile or compression zone with certain length before being eccentrically loaded. The specimen was shown in Appendix A. Through the test, two major discoveries had been made as follows.

Longitudinal and transverse reinforcement of RC columns with partial length corrosion would transform the performance under eccentric loading. The test showed the columns had reduced ultimate load and brittle failure behavior. Compared to columns with partial corrosion in the compressive zone, columns with corrosion in tensile zone had greater reduction ratio of load capacity when under large eccentric loading. And columns with tensile zone partially corroded experienced greater load capacity reduction under small eccentric loading.

Within the partial corrosion length, the crack resulting from loading had emerged with longitudinal corrosion cracks. The corrosion of stirrups deteriorated the confinement at the corroded zones. These greatly reduced the mechanical integrity of the column and created a weak zone within the whole length of the column, which significantly reduced the strength of the columns.

Ma et al. (2012) had investigated on how reinforcement corrosion would change seismic behaviour of circular columns (Ma et al., 2012). Thirteen RC circular columns were included in the experiment. The columns were electro-accelerated corroded on different levels ranging from 0% to 15.1%, after which the columns were subject to constant axial load and cyclic lateral displacement. Ma Y. had proposed different equations for loading capacity, stiffness, ductility, and energy dissipation of the columns based on the experiment results. The equations established linear relationship between the seismic behaviour of the columns with corrosion levels, as is shown below.

$$F_y = F_{y0}(1 - 0.885\rho) \quad (3)$$

$$F_u = F_{u0}(1 - 0.878\rho) \quad (4)$$

$$\bar{\eta} = \bar{\eta}_0(1 - 1.182\rho) \quad (5)$$

$$\mu = \mu_0(1 - 1.795\rho) \quad (6)$$

$$E_{sum} = E_{sum0}(1 - 4.789\rho) \quad (7)$$

where F_y , F_u , $\bar{\eta}$, μ is the yield strength, the ultimate strength, the average normalized stiffness and ductility of columns with corrosion level ρ ; F_{y0} , F_{u0} , $\bar{\eta}_0$, μ_0 is the yield strength of uncorroded column. Furthermore, the following conclusions were drawn.

1. Highly corroded RC columns under high axial load experienced brittle failure in the test. Corrosion could change the failure behavior of columns under mild axial load.

2. Poor hysteretic response, reduced stiffness, degraded ductility and steep descending branch in envelop curve were observed in columns with axial load ratio as high as 0.4 and corrosion level from 10% to 20%.
3. Columns with corrosion loss ratio below 14% showed increased cumulative energy dissipation as the ultimate displacement increased. They had almost the same energy dissipation capability with the uncorroded columns.

4 EXPERIMENT RESEARCH ON CORRODED RC JOINT

Dai & Yuan (2005) had carried out cyclic reverse loading tests on exterior beam-column connection. Two connection specimens were electric-accelerating corroded. Then the same loading history was exerted on all the specimens. Test results of the corroded joints were compared against the control uncorroded specimen. The ratio is 1:3. Each specimen was composed of beam with cross section of 100 mm × 200 mm, and column with cross section of 150 mm × 150 mm. More details are shown in Appendix A. It had been found out that bond strength deterioration and concrete valid cross section reduction would seriously degrade the seismic capacity of exterior beam-column joints (Dai & Yuan, 2005). Other general conclusions were as follows:

1. Flexural crack would develop and connect with the corrosion crack as a consequence of the reversed loading action. The concrete was subdivided by these cracks, which resulted in serious cover spalling.
2. At the connection the spalling of concrete and slip of reinforcement bars caused the rebar to pop out, demonstrating the characteristic of shear failure.
3. Corrosion of reinforcement would result in bar area decrease and concrete valid cross section reduction, as well as the bond strength degradation between concrete and longitudinal bars, which all contributed to the decline of stiffness and seismic strength. The deformation capacity of the corroded beam-column joints was seriously reduced.

Liu (2010) had conducted research on the seismic behaviour on corroded beam joints (Liu, 2010). His experiment included five specimens which were designed according to common frame design specifications. Based on the actual corrosion rate, the corrosion levels for each beam joints specimen were 0%, 3.19%, 5.77%, 9.12% and 12.21% respectively. Cyclic loading were exerted on the specimens with the objectives to find out how the corrosion level

affected the failure mode, hysteretic loop, and steel stress-strain curve. Based on the results, Liu (2010) had put forward formula to calculate the strength and ductility of beam joint with the consideration of corrosion damage. The test concluded that:

1. Before the test, the corrosion cracks were seen in the specimens. During the test, more rapid and wider crack grow were observed in the plastic hinge of joint beam as the corrosion level increased.
2. The area enveloped by the hysteretic loop was reduced by the increase the corrosion level. At the high corrosion level, the load-displacement curve was changed from fat robust shape to thin unstable S shape, which clearly demonstrated the decrease of energy dissipation capacity. The pinching effect was very apparent in highly corroded specimens.
3. The strength and stiffness of joint beam were reduced by the corrosion, for which there are three major reasons. Firstly, non-uniform corrosion had made pits on the bar, which could give rise to stress concentration. Secondly, corrosion had reduced the cross section area of rebars. Thirdly, the corrosion products expanded and caused the cracking of cover concrete, which results in the reduction of valid cross section of the RC.

5 CONCLUSIONS

This paper has covered the material degradation caused by corrosion of steel rebar in reinforced concrete structures. The mechanism of steel corrosion in concrete is an electro chemical process, which is often accelerated by the ingress of aggressive chemicals. Chloride attack is one of the most dominant factors stimulating rebar corrosion and thereof causing serious structure deficiency. Submerging reinforced concrete in sodium chloride (NaCl) solution and applying the direct current on steel bar makes a more efficient method to obtain corrosion-damaged concrete than natural corrosion. The accumulation of corrosion products on steel rebar is able to generate circular stress which could cause cover cracking. Cover cracking will reduce the cross section of the concrete component and degrade bond between concrete and steel bar. Corrosion of steel rebar will degrade the physical appearance and reduce its original cross section.

The paper has reviewed on major experimental studies on the corrosion damaged reinforced concrete beam, column, and joint. Accelerated corrosion or natural corrosion methods had been employed to generate corrosion on steel rebar. Different kinds of loading test had been conducted on different types of RC elements. The test results

have demonstrated that the reason for the mechanical degradation of corroded RC element lies in three kinds of material deteriorations, corrosion-induced concrete cracking, the reduction of rebar cross section and the bond strength at the interface between concrete and rebar.

Corrosion-damaged element has smaller stiffness and strength. Besides, corrosion could have a major negative impact on the energy dissipation and ductility of column under axial compression and lateral loading. It also discovered that corrosion of steel rebar have the potential to change the failure modes of column from ductile to brittle. In general corrosion is non-uniform and localized in morphology. The localization of corrosion leads to asymmetry of hysteretic loop and reduction of seismic resistance.

The strong coupling effect of rebar cross section area reduction and bond degradation has been studied by testing on corroded RC beams (Castel et al., 2000a, Castel et al., 2000b). The corrosion-induced bond loss can result in less tensile contribution from concrete and the increase of tensile stress in steel rebar. El Maaddawy et al. (2005) found sustained load will increase the corrosion at the early stage and pit corrosion is the main reason for the reduction of flexural capacity when the corrosion level is high. Xia et al. (2011) discovered that shear capacity could be reduced most effectively by the average crack width and highly corroded beam fail by the fracture of stirrup. Ou and Chen (2014) had concluded that corroded hoop also lose the ability to constrain the development of shear cracking.

Partial length corrosion has detrimental effect on the performance of column under eccentric loading (Wang & Liang, 2008). Compared to columns with partially corrosion damaged compressive zone, columns with corrosion in tensile zone had greater reduction ratio of load capacity when under large eccentric loading, while columns with tensile zone partially corroded experienced greater load capacity reduction under small eccentric loading. Test results imply that the cause is that corroded stirrups deteriorated the confinement behaviour at the corroded zones, which results in damage of the mechanical integrity and creation of weak zones within the whole length of the column.

Experiments have shown that corrosion-damaged joint could seriously reduce deformation capacity because of bond loss, valid concrete area loss, and rebar area loss (Dai & Yuan, 2005). Flexural crack would develop and join with the corrosion crack as a result of the cyclic loading action. Furthermore it causes serious cover spalling at the joint. Results indicate cover spalling and rebar slip could cause brittle shear failure to corrosion-damaged joint. As the corrosion increases, the load-displacement curve of joint under cyclic loading was changed

from fat robust shape to thin unstable S shape, which clearly demonstrated the decrease of energy dissipation capacity. The pinching effect was very apparent in highly corroded joints.

Literature review on corroded RC element experiments and the above discussion has insinuated that the corrosion of transverse rebar affects mechanical confinement on concrete (Castel et al., 2000b, Dai & Yuan, 2005, Wang & Liang, 2008, Ou & Chen, 2014, Xia et al., 2011). Transverse rebar has great significance on confined concrete strength and stress-strain relationship, which closely relates to the performance of RC element. Transverse rebar is the key element for providing shear capacity for RC element (Xia et al., 2011). Corroded transverse reinforcement has reduced cross section area, which results in decreased confinement force to concrete. Furthermore, pitting corrosion on transverse rebar can leads to yielding only at some localized point and thus decrease of deformation capacity and loss of ductility of concrete (Higgins & Farrow III, 2006). But even though there are many mature models to calculate the stress strain relationship of confined concrete, none have taken into account the corrosion of steel rebar. There is a lack of experimental research on confinement behaviour of corroded transverse rebar. Experimental study also shows that corrosion is more than just rebar cross section reduction. The localization of corrosion could cause major changed in the performance. Further, limited researched had been conducted to investigate how corroded longitudinal rebar and stirrup affect the performance of RC component independently.

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Parametric analysis on fire resistance of Angle Steel Strengthened Reinforced Concrete (ASSRC) columns

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ABSTRACT: Angle steel reinforcing is an effective, economical and easily applied strengthening method which was widely used for RC (Reinforced Concrete) columns. However, the traditional mortar proof for the angle steel has a high probability to flake off when exposed to fire, which leads to the poor fire resistance behavior of Angle Steel Strengthened Reinforced Concrete (ASSRC) columns. To investigate the fire behavior of ASSRC columns, finite element modeling was carried out to do fire-structure coupling analysis of ASSRC columns under ISO834 standard fire using ABAQUS, which was validated using some experimental test data. Then the validated model was used to analyze the temperature field distribution, time-axial displacement curves, failure patterns and fire resistance of ASSRC columns. A parametric study was carried out to investigate the influence of different parameters on the behavior of ASSRC columns exposed to fire, including load level, eccentricity ratio, fire protection thickness, axial compression ratio and increasing coefficient of carrying capacity. The results showed that, epoxy resin between the angle steel and internal RC columns reached its failure temperature in a short time and lost both bond strength and shear strength. It is the start of the plastic behaviour of the hinges of the steel angle at the critical temperature that caused the damage of ASSRC columns. When other parameters remain constant, the higher the load level, the less the fire resistance will be, corresponding with shorter times for the angle steel to reach the critical temperature. With increasing eccentricity ratio, the angles on the eccentric direction side will take on more compressive load that causes the plastic hinges to form earlier, resulting in a decrease in fire resistance, but not so effective. The fire protection of mortar can retard the heat transmission process, hence the thickness of protection has a notable influence on the time to reach the critical temperature of the steel angle. Increasing the coefficient of carrying capacity has little effect on the fire resistance of ASSRC columns, which can be explained by the fact that the critical temperature of steel angles was almost the same under the condition of the same load level. In comparison with the axial compression ratio, the carrying capacity ratio should be a more suitable parameter to indicate the combined influence of axial load ratio and eccentricity ratio during fires.

1 INTRODUCTION

Angle steel strengthening is one of the mature methods presently available for strengthening RC columns. The method involves the use of longitudinal angle steel fitting to the corner of the RC columns while transverse steel strips are welded (Adam et al. 2009, Garzón-Roca et al. 2012). Although this kind of strengthening method was widely used in practical engineering, we have little understanding about the fire resistance of the ASSRC columns (Adam et al. 2009, Garzón-Roca et al. 2011).

Literature review shows that some research work has been done on strengthened structure members. Wu et al. (Wu 2009, He 2006, Sun 2007, Gao et al. 2010) studied the fire resistance of RC beams strengthened by CFRP, both experimentally

and numerically. Yang (2008) has discussed the fire-resistant performance of carbon fiber reinforced concrete members. A detailed study of fire-resisting design of CFRP strengthening RC beams was carried out by Hu et al. (2013). Zhang (2014) carried out a numerical simulation on fire performance and fire-resistant protection of ASSRC columns, in which bounding between the steel angle and concrete was assumed to be perfect, so the melt of epoxy resin and the slip between steel and concrete did not take into account. As far as we are aware, no experimental results has been given relate to any of the fire resistance of ASSRC columns.

In this paper, a FE model was established to investigate the influence of related parameters on fire performance of ASSRC column considering the slip between steel angle and concrete.

2 FINITE ELEMENT MODEL

2.1 General view of columns

Total length of each column was 3860 mm with both ends hinged. Detail dimensions of specimens were showed in Figure 1. Cement mortar with a thickness of 25 mm outside the concrete column and steel angles was functioned as fire protection. Four gusset plates with dimensions of 244 mm × 200 mm × 4 mm were welded between steel angles with the objective of avoiding early failure of the thin component (Adam et al. 2009).

2.2 Brief introduction of FE model

The ASRRC columns were numerically simulated by ABAQUS program. The concrete, cement

mortar and steel plate at each end of column were modeled using hexahedral elements with 8 nodes and 3 degrees of freedom per node, with reduced integration and hourglass control (C3D8R). The longitudinal and transversal reinforcement were modeled using 2-node linear displacement truss element (T2D3) that can transmit only axial force. Steel angles and strips were modeled using quadrilateral shell elements (S4R) with 4 nodes and 6 degrees of freedom per node, allowing the simulation of buckling.

2.3 Interface interaction and boundary conditions

During the temperature field calculation, ASRRC columns were exposed to ISO834 standard fire from 4-sides. A value of $25 \text{ W/m}^2 \cdot \text{K}$ was taken for the convection coefficient. Radiation coefficient

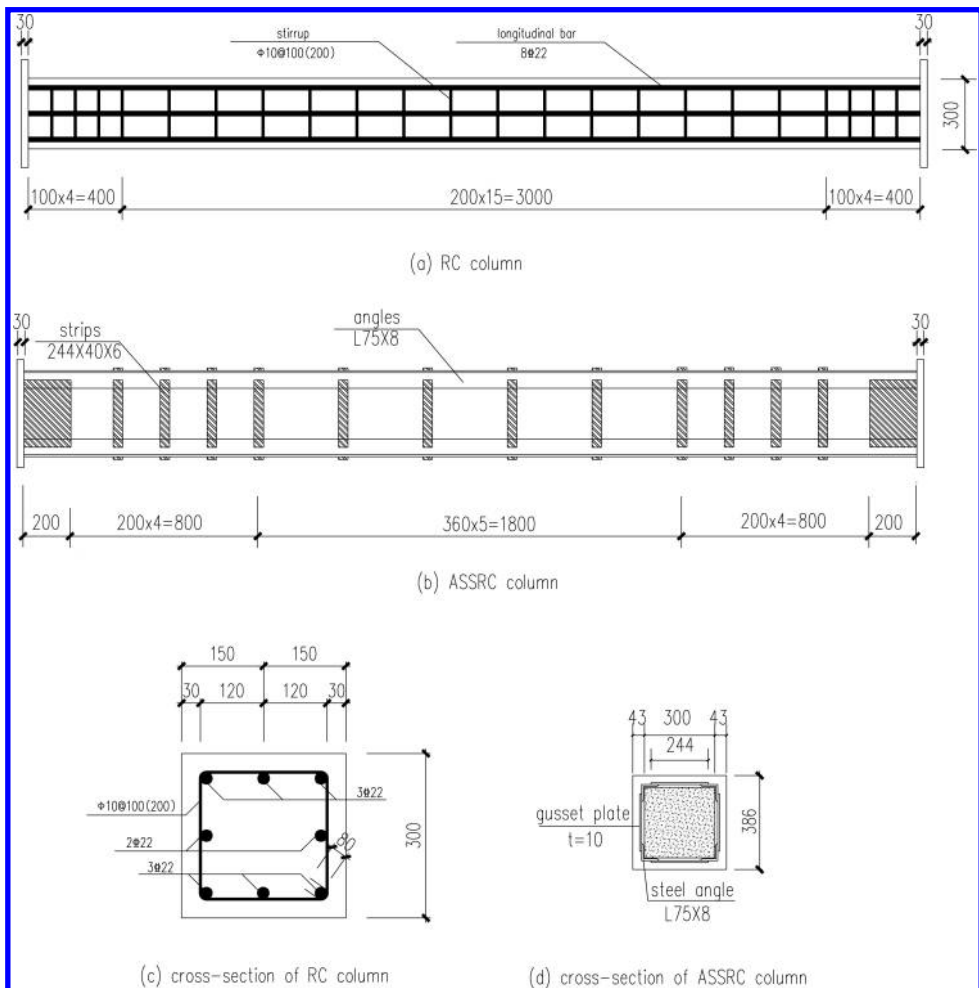


Figure 1. Specimen geometry and reinforcement (dimensions in mm).

and Stefan-Boltzmann coefficient are 0.5 and $5.67 \times 10^{-8} \text{ W/m}^2 \cdot \text{K}^4$, respectively. Interface between concrete and strengthening steel, strengthening steel and cement mortar, concrete and steel plate, concrete and reinforcement was coupled by means of the TIE option.

When doing fire-structure coupling analysis, ASRRC columns was hinged at both ends and an axial load was applied with a certain value which kept constant for the duration of the test. The reinforcement was embedded in the concrete by means of the embedded region option, assuming a perfect bond between the bar and the concrete. Angle steel was fixed to the steel plate using Shell-to-solid coupling. Contact between concrete and steel plate was considering only normal direction as “hard” contact (Giménez et al. 2009). When the temperature of epoxy resin exceeds its glass transition temperature (150°C), adhesive lose both bond strength and shear strength (Blontrock et al. 1999).

Hence, the behavior between strengthening steel and concrete in the normal direction is modeled as “hard” contact, in the tangential direction is modeled with the classical isotropic Coulomb friction model with a friction coefficient of 0.3.

2.4 Property of material

Thermal properties and constitutive model of concrete under different temperature such as thermal conductivity, specific heat and expansion provided by Lie (Lie & Denham 1993) was used, as well as constitutive model of steel. Thermal properties of steel were calculated as Eurocode3 and Eurocode4. Compressive strength of concrete (f_c) is 25 MPa and poisson ratio was assumed to be 0.2 and 0.3 for concrete and steel at room temperature, respectively. Yield stress for strengthening steel, longitudinal bar and stirrup was 235 MPa, 400 MPa and 300 MPa, respectively at

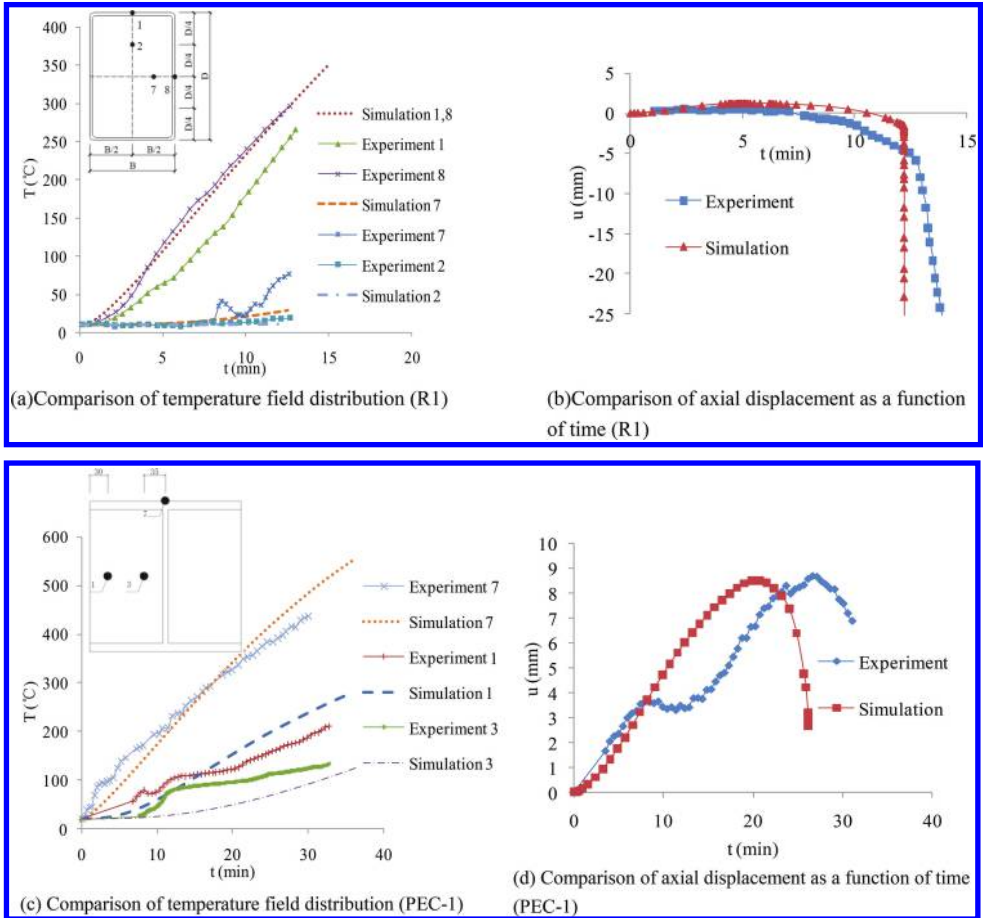


Figure 2. Comparison of finite element results with experimental results.

room temperature. Cement mortar was taken no account of the mechanical property and assumed the same thermal properties as concrete.

2.5 Verification of the finite element models

A comparison between the experimental results and finite element results was carried out to verify the finite element model, as shown in Figure 2 (a) and (b). Because there is no experiment result relate to fire resistance of ASSRC columns, test result of specimens R1 in reference (Liu 2010) and PEC-1 in reference (Man 2012) was chosen to make the validation. As can be observed in Figure 2, results from FE model agree well with the experimental results, both the temperature field distribution and the axial displacement curve as a function of time.

3 PARAMETRIC STUDY

Base model (ASRRC-0) for the parametric study was strengthened by $L75 \times 8$ angle steel with an axial load level of 0.4 and a fire protection thickness of 25 mm. Influence of related main factors such as load level, eccentricity ratio, fire protection thickness and increasing coefficient of carrying capacity were investigated.

Figure 3 shows the damaged model of ASRRC-0, that is, when ASRRC-0 get to its fire resistance, buckling occurred in the angle steel, leading to the result that angle steel losing confinement effects to internal concrete. The concrete near the buckling area was compressed to crash immediately. Hence, local buckling of the angle steel is a key issue.

3.1 Influence of load level μ

Figure 4 shows the axial displacement as a function of time for ASSRC columns with different load level μ (ratio of the applied axial load to the ultimate load at room temperature).

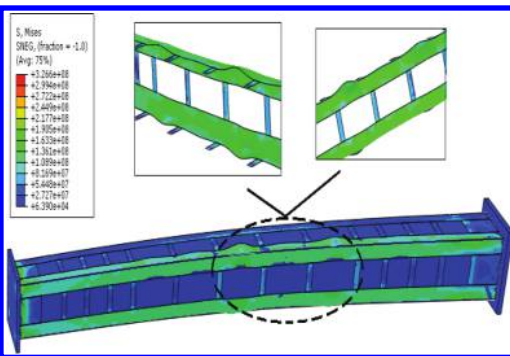


Figure 3. Buckling of steel angle of ASRRC-0.

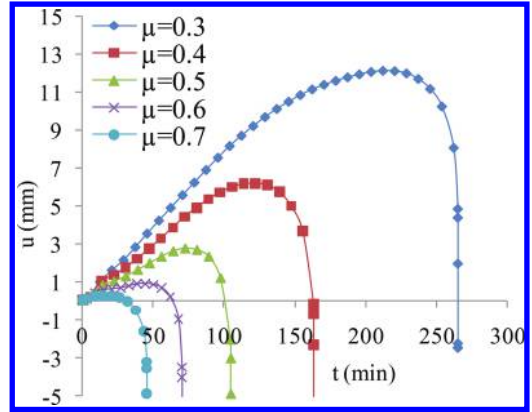


Figure 4. Axial displacement as a function of time under different load level μ .

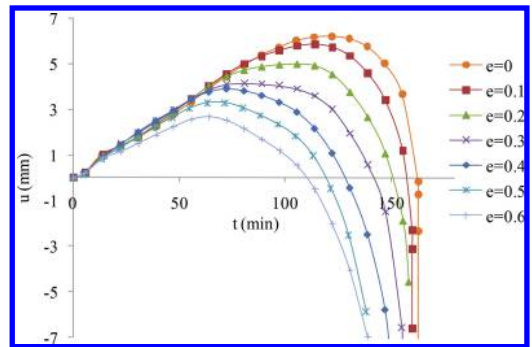


Figure 5. Axial displacement as a function of time under different load eccentricity e .

It can be seen that, higher the load level, less fire resistance and less ultimate expansion displacement will be. It is due to the fact that higher load level increase the axial load in angle steel and concrete, which cause the angle steel buckling earlier under elevated temperature condition.

3.2 Influence of eccentricity ratio e

Figure 5 shows the axial displacement as a function of time for ASSRC columns under different eccentricity ratio e (ratio of the eccentricity to the half of the side length of ASRRC column cross-section) with the same μ value of 0.4.

It can be seen that, higher the eccentricity ratio, less ultimate expansion displacement fire resistance will be, but the influence is not significant. The reason may be that the higher eccentricity ratio, the angles fixed at the eccentric direction will carry more compressive load. Consequently, it will buckle earlier under the same elevated temperature condition.

It's noteworthy that following the eccentricity ratio getting higher, the compressive load applying at the end of the ASRRC column should be reduced to keep the certain load level because of the lower ultimate load of columns.

A given axial compression ratio have a certain compressive load corresponding to different eccentricity ratio, this load even exceed the ultimate load of the ASRRC columns with a large enough eccentricity ratio. So comparing with axial compression ratio, carrying capacity ratio should be a more suitable parameter to indicate the combined influence of axial load ratio and eccentricity ratio under fire condition.

3.3 Fire protection thickness b

Using cement mortar as fire protection can retard the heat transmission process. Figure 6 shows the fire resistance of ASRRC columns when the thickness of cement mortar was the only parameter to be changed. Obviously, with certain load level and eccentricity ratio, thicker fire protection accompanies greater fire resistance of ASRRC columns and exhibits a good linear relationship between each other. The critical temperature (the temperature when the ASRRC column gets its fire resistance) of the angle steel is given in Figure 7. It's easily to find out that, under the condition of controlling other variables, the critical temperature of each angle steel is nearly same.

3.4 Influence of increasing coefficient of carrying capacity β

β is a parameter to represent the strengthening level of steel angle comparing with the carrying capacity of un-strengthened RC column. It can be calculated as the ratio of the ultimate load after strengthening to the un-strengthened one. In this section, different size of angle steel was used in models to get different β (L75 $\times$ 8, L90 $\times$ 10,

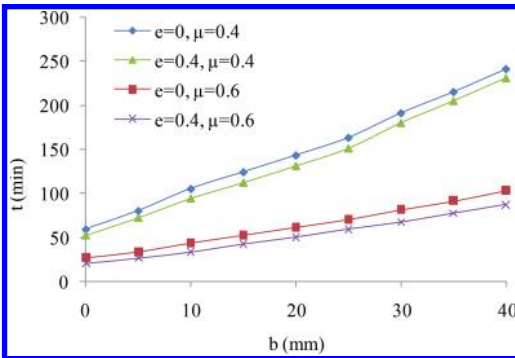


Figure 6. Influence of cement mortar on fire resistance.

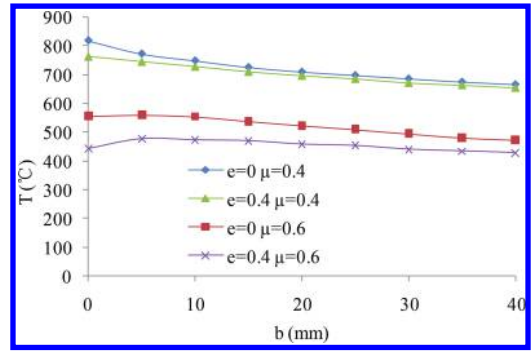


Figure 7. Influence of cement mortar thickness on critical temperature.

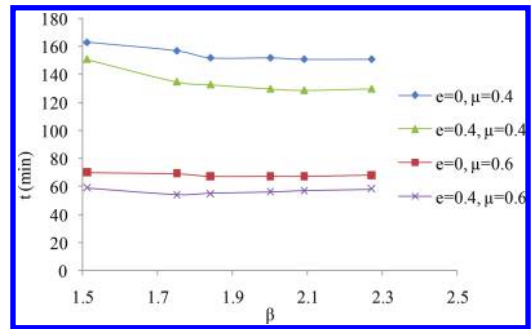


Figure 8. Influence of increasing coefficient on fire resistance.

L100 $\times$ 10, L100 $\times$ 12, L110 $\times$ 12, L110 $\times$ 14 corresponding to β value of 1.51, 1.75, 1.84, 2.00, 2.09, 2.27, respectively).

It can be found from Figure 8 that, changing the increasing coefficient of carrying capacity has very little influence on the fire resistance of the ASRRC columns when other parameters remain constant. Due to the good thermal conductivity of steel, the size of angle steel hardly affects heat transmission of the columns so that ASRRC columns with different β have the almost same temperature field. It means the critical temperature of angle steel in ASRRC columns with different β is almost same leading to the nearly same fire resistance.

4 CONCLUSION

It is the local buckling of steel angle at the critical temperature that caused the damage of ASRRC columns. Steel angle lose carrying capacity and confinement to the concrete under elevated temperature conditions.

Under fire condition and other parameter constant: (a) Higher load level brings higher axial stress in the steel angles, which leads to steel angles reach its critical temperature earlier corresponding to a poorer fire resistance; (b) The steel angles fixed at the eccentricity direction will carry more axial load which causes buckling appear earlier; (c) Improving the thickness of fire protection retard the heat transmission process, it takes more time for angle steel to heat up to its critical temperature; (d) Increasing coefficient of carrying capacity hardly affect the fire resistance of ASRRC columns, with certain load level and eccentricity ratio, the critical temperature of different size of steel angles is close.

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Sustainable steel structures

Seismic performance of multi-story self-centering structures with angles used as energy-dissipaters

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ABSTRACT: The structures of green buildings, or sustainable buildings, do not mean that they never be damaged by any severe unusually loads, but means that they can recover from these damages quickly and economically. To buildings in high seismic zones, self-centering structural system maybe the best choice to have their structural function recovered after a large earthquake attack. To understand the seismic performance of multi-story buildings with beam-to-column self-centering joints and angle steel energy-dissipaters, 3-story structural models are built and investigated. The complicated self-centering and energy-dissipating features of the beam-to-column connection joints with pre-stressed strands and bolted angles were simplified as a series of spring links based on their force-deformation curves. While structural members as columns and beams were simulated as fiber elements. Both monotonic loading and cyclic loading were applied to these 3-floor 1-bay self-centering steel frames in OpenSees. Parameters which may influence the structural behavior, such as angle size, cross section area and vertical layout of steel strands, are analyzed. The results shows that steel strand provides the main part of bending-moment bearing capacity for connection joints as the gap between the beams and column appears, the space between the strands has no effects on bearing capacity of structure for symmetrically strands layout, the initial lateral stiffness of self-centering steel structure is correlated to the sectional area of the strands but not influenced by the initial stress of the strands. The thickness and size of angel legs affect not only the energy-dissipation capacity but also the load carrying capacity of self-centering structures.

1 INTRODUCTION

If green buildings, or sustainable buildings, mean that their structures never be damaged by any severe unusually loads, this kind of buildings will be too expensive to be built. So, on the probability sense, when green buildings suffer to some very rare encountered loads, they maybe damages but they can recover from these damages quickly and economically.

At present, structures designed according to current codes may easily achieve the design goal of collapse-prevention under large earthquake events. But to a less severe earthquake, such as frequently-encountered earthquake, or design-base earthquake, most current codes requires structures should be repaired from some damages. Currents codes allows structural main members, such as beams and columns, and beam-to-column joints to enter plastic load-bearing states, and relies on their plastic deformation to dissipation earthquake energy. This makes structures have permanent drift and damages, and makes it impossible to be repaired after earthquake (Kurobane et al. 1995). To solve this problem, many new structural systems have been tried and studied by some scholars and engineers. Self-centering system (Garlock 2002,

Garlock et al. 2005, 2007, Pan 2010, Pan et al. 2011, McKenna & Fenves 2000, Pan et al. 2010) is one of these new systems.

Self-centering systems can return to their original position and concentrate damages to some easy-to-be-replaced members after earthquakes. System with beam-to-column self-centering joints and using angle steel as energy dissipaters is one of these innovated systems. But their study is mainly focus on joints and small parts of the whole structures. Studies on the whole structure system are quite few. So a multi-story self-centering system will be studied in this article.

2 SPECIMEN DESIGN

Three series of 3-floor 1-bay self-centering models using angle energy dissipaters are designed. Analyses are done to get a wider knowledge of the performance of self-centering frames. Shapes of BASE specimen and details of the self-centering connections are show in the [Figure 1](#). Q345B steel is used for beams and columns. The sections of beams and columns are $H500 \times 200 \times 10 \times 16$ and $H500 \times 450 \times 16 \times 25$ respectively. The angles $L300 \times 250 \times 16$ and $L300 \times 200 \times 8$ are specially

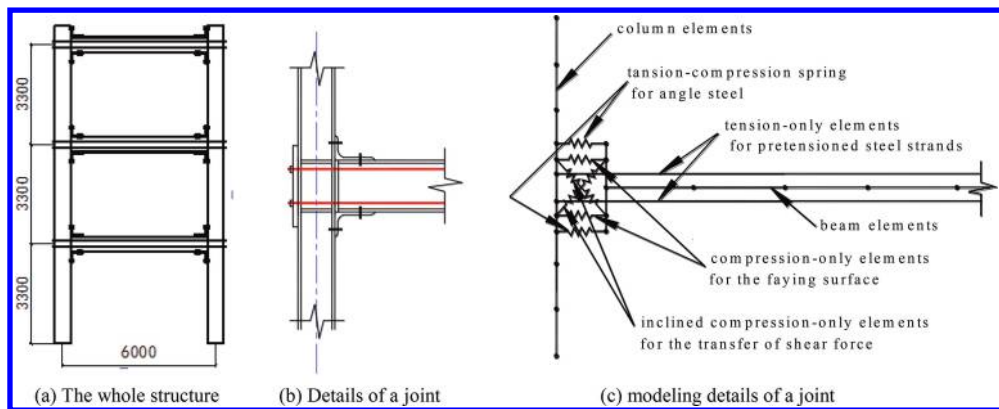


Figure 1. Details of the 3-floor model.

Table 1. Details of all specimens.

Specimen	S/mm	A_T/mm^2	n	$\sigma_0/\text{N/mm}^2$	Angle steel
BASE	100	370	2	500	$L300 \times 250 \times 16$
JL1	250	370	2	500	$L300 \times 250 \times 16$
JL2	400	370	2	500	$L300 \times 250 \times 16$
JM1	100	370	2	280	$L300 \times 250 \times 16$
JM2	100	370	2	615	$L300 \times 250 \times 16$
JM3	100	660	2	280	$L300 \times 250 \times 16$
JG1	100	370	2	500	$L300 \times 250 \times 8$
JG2	100	370	2	500	$L300 \times 250 \times 24$
JG3	100	370	2	500	$L200 \times 180 \times 16$

Note: S : steel strand spacing; A_T : sectional area of one strand; n : number of strands per row; σ_0 : initial tensile stress of one strands.

made by cutting from some H-type sections. Four post-tensioned strands are arranged along the axial line of the beam on both sides of the beam web. The distance between the steel strands on both sides is 90 mm.

The BASE specimen is used as a comparative one to all specimen series. Other specimens are fall in three series of JL, JM and JG. The group JL is designed to study the effects of strands layout on the structural seismic performance by changing the distance between two adjacent rows of strands. The second group JM is chosen to analyze the effects of the sectional area and initial stress of the strands on the structure. Since structural frame is designed to remain elastic, it is believed to have no much effect on the whole system. So no study of the influence of the structural members is carried out. The last group JG is designed to show the effect of the thickness and size of angel legs. Each group has some key parameters varied. Details of all specimens are listed in Table 1.

3 SIMULATION METHOD

The two-dimensional 3-floor 1-bay structural system is modeled in OpenSees. Figure 1(c) shows some details of the modeling. Beams and columns are modeled using material and geometry nonlinear beam-column elements. At the beam-to-column connection area, an end beam, whose length is the height of the beam section, is used to model connection contact area between beam end and column surface. All the nodes between beam end and column surface are connected by compression-only elements. The energy-dissipation effect of angle steel is modeled by tension-compression spring elements. The post-tensioned strands is represented by tension-only elements and fixed at both ends on the column by sharing same nodes. The shear force between beam and column is resisted by inclined compression-only elements. Column ends are simply supported to avoid damage. For column end damage is difficult to repair, in self-centering system, this kind of column connection is used. Horizontal cyclic load, which is controlled by displacement, are applied at the top of column. The load is applied as displacement. Which is composed of 10 mm, 30 mm and $(30 + n \times 40, n = 1, 2, 3 \dots 9)$ mm. Every loading step is repeated once again. The maximum loading displacement is $30 + 9 \times 40 = 390$ mm, which is corresponding to the drift ratio of 4%. For details of the modeling method and how all the parameters are set, literature (Cui 2015) is recommended.

4 CALCULATION RESULTS

4.1 Specimen BASE

Figure 2 shows the monotonic loading $P-\Delta/L$ curve of Specimen BASE. Where P is the applied

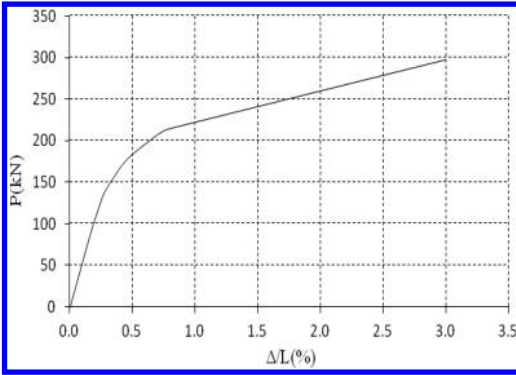


Figure 2. Monotonic loading P - Δ/L curve of Specimen BASE.

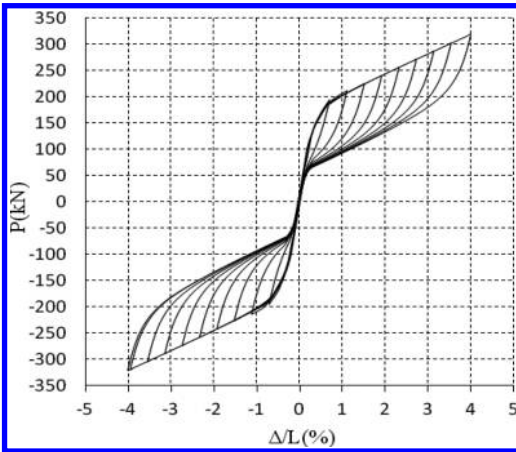


Figure 3. P - Δ/L curve of specimens BASE at the first floor column bottom.

horizontal load, which equals to the total column base shear; Δ/L is the horizontal drift divided by the column height, or may called drift angle or drift ratio. Figures 3–5 show the cyclic loading P - Δ/L curve of Specimen BASE at the column bottom of different floors. Here P is corresponding to the different floors; and Δ/L is all corresponding to overall drift ratio. Comparing the monotonic loading curve and the cyclic loading curves, it will be found that they have the same tendency and the cyclic loading value is somewhat a little lower than the monotonic ones at the same drift ratio. Because of the difficulty in simulation, the monotonic loading is only goes up to 3% of drift ratio but the cyclic one is up to 4%. From Figures 3–5, it is noted that the shear force at every floor is decreasing dramatically along the height of the structures. In this design of modeling specimens, different floors have the same

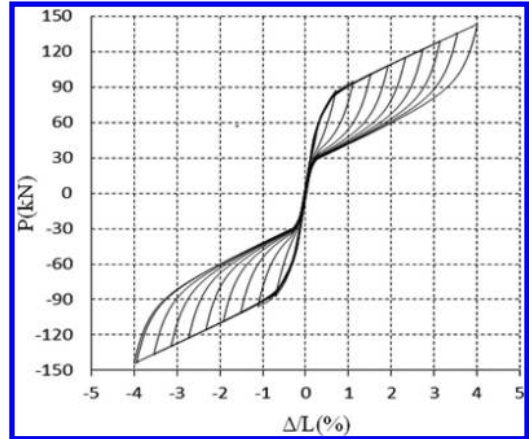


Figure 4. P - Δ/L curve of specimens BASE at the second floor column bottom.

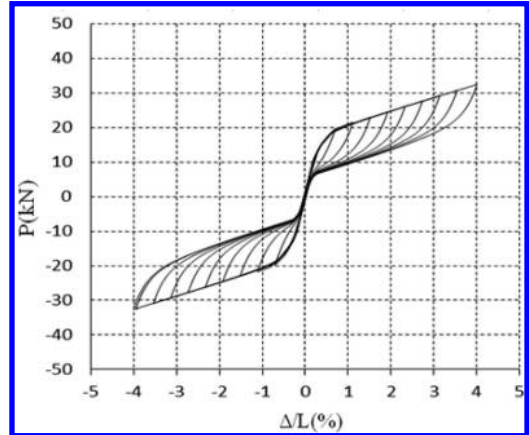


Figure 5. P - Δ/L curve of specimens BASE at the third floor column bottom.

structural members and tension strands. If it were not showed with the same drift ratio of the whole structures but with the drift ratio at different floors, the curve of the second and third floor may not show any hysteresis enveloped area. For decompression may not happen at these floors.

4.2 JL series of specimen JL series of specimens

The bolt vertical spacing of specimens Base, JL1 and JL2 are 100 mm, 250 mm and 400 mm accordingly. Comparing the cyclic loading curves of JL1 and JL2 specimen in Figures 6–7 to Base specimen in Figure 3, It is found that the structure's maximum base shear, initial structure stiffness, stiffness after decompression, ductility, load bearing capacity

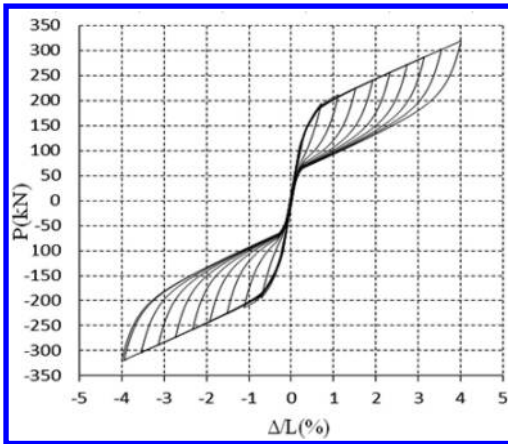


Figure 6. P - Δ/L curve of specimens JL1.

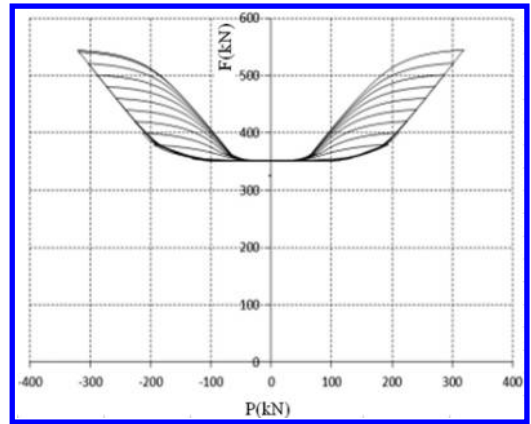


Figure 8. Force in tension strands vs. applied load of specimen BASE at first floor.

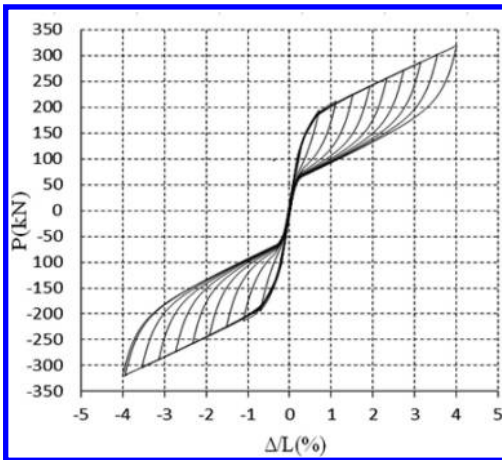


Figure 7. P - Δ/L curve of specimens JL2.

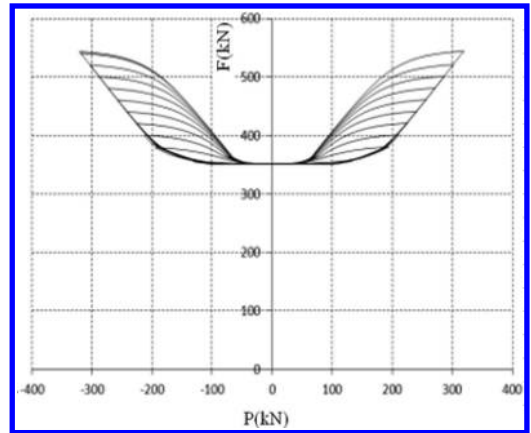


Figure 9. Force in tension strands vs. applied load of specimen JL1 at first floor.

and the energy-dissipation capacity are not differs notably. Figures 8–10 showed the strands internal force of specimens Base, JL1 and JL2 at first floor. It can also noted, they are nearly the same. From these results, it seems that the distance between the strands do not influence the seismic behavior of the structures. The reason for why this grotesque phenomenon happens is that the vertical layout of the strands is symmetric in height. This will result in the sum of all the arm of force is the same with different layout. In some research literatures, strands of only one row at the middle height of beam web are placed. But in practice, this kind of strands layout is not recommended because of two reasons: 1) when used single layer of strands, it needs to exert a greater pretension on steel strand. This will cause great stress concentration and early plastic deformation at the anchor region of column flange. 2) If it were

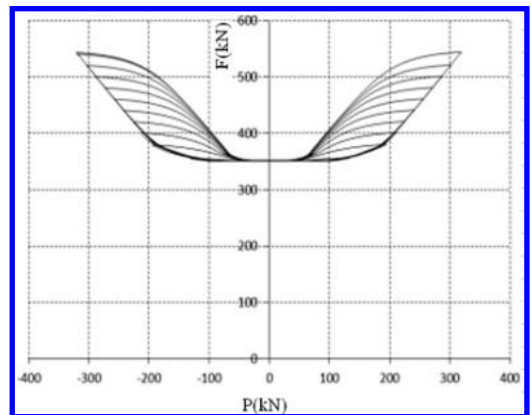


Figure 10. Force in tension strands vs. applied load of specimen JL2 at first floor.

single layer of strand, there would be no safety storage, or multi-layer of seismic defense lines. So strands should be designed as multi-layers and symmetrically equidistant placed along the vertical axis of the beam web.

4.3 JM series of specimens

For a directly compression, this time the $P-\Delta/L$ curves of specimens JM1 and BASE are put together in Figure 11. Before the beam-to-column faying surface decompressed, $P-\Delta/L$ curve of specimen JM1 is nearly the same as specimen BASE. This shows that the initial stiffness of specimen JM1 is the same as that of specimen BASE. The beam-to-column decompression and angle steel enter plastic deformation of JM1 specimen happens when the lateral force is 130 kN, where that of BASE specimen is 175 kN. JM1 specimen's lateral force was decreased by 26% from BASE specimen. But after decompression, force-displacement curves of specimen BASE and JM1 are nearly parallel. This shows that when reduces the strand pretension, the lateral force bearing capacity of self-centering steel frame will be reduced, but the initial stiffness and stiffness after decompression cannot be affected.

The difference between specimens JM1 and JM2 is only that the initial pre-tension forces in strands are respectively 0.6 and 1.3 times that of specimen BASE. The hysteretic curves of specimen JM1 and JM2 are shown together in Figure 12 for the convenience of closely detect their difference. Due to the increasing of tension in steel strands, load bearing capacities of JM2 before and after decompression are increased. But the curve slopes are nearly the same. This shows that the structural stiffness before and after decompression happens is nearly the same.

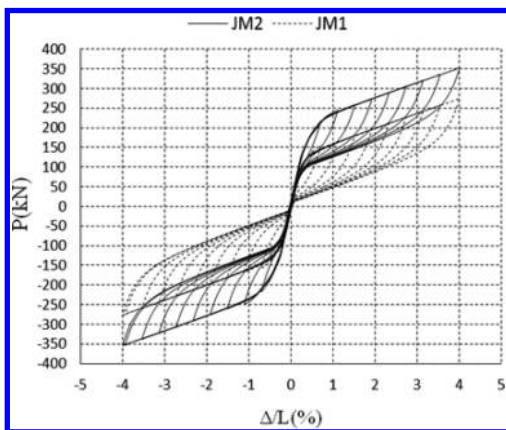


Figure 11. $P-\Delta/L$ curve of specimen BASE and JM1.

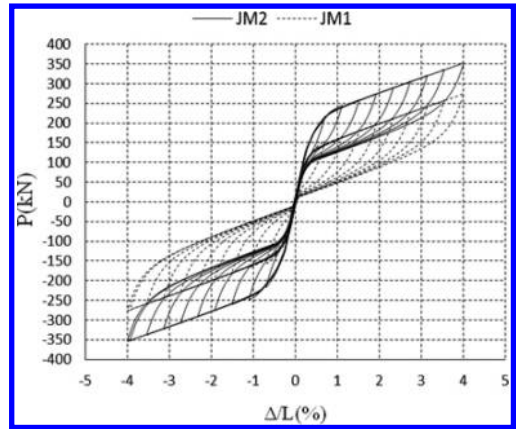


Figure 12. JM1 and JM2 specimen's $P-\Delta/L$ curve.

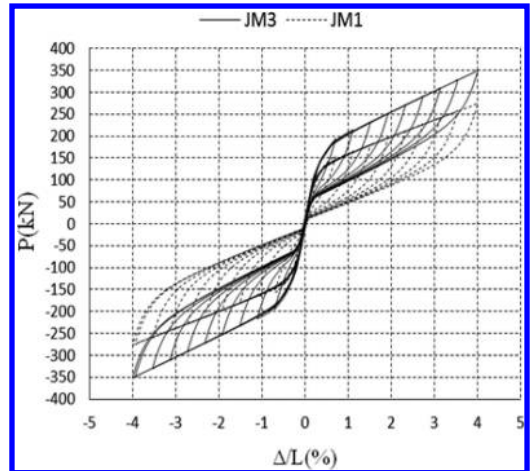


Figure 13. JM1 and JM3 specimen's $P-\Delta/L$ curve.

The sectional areas of specimen JM1 and JM3 are 370 mm² and 660 mm² respectively. And the initial stress in steel strands of JM1 and JM3 are all 280 N/mm². It can be noted from Figure 13 that with the increasing in the sectional area of strands, the load carrying capacity and the stiffness is increasing. But it is not clear to get the conclusion of only increasing the sectional area of strands, for by maintaining the initial stress of strands, the initial inner force of strands with bigger sectional area are increased.

So to get a clearer conclusion, specimen JM3 in Figure 13 can be compared with specimen BASE in Figure 11. The initial tension force of steel strands of specimen JM3 and BASE is nearly equal, but the sectional area and initial stress in steel strands are differs. At 4% drift ratio, Specimen JM3 can carry a load of 350 kN, but Specimen BASE 320 kN. Specimen JM3 has a higher load carrying capacity

and stiffness because it has a bigger sectional area and lower initial tension stress in strands.

Figures 14–16 shows the inner tension force which are changing with the external lateral force P . The initial inner force can be calculated from Table 1 by multiply the figures of A_T , n and σ_0 for each specimen.

4.4 JG series of specimens

Series of specimens JG is designed to study the influence of the thickness and length of angle legs on self-centering structure performance. The thickness of angle Specimen JG1 and JG2 is 0.5 and 1.5 times that of Specimen BASE. The leg length of two specimens is same as that of specimen BASE. Specimen JG3 use angle steel L200×180×16, a decrease of 100 mm and 70 mm of both leg lengths from L300×250×16 of Specimen BASE. The designed fixing position of

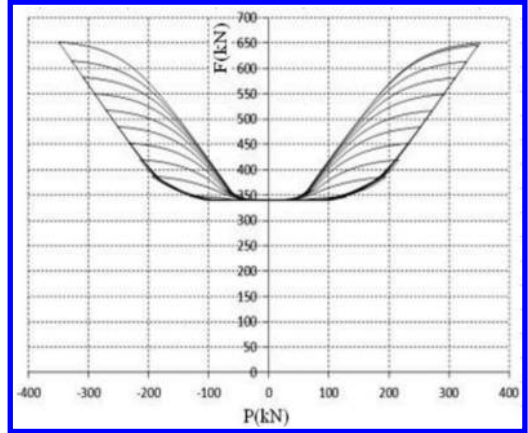


Figure 16. JM3 specimen's tension-shear curve of the first tier's steel strand.

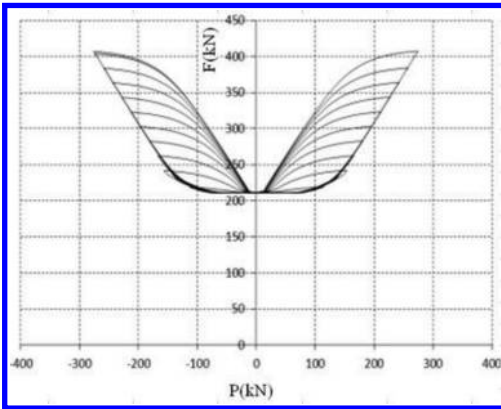


Figure 14. JM1 specimen's tension-shear curve of the first tier's steel strand.

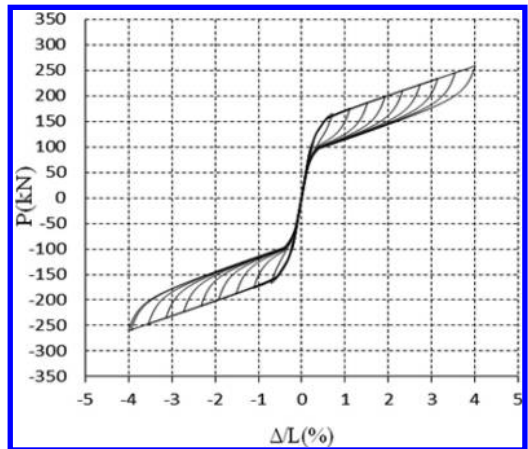


Figure 17. P - Δ/L curve of specimens JG1.

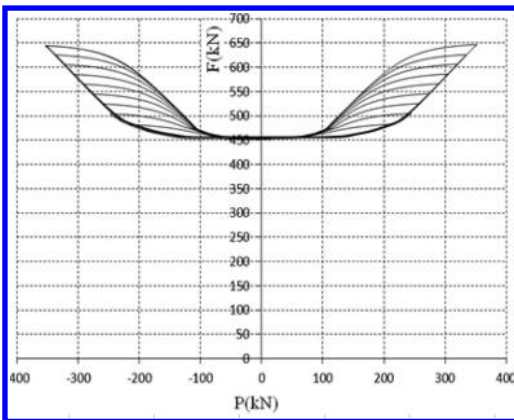


Figure 15. JM2 specimen's tension-shear curve of the first tier's steel strand.

the vertical and horizontal leg of angle is adjusted with the proportion of specimen BASE. Vertical fixing point from the top is 33.33 mm. And the lateral fixing point from the far end is 63 mm.

The P - Δ/L curve of specimens JG1, JG2 and JG3 is shown in Figures 17–19. They all have the same tendency but their difference is also clearly seen. The maximum lateral force of JG1, JG2 and JG3 is 255 kN, 370 kN and 358 kN. And JG1 is 20% lower than specimen BASE. Specimen JG3 has a short arm of force of angle steel for its energy-dissipation. So it has a higher loading carrying capacity but cannot go through the 4% drift ratio. The angle is fractured at about 3% loading. Specimen JG1 has the thinnest angle legs, so its load carrying capacity and energy-dissipation capacity is the worst one. JG2 has the best load

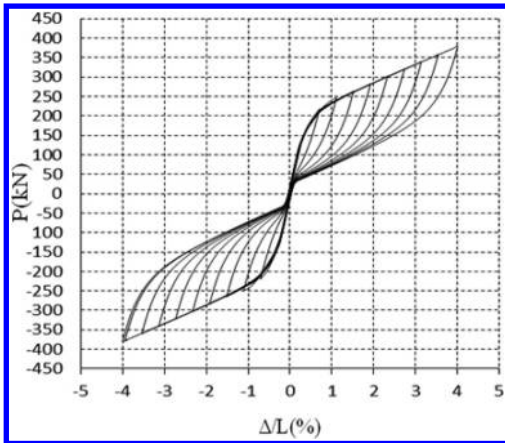


Figure 18. P - Δ/L curve of specimens JG2.

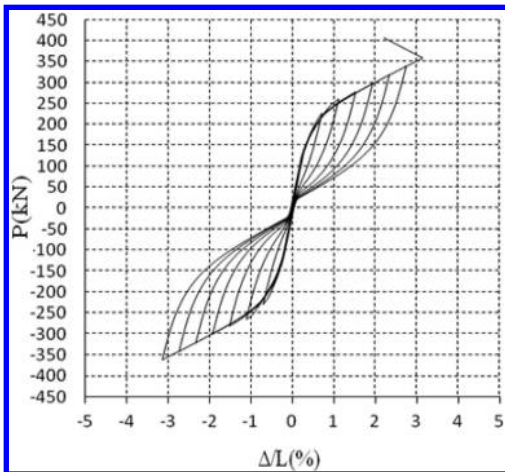


Figure 19. P - Δ/L curve of JG3 specimens.

carrying capacity and energy-dissipation capacity because it has the strongest angle steel.

5 CONCLUSIONS AND SUGGESTIONS

Through the analysis of these three-story self-centering structures, conclusions can be drawn as follows:

1. The symmetric placed space of steel strands has no effects on bearing capacity of multi-story self-centering structural system. Steel strands should be designed along the beam web symmetrically, and avoiding the use of only one row of steel strands.
2. The pretension stress of steel strands may influence the load carrying capacity of self-centering

steel structures, but no effect on the initial stiffness and stiffness after decompression.

3. The sectional area of steel strands may influence the load carrying capacity, the initial stiffness and stiffness after decompression of the whole structural system.
4. The thickness and size of the legs of angle steel both may seriously influence the load carrying capacity, energy-dissipation capacity and the system working function of the structural system.

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Study on progressive collapse of steel frame based on Alternate Load Path Method

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ABSTRACT: 2-D and 3-D Sap2000 models have been created to simulate the realistic condition of an existing four-story steel frame which was tested based on APM (Alternate Load Path Method) before the demolition process. Strain and displacement data were measured on site. The calculated data is compared to the test data in order to evaluate the accuracy of the models. That shows the 3-D model is more accurate which is better used for further study on the progressive collapse potential of steel frames. The Linear Static Method (LS) and Nonlinear Dynamic Method (ND) are conducted to analyze the progressive collapse of steel frames separately. For LS, DCR is used to find the weakest part of the structure and the time history method has been used for the ND analysis procedure. Some valuable conclusions have been given, which are of benefit to the progressive collapse study of steel frames.

1 INTRODUCTION

A building may be subjected to some natural disasters (e.g. earthquake, hurricane) and man-made hazards (e.g. fire, explosion), which would cause the structural damage, even to collapse. This should harm to the sustainability of building seriously and may lead to huge economic losses and casualties. Good performance of anti-disaster is one of the main characteristics of sustainable building, so a sustainable building should have a high degree of redundancy, which is considered to resist various kinds of disasters. Steel frame is the one of the most popular structures considered sustainable. This paper is focused on the progressive collapse of steel frame based on APM (Alternate Load Path Method), and the results could be helpful to disaster mitigation and sustainable development for other similar buildings. Progressive collapse, a catastrophic damage, occurs when an initial local damage or failure spreads. It is defined by General Services Administration as “a situation where local failure of a primary structural component leads to the collapse of adjoining members, and hence, the total damage is disproportionate to the original cause” (GSA 2003). In this paper, numerical simulation is performed to investigate the progressive collapse potential of an existing four-story steel frame. One column in the first story was physically removed to simulate the sudden loss of vertical member. Strain gauges and displacement sensors were installed to measure the strain and displacement (vertical and horizontal) of the neighboring columns and beams before and after the column removal. In this paper, 2-D and

3-D models of the steel frame are proposed to study the progressive collapse potential with one column lost and learn more about the structure response to a vertical member failure. The accuracy of the model simulation is evaluated by comparing with the test data.

Methods for progressive collapse analysis could be divided into two categories: static and dynamic. Obviously, the dynamic method should be more accurate, but it is time consuming especially for nonlinear dynamic situation. Based on Alternate Load Path Method (APM), the linear and nonlinear methods are applied to analyze the progressive collapse of steel frame. For linear analysis, DCR (Demand Capacity Ratios) could be easily used to find the weakest part of the structure, while the deformation is used as the criteria to indicate the damage location and collapse region in the nonlinear analysis clearly. In this paper, the most simplest Linear Static Method (LS) and the most complicated Nonlinear Dynamic Method (ND) are conducted in the analysis process separately.

2 THE TESTED BUILDING

The tested building (see Fig. 1) is a four-story steel frame building without basement, located in Ohio State University (Columbus, OH., U.S.), which was constructed in 1925 and demolished in 2011. Series of tests have been conducted before the demolition of the building (Akah 2013). One first floor column next to the corner (see Fig. 2) was physically removed in a short time. Seven strain



Figure 1. The tested building.

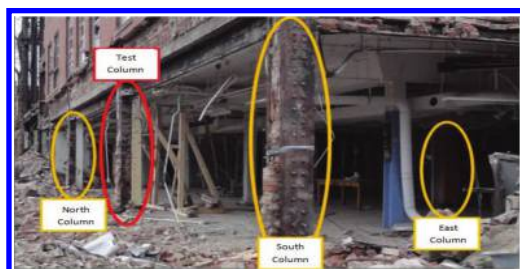


Figure 2. The location of removed column.

gauges were installed on the columns to measure the longitudinal axial force, nine stain gauges were used to measure the compressive and tensile stress due to the bending moment of the beams connecting to the removed column, three displacement sensors were used to measure the vertical and horizontal displacements.

3 THE EVALUATION OF SAP2000 MODELS

In order to compare with the test data, 4 models have been created. 2-D models with and without the remove of the column, and the same as 3-D models. Models with column represents the natural static equilibrium before the column removed, models without column represents the static equilibrium after the column removal. The strain changes can be calculated by subtracting the strain data in two related models.

The comparison of test and calculated data are shown in Figure 3.

It can be concluded from Figure 3 that the absolute value of strain in 3-D model is generally smaller than that of 2-D model mainly because of the 3-D space function in the steel frame. In the location of gauge 7, the calculated strain and the test data have the minimum error of 24.2%. The calculated strain is generally larger than the actual data. The error mainly come from the simplified of the

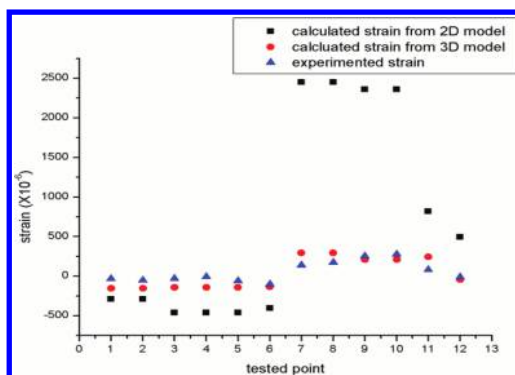


Figure 3. Comparison of strain data.

analysis procedure, such as the default rigid joint assumption in Sap2000, the inaccuracy of the load calculated.

What's more, the theoretical displacements in 2-D model exceeded 110 mm, while the test displacements didn't exceed 12 mm, showing a drastic difference. The deformation quantify the strain are positive proportional relationship, it is obvious that the calculated displacement will be much larger than the test displacement. Similar to the factors caused the increase of the strain, those factors also can be used to explain the displacement increase here. The theoretical displacements in 3-D model reached 11 mm, which is more closed to the measured value. The calculated displacement data at the location of displacement gauge 1 only had an error of 8.9%.

From the comparison above, the 3-D model of the steel frame is more accurate and acceptable than the 2-D model. So, the 3-D model could be used for further study.

4 PROGRESSIVE COLLAPSE POTENTIAL EVALUATION OF THE STEEL FRAME

Different analysis methods are used to study the potential of the steel frame. Figure 4 shows the 3-D model created in Sap2000. Based on the Alternate Path Method, the column (Fig. 2) next to the corner in the longitudinal direction is removed. Linear Static (LS) and Nonlinear Dynamic (ND) methods have been performed to simulate the response of the damaged steel frame. Different methods have their own applicability. Obviously, the dynamic method should be more accurate, but it is time consuming especially for nonlinear dynamic situation. The detailed analysis procedure will be depicted and the progressive collapse mechanism will be further studied.

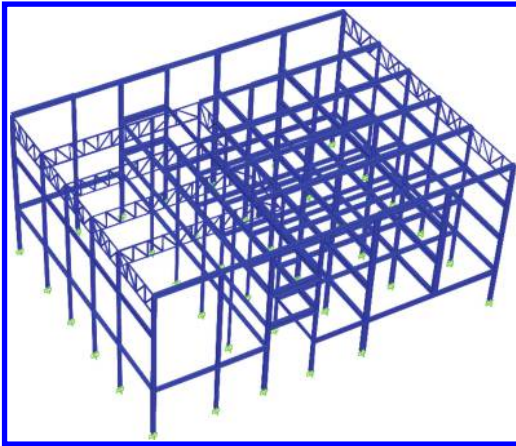


Figure 4. The 3-D model created in Sap2000.

4.1 Linear Static (LS) analysis

In the linear analysis procedure, the nonlinear factors of material and deformation are neglected. The steel frame is loaded only once, and only the final calculated result is retained, the structural response caused by the functional loss of critical element is also neglected. The Demand Capacity Ratio (DCR) of each component is proposed to compare to the defined acceptance criteria in GSA guidelines. DCR is defined as the Q_{UD}/Q_{CE} , Q_{UD} is the internal force and moment result from calculation, and Q_{CE} is the expected strength of the component or element. According to the GSA, width/thickness ratio of the member determine the acceptable numerical range of the members, the acceptable DCR value varies from 1.25 to 3.0. For steel frame structure, if a given member exceed 2.0 for column or 3.0 for beam, the component is considered to be failed.

The step by step procedure of the linear static analysis is shown as follows:

1. Create a model of initial steel frame shown in Figure 3.
2. According to the GSA guideline, remove the column next to the corner in the first floor.
3. Loading: distributed load and concentrated load are applied to the beam and column, while the load in the influenced bays just above the removed column should be amplified, and the Load = $2(DL + 0.25 LL)$, where DL represents dead load and LL represents live load. LL was assumed to be zero in the calculation.
4. Check the DCR number of every member and compare the value to the criterion in GSA. Evaluate the progressive collapse potential of the steel frame. The DCR value of every member is shown in Figure 5.

Two beams directly connected to the removed column have the DCR value of 1.200 and 0.906. In the fourth floor, the maximum DCR value occurred in the two beams in the two bays above the removed column, are reached 4.705 and 4.439. So the structure is considered to have the potential of progressive collapse.

After removal of the column, the axial and bending moment supplied by the column suddenly disappeared. The two bays above the removed column become one longer bay. At this time, the internal force would be redistributed, and then the structure reaches a new equilibrium. It is obvious that the moment and shear in the four beams could be sharply increased. The DCR value in the beams would become relatively larger. The beams in the two bays and the columns above removed column are most sensitive to the loss of the column in the first floor.

What's more, most DCR value in columns are not exceed 1.0, the maximum DCR value in the column only reaches 0.475, not exceed half of DCR limit of the column. While the maximum DCR value in the beam is twice of the DCR limit of the beams. So it is easily concluded that beams are more inclined to be influenced than columns for the removal column.

4.2 Nonlinear Dynamic (ND) analysis

Nonlinear Dynamic (ND) analysis is a more accurate method, which is usually called time history analysis method. The material non-linearity (yield) and the geometric non-linearity (P-Δ effect) can be simulated, which are just neglected in the Linear Static (LS) analysis. As an important factor, dynamic effect of progressive collapse is also taken into account. Three methods (ASCE 2005) for the Nonlinear Dynamic (ND) analysis have been put forward: initial condition method, instantaneous stiffness degradation method, instantaneous loading method. The initial condition method is used in the paper.

The step by step procedure of the nonlinear dynamic analysis is shown as follows:

1. Create a 3-D model of initial steel frame.
2. Obtain the axial force of the column before removal, $P = 466$ KN. Because the moment value in the column is relatively small, the moment in the column is neglected.
3. Remove the column next to the corner, and use a reaction force $P = 466$ KN to replace the removed column. The time history function curve of the force is defined, shown in Figure 6. The M-3 hingedefined by FEMA 273 is defined in the four beams above in the two bays above the removed column.
4. Loading: distributed load and concentrated load are applied to the beam and column, and the Load = $(DL + 0.25 LL)$, where DL represents

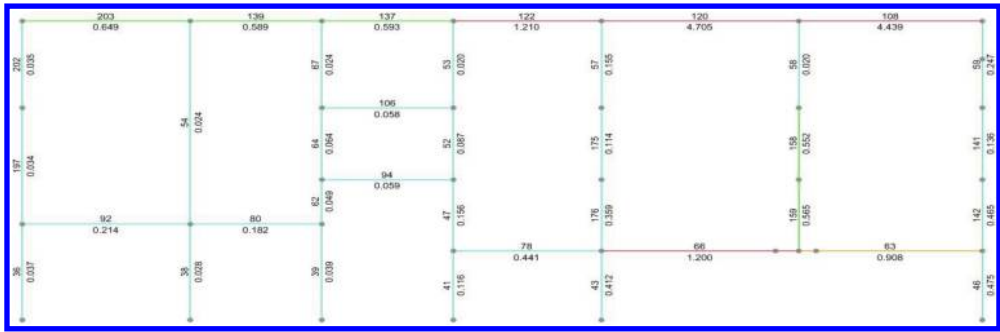


Figure 5. The DCR value from the linear static analysis (with element labels shown).

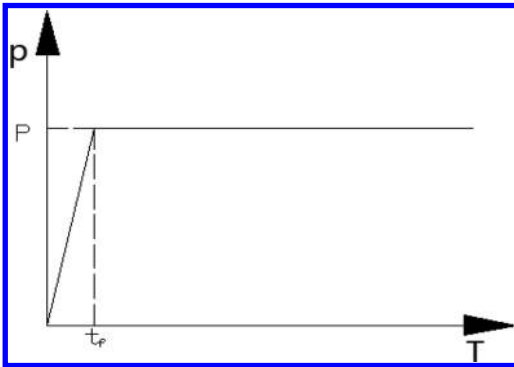


Figure 6. The time function curve.

dead load and LL represents live load. Consider the real situation, LL was assumed to be zero (Song 2014) in the calculation. The concentrated reaction force P in step 3 is applied.

5. The initial load case is set as nonlinear static, and the calculated results will be used as the initial condition as the next step.
6. Start from step 5, and a nonlinear time history load case is defined. Some factors need to be calculated, from the model analysis, such that the mass damping coefficient is set as 0.9 and stiffness damping coefficient is 0.0002.

4.2.1 Internal force redistribution in the column

The real transfer process of loading can be summarized from the internal force's change of the components. The axial force in column 41, 70, 71, 73, 84, 85, 87 are little effected and vibrated in a little range, shown in Figure 8. The time history curves of axial force in column 43, 46, 72, which are the three columns adjacent to the removed column, are shown in Figure 9. It is easily to observe that, the axial force in column 43, 46 and 72 sharply increased at the moment the column is removed, then the curve vibrated and become stable at the

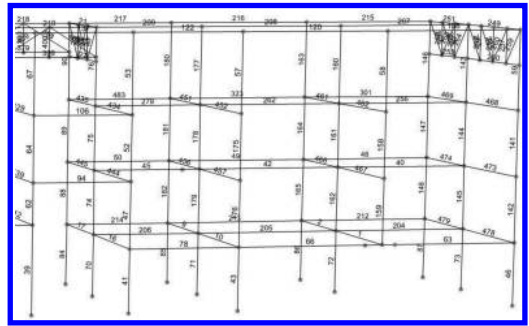


Figure 7. Labels of some components around the demolished column.

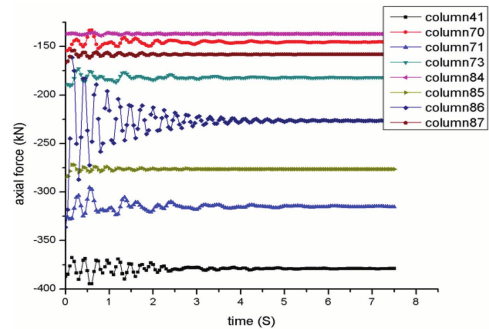


Figure 8. Axial force time history curve in columns far away from the removed column.

end of 7.0 seconds. The three columns adjacent to the removed columns are greatly influenced by the column loss. The axial force carried by the removed column is redistributed to the adjacent columns. In the column 43, the initial axial force was 171 kN, then the axial force vibrated and stable at the value 212 kN, increased 24%. And the axial force increase percentage in column 46 and 72 is

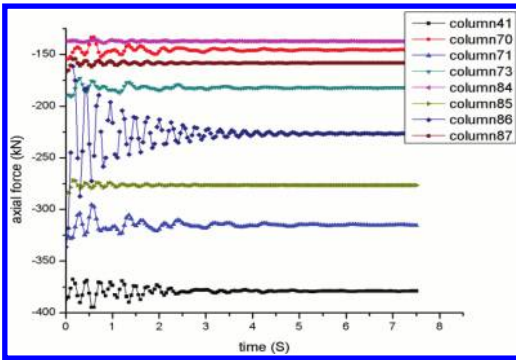


Figure 9. Axial force time history curve in surrounding columns.

41% and 65% respectively. The column 72 is mostly affected mainly because it is the most closed one, only 6.63 m away from the removed column in the horizontal direction. The so called “shortest path principle” is evident here, which could be simply explained as “force inclined to transfer to the most closed components”.

While, there is also an interesting phenomenon that the axial force in column 86 (shown in Fig. 8) decreased and vibrated to the opposite direction, just because the so called “leverage effect”. The top of the column 72 around the removed column can be imagined as the fulcrum of a crowbar, beam 1 and beam 2 can be imagined as the crowbar, and column 86 as pried. The “leverage effect” in column 86 is more significant than other factors. As a consequence, the axial force in the further column 86 vibrated to the opposite direction, the axial force in column 86 decreased 31% when it is stable.

4.2.2 Internal force redistribution in the beams

The time history curves of beams connected to the removed column are shown in Figure 10 and Figure 11. The initial axial force in beam 63 and 66 vibrated drastically at the moment the column is removed, and changes from pressure to tensile force. The axial force in beam 63 before the column removed is -1.53 kN, then become 3.50 kN when it is stable. The axial force in column 66 before the column removed is -1.53 kN, then become 3.50 kN and the axial force in beam 66 changed from 0.323 kN to 2.478 kN. It is considered that the catenary effect is considered to have been formed.

The moment in the left end of the beam 63 and the moment in the right end of beam 66 are shown in the moment time history curves in Figure 11. The moment in beam 63 is -118 kN*m before the column removed, then the moment vibrates to the maximum moment of 281 kN*m at 0.15 s, the moment finally becomes stable at the value

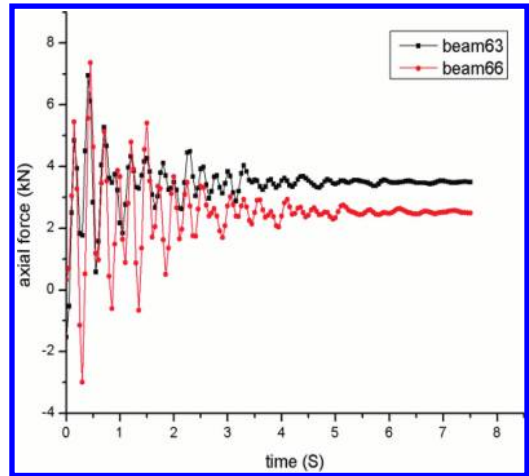


Figure 10. Axial force time history of beams connected to the removed column.

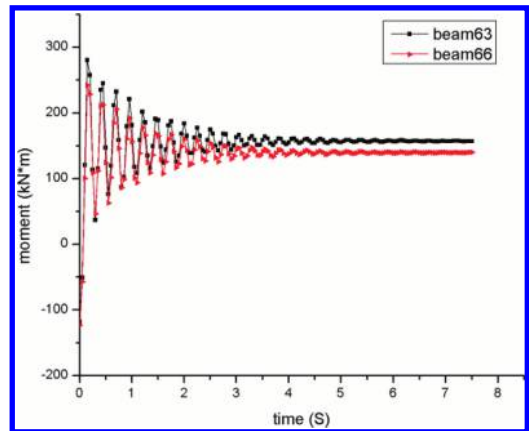


Figure 11. Moment time history of beams connected to the removed column.

of 157 kN*m. The moment time history of beam 66 is similar to that of beam 63. It is considered that the beams connected to the removed column are the key components in the internal force redistributed process. The joint displacement above the removed column is reached 37.1 mm. According to the guidelines, the rotation angle $\theta_{\max} = 2.8^\circ > 2^\circ$, indicated that the steel frame couldn't resist the progressive collapse.

4.3 Comparison of LS and DL

The displacement of joint above the removed column is compared in Figure 12. The displacement from LS method is 23.5 mm, and the maximum

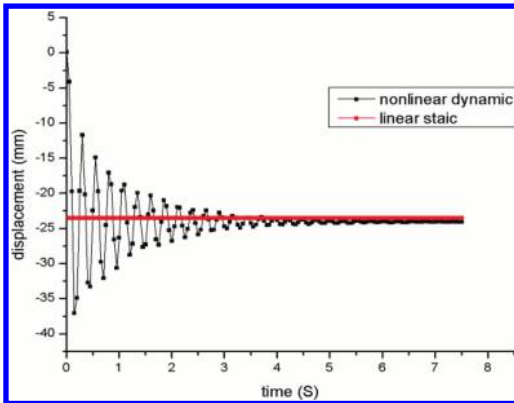


Figure 12. Displacement curves from LS and ND methods.

displacement from ND method is 37.1 mm, which is 58% larger. In LS analysis, in order to simulate the strong impact of the collapse members on the residue part of the structure, an amplification factor 2 is applied. In the ND analysis, the dynamic effect is better simulated, as shown in Figure 10. In the LS calculated results, the moment at the left end of beam 63 is 151 kN*m, while the moment from ND reaches a maximum moment of 281 kN*m. The moment calculated from LS method is 133 kN*m at the right end of beam 66, the maximum moment from ND analysis is 242 kN*m. The LS analysis calculated is relatively conservative.

5 CONCLUSIONS

The Alternate Load Path Method (APM) is used without considering the specific form of unconventional load caused by initial damage. Only the structure's characteristic response after local failure of main component (column) is analyzed. The method is simple and easy to conduct in computer, so it is widely used in the progressive collapse study.

LS and ND are conducted to analyze the progressive collapse of steel frame separately. Different evaluation criterions are used in the progressive collapse potential evaluation. The DCR criterion is used in the LS analysis, the maximum DCR value from LS analysis is 4.7, which is 160% of

the acceptance DCR value 3. The structure is considered to be progressive collapse from the LS analysis. The deformation criterion is used in the ND analysis. The maximum rotation angle in the beam is 2.8° , which is 140% of the acceptance rotation angle for steel beam 2° . From the LS, the structure is considered a greater possibility of failure. In other words, the LS method is more conservative than ND method in progressive collapse study.

In the real condition, some columns (columns in the entrance of the parking lot, columns in the street corner) are vulnerable. And the components adjacent to the vulnerable columns are the key components in the force redistribution, and the beams in the bay above the failure column are also most susceptible, which should be specially designed. The bearing capacity and ductility of the components around the vulnerable columns must be specially enhanced in order to resist the progressive collapse.

ACKNOWLEDGEMENTS

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Study on mechanical behaviors of assembly truss-column joint in steel staggered truss structure

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ABSTRACT: Two half-scaled specimens of staggered truss structures with two different types of joints were designed to study the mechanical behaviors of truss-concrete filled square steel tubular column joints in assembly steel staggered truss structures under vertical load and static experiments were conducted. The finite element models of test specimens were established, and nonlinear inelastic analyses were considered to study the influence of compression ratio and the strength grade of concrete in the steel tube on the load bearing capacity of structures. Results indicated that the joints designed by full strength had an obvious influence on the structures with hinged joints while the joints designed by actual load had a great effect on the structures with semi-rigid joints. Decreasing the compression ratio or increasing the strength grade of concrete in the steel tubes can slightly improve the load bearing capacity of the structure.

1 INTRODUCTION

Steel staggered truss system was developed at the Massachusetts Institute of Technology in 1960s, which utilized story-deep trusses in alternative stories spanning the full width of building. In this system, the trusses not only carry vertical load, but also resist the lateral forces, and the floor system carries vertical loads, meanwhile, transfers the horizontal force as a diaphragm. This double function resulted in a lighter structure and provides more column-free space than a conventional moment resistant frame structure. Steel staggered truss system had been widely used in structure engineering (Bakke et al. 1969, Hassler 1986, Huzzard & Larson 1988, McNamara 1999, Levy 2000).

Concrete-filled square steel tubular column was adopted in assembly steel staggered truss system, and hinged joints, semi-rigid joints and rigid joints can be used in this system. The concrete-filled square steel tubular column will improve the local buckling and stability of the structure, also supply higher lateral stiffness to this system which improve the seismic performance of this system.

Many theoretical and experimental researches were carried out to research the mechanical behavior and seismic performance of steel staggered truss system (Gupta & Goel 1972, Goel & Itani 1994, Pan & Zhou 2002, Zhou & Mo 2006, Ran et al. 2007, Ran 2008), which indicated that this structural system had good mechanical behavior and

seismic performance. But only a few of them were focused on mechanical behavior of truss-column joints. In order to research the mechanical behaviors of truss-concrete filled square steel tubular column joints in assembly steel staggered truss structures under vertical load, two half-scaled specimens of joint with different types in staggered truss structures were designed. Experiments were conducted under vertical static loading to research the failure mode, load bearing capacity, stiffness and stress distribution of the specimens. Then, the finite element models of test specimens were established by using ABAQUS, and nonlinear inelastic analyses were considered to study the influence of compression ratio and the strength grade of concrete to the load bearing capacity of assembly steel staggered truss structures.

2 DESIGN OF TEST MODEL AND EXPERIMENTAL PROGRAM

2.1 Design of test specimen

The prototype structure of the test specimen was designed based on the AISC code for staggered truss framing systems, Chinese code for seismic design of buildings (GB50011-2010), Chinese code for technical specification for steel structure of tall buildings (JGJ 99-98) and Chinese code for design of steel structure (GB50017-2003). There were five stories in prototype structure; four bays

in x-direction and one bay in y-direction. The story height, the span in the x-direction and the span in the y-direction are 3, 6, and 12 m, respectively. The frames located in y-direction were designed to resist seismic loads and incorporated steel staggered truss. Q235 steel with nominal yield strength of 235MPa was used in all structural members of prototype structure and C30 concrete was used in the concrete-filled square steel tubular column. The steel staggered truss in the second story was selected as the prototype of test specimens (refer to Fig. 1).

According to the experimental conditions, two half-scaled specimens were designed based on the prototype structure. The tube connections between steel column and truss in specimens were designed based on load and full strength, respectively, and which named STJ-1 and STJ-2, respectively. Table 1 shows the structural member sections and properties of test specimens. Main geometric sizes of specimens and joint details are shown in Figure 2 and Figure 3, respectively. Table 2 shows the mechanical properties of steel used in test specimens.

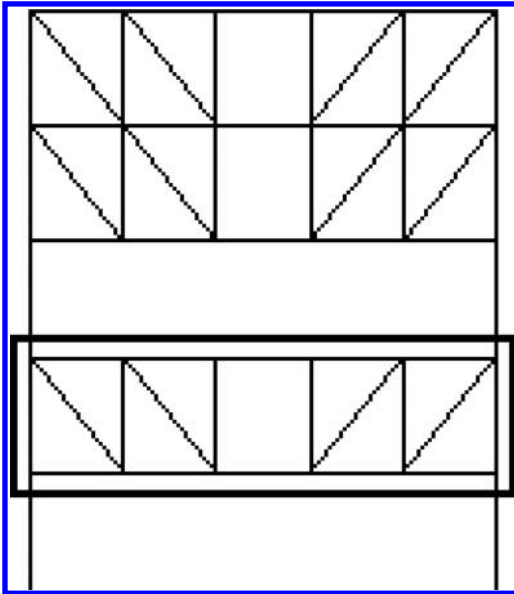


Figure 1. The selection of test specimen.

2.2 Test setup and experimental program

The vertical loads were applied by the oil jacks pushing against the top of joints and a constant axial load of 420kN with the axial compression ratio of 0.4 was applied at the top of the column. Lateral support was used to supply the out of plan stiffness to specimens. Rigid connections were adopted to connect column base to ground. The setup of the test is shown in Figure 4. The vertical

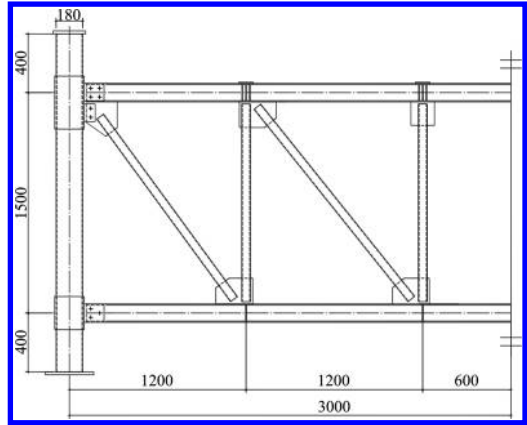


Figure 2. Main geometric sizes of specimens.

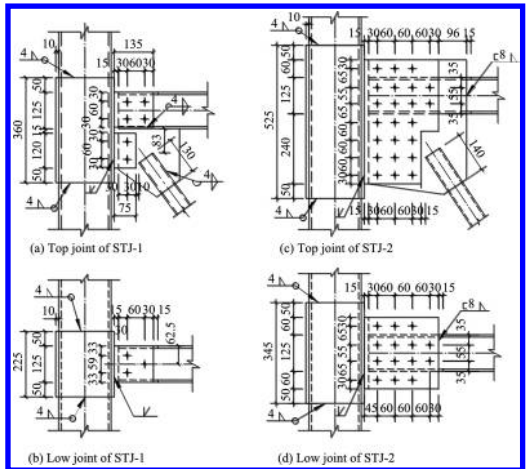


Figure 3. Joint details.

Table 1. Properties of test specimens.

Designation	Column section	Chord section	Web member	Joint type	Design load
STJ-1	Box 180 × 4	HN125 × 60 × 6 × 8	Box 60 × 5	Tube connection	55.62 kN
STJ-2	Box 180 × 4	HN125 × 60 × 6 × 8	Box 60 × 5	Tube connection	Full strength

Table 2. Mechanical properties of steel.

Structural member	Thickness t/mm	Yield strength f_y/MPa	Ultimate strength f_u/MPa	Elastic modulus $E/\times 10^5\text{MPa}$	Elongation ratio δ
Chord	8	266.99	417.58	2.02	0.42
Chord	6	259.57	406.67	2.14	0.31
Beam flange	8	289.12	433.49	2.22	0.30
Beam web	6	333.65	456.77	2.22	0.32
Column	4	411.23	452.54	2.31	0.22

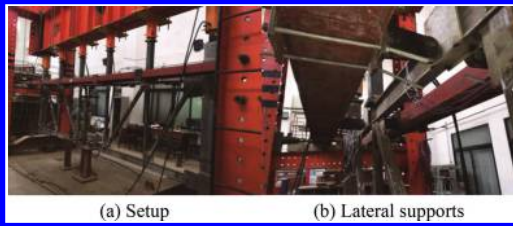


Figure 4. Setup of test.

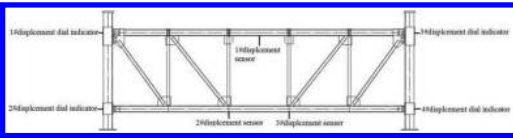


Figure 5. Layout of displacement sensors.

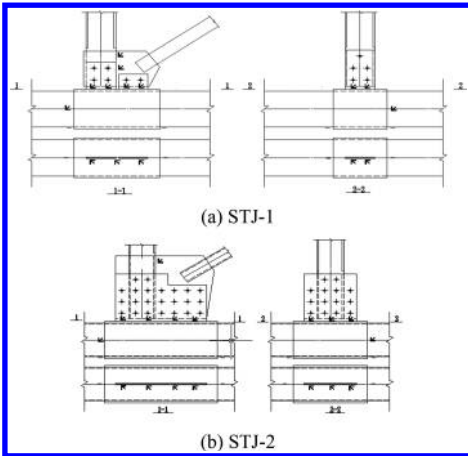


Figure 6. Layout of strain gauges in specimens.

loads at the top of joints were loaded by ranks. Using the increasement of 20 kN before the load reach 100 kN; then using the increasemet of 5 kN was used as the load increasement till the end of test.

2.3 Instrument arrangement

As shown in Figure 5 and Figure 6, displacement sensors and strain gauges were used to obtain the displacement response and strain response, respectively.

3 EXPERIMENTAL OBSERVATIONS

The structural members and joints of specimen STJ-1 were in good conditions at the beginning of loading. When the vertical load reached 115kN, the bottom chords occurred out-plan bending obviously. When the vertical load reached 140kN, the out-plan bending also appeared in the top chords, web members and joint plate in specimen STJ-1. The specimen could not continue to bearing the load and the test was stopped. Figure 7 shows the damage of specimen STJ-1 during the test.

There were no phenomena occurred in specimen STJ-2 at the beginning of loading and the structural members were at the elastic stage. When the vertical load reached 115kN, out-plan bending obviously appeared at the mid-span but all structural members and joints were in good conditions. The specimen could not continue to bearing the load and the test was stopped. Figure 8 shows the damage of specimen STJ-2 during the test.

4 TEST RESULT AND ANALYSIS

4.1 Load-displacement curves

The load-displacement curves of specimen STJ-1 is shown in Figure 9, P and Δ are vertical load at joints and deflection of mid-span. The increase of deflection at mid-span was linear with vertical load at joints in the beginning of the curves which indicated that the specimens were at elastic stages. It is obvious that the STJ-1 entered plastic stage when the vertical load at joints reached about 120 kN and the load bearing capacity increased after that. It indicated that STJ-1 had good load bearing capacity.

The load-displacement curves of specimen STJ-2 is shown in Figure 10, P and Δ are vertical



Figure 7. Phenomena of STJ-1 during test.



Figure 8. Phenomena of STJ-2 during test.

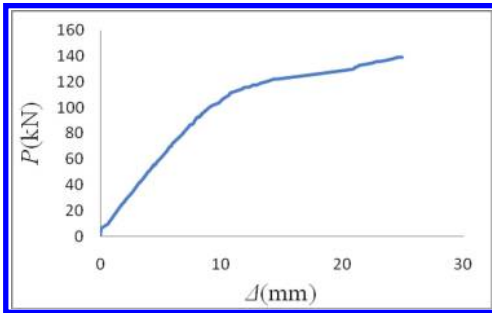


Figure 9. Load-displacement curves of STJ-1.

load at joints and deflection of mid-span. The curve shows that the increase of deflection at mid-span was linear with the increase of vertical load at joints which indicated that structural members still remained in elastic stage. And the load bearing capacity of STJ-2 did not reach the ultimate value due to the out-plane instability.

4.2 Stiffness degradation

Figure 11 shows the stiffness degradation curves of STJ-1 and STJ-2. The stiffness of specimens decreased obviously with the increase of vertical load at the joints and the stiffness decreased more obviously at the beginning. Moreover, according to the comparison of stiffness degradation curves of STJ-1 and STJ-2, STJ-2 had higher elastic stiffness than that of STJ-1, which indicated that full

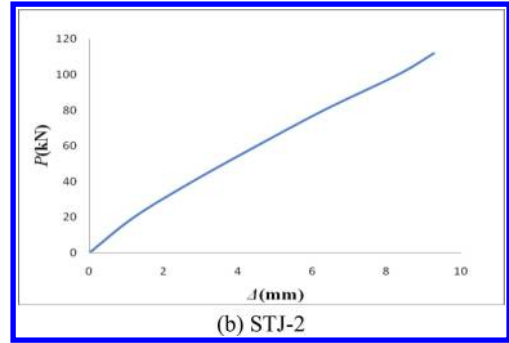


Figure 10. Load-displacement curves of specimens.

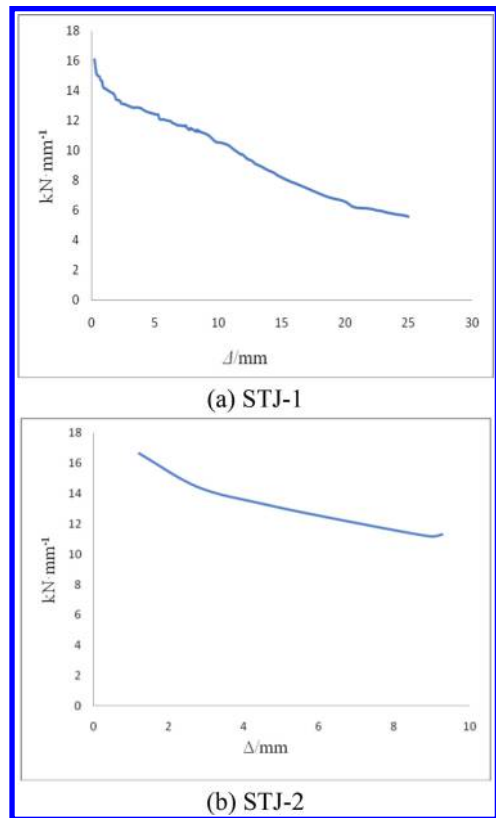


Figure 11. Stiffness degradation curves of specimens.

strength designed joint could supply higher stiffness to the structure than that of load designed joint.

4.3 Strain response

Figure 12 shows the strain response of STJ-1 during the test. When STJ-1 yield, only the plate at end-plate connection entered inelastic stage and

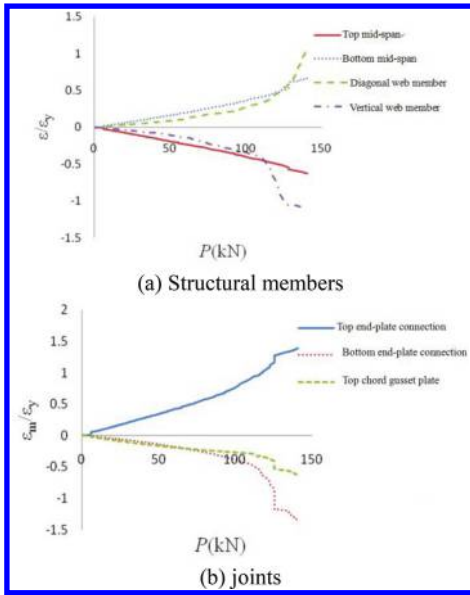


Figure 12. Stress response of STJ-1.

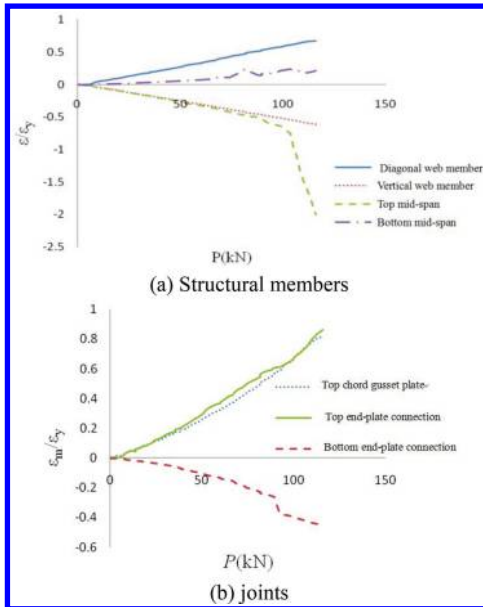


Figure 13. Stress response of STJ-2.

the strain value of other members were very small. The sudden change occurred in the response curve was because of the out-plane instability.

Figure 13 shows the strain response of STJ-2 during the test. The strain response of STJ-2 was linear with the vertical load and the strain of top mid-span increased obviously is because of the

out-plan instability. Furthermore, all strain values were lower than yield ones and the strain value of top end-plate connection was closer to the yield value than those of others in STJ-2.

Moreover, it shown that the yield strength of STJ-1 was lower than that of STJ-2 by comparing the strain response curves of STJ-1 and STJ-2, which indicated that full strength designed joint could supply higher yield strength to structure than load designed joint.

5 FINITE ELEMENT ANALYSIS ON MECHANICAL BEHAVIORS

5.1 Finite element models of test specimens

In order to better observe the mechanical behaviors of steel staggered truss system, ABAQUS was used to establish the finite element models of test specimens. Solid elements were used for test specimens that were meshed by using the software's "Structure" mesh type. Figure 14 shows the finite element model in ABAQUS. All DOF of column base was constrained to consider the rigid connection and out-plan translational DOF of specimen was constrained in order to consider the impact of lateral supports. Mechanical properties of steel used in the finite model are presented in Table 2. Furthermore, the kinematic hardening material model and the Mises yield condition were adopted for the steel in the numerical model. The elastic modulus and Poisson's ratio are assumed to be 206,000 MPa and 0.3, respectively.

5.2 Comparison of finite element results with test ones

Load-displacement curves for both the test specimen introduced in this paper and the relevant finite element model are shown in Figure 15. The analysis

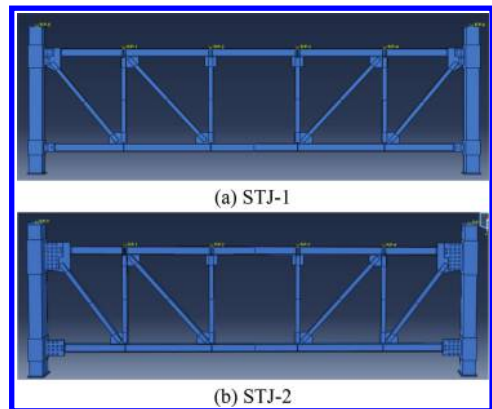


Figure 14. Finite element models of specimens.

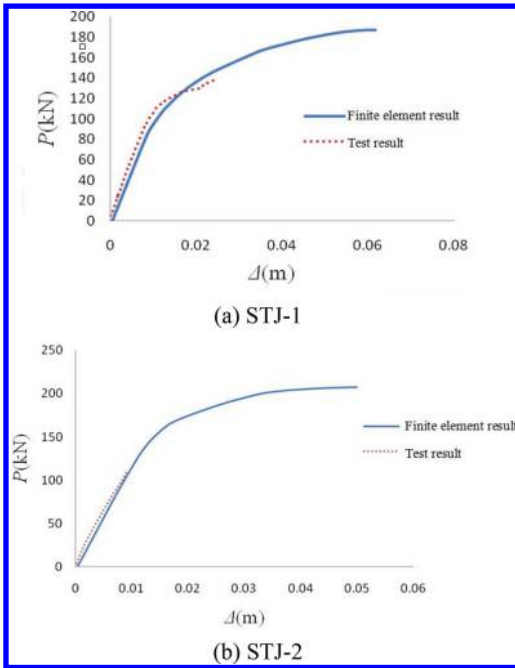


Figure 15. Comparison of calculated curve and test ones.

result had moderately good agreement with the experimental result, although the numerical results of elastic stiffness were close to the test ones. Also, similar elastic and plastic behavior was observed between the test and the analysis. The comparison between the finite element model and the test is carried out that the results of using ABAQUS for nonlinear analysis are available.

The numerical ultimate load bearing capacity of STJ-1 and STJ-2 in ABAQUS were 187.07 and 207.40kN, respectively. And load bearing capacity of STJ-2 is 10.9% higher than that of STJ-1. Furthermore, the joint plate yielded firstly in STJ-1 and the web member yielded firstly in STJ-2 and STJ-2 had higher elastic stiffness than that of STJ-1. Thus, full strength design method is preferentially used for joint design in steel staggered truss structure.

5.3 Finite element analysis on mechanical behaviors

In order to observe the mechanical behaviors of steel staggered truss system, nonlinear finite element analysis were carried out. Moreover, compression ratios and strength grade of concrete were selected to research their influence to the load bearing capacity of steel staggered truss system.

Figure 16 shows the load-displacement curves of STJ-1 with different compression ratios. The curves were close to each other at the elastic stage and they had similar yield strength and yield displacement,

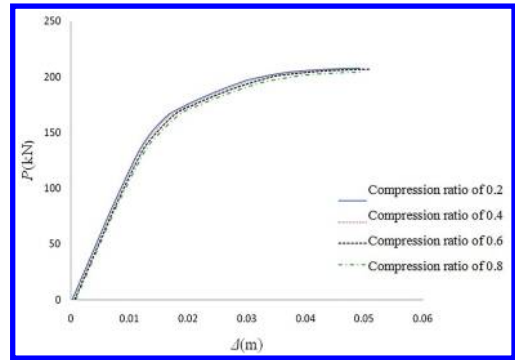


Figure 16. Load-displacement curves of STJ-1 with different compression ratios.

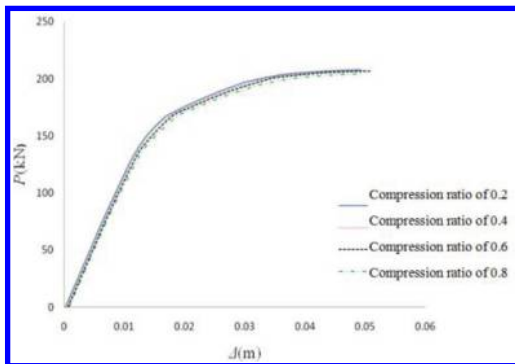


Figure 17. Load-displacement curve of STJ-2 with different compression ratios.

which indicated that the compression ratio had slight influence to the elastic stiffness of structure. The ultimate load bearing capacity of STJ-1 decreased with the increase of compression ratio. It could be inferred that the ultimate load bearing capacity decrease significantly when the compression ratio reached 0.8. So the yield strength and elastic stiffness of steel staggered truss structure with load designed hinge-joint stay the same with the increase of compression ratio while the ultimate load bearing capacity of structure decrease with the increase of compression ratio. Figure 17 shows the load-displacement curves of STJ-2 with different compression ratios. It was clear that all load-displacement curves of STJ-2 had similar yield strength and yield displacement, which indicated that changing the compression ratio had no influence to STJ-2. Furthermore, the ultimate load bearing capacity slightly decreased with the increase of compression ratio, it indicated that the change of compression ratio had slight influence to ultimate load bearing capacity of STJ-2. So changing the compression ratio had inconspicuous influence to the ultimate load bearing capacity of structure.

Table 3. Ultimate load bearing capacity of models with different strength grades of concrete.

Model	Strength grade of concrete	C20	C30	C40
STJ-1	P_u /kN	185.14	187.07	189.25
	Δ_u /mm	61.4	62.1	64.0
STJ-2	P_u /kN	204.83	207.40	208.67
	Δ_u /mm	48.1	48.7	49.0

Moreover, the ultimate load bearing capacity of STJ-2 with full strength designed joints was 10% higher than that of STJ-1 with load designed joints in the same conditions of compression ratio.

Table 3 shows the ultimate load bearing capacity of STJ-1 and STJ-2 with different strength grades of concrete strength. It could be inferred that the ultimate load bearing capacity and ultimate displacement of models increased slightly with the increase of strength grade of concrete. The ultimate load bearing capacity of STJ-1 and STJ-2 using C40 concrete was only 2.2% and 1.9% higher than that of STJ-1 and STJ-2 using C20 concrete, respectively. It indicated that changing the strength grade of concrete had no influence to the load bearing capacity of steel staged truss structure.

Furthermore, the ultimate load bearing capacity of STJ-2 with full strength designed joints was 10% higher than that of STJ-1 with load designed joints in the same conditions of strength grade of concrete.

6 CONCLUSIONS

In order to study the mechanical behaviors of truss-concrete filled square steel tubular column joints in assembly steel staggered truss structures under vertical load, two half-scaled specimens of staggered truss structures with two different types of joints were designed and static experiments were conducted. The finite element models of test specimens were established and nonlinear inelastic analyses were considered to study the influence of compression ratio and the strength grade of concrete in the steel tube to the load bearing capacity of assembly steel staggered truss structures.

The following conclusions can be made on the basis of this study:

1. Joint plate yield earlier than other members in steel staggered truss structure with load designed hinge-joints while chords yield earlier than other members with full strength designed hinge-joints.
2. The yield strength, ultimate load bearing capacity and stiffness of steel staggered truss structure

with load designed hinge-joints were lower than those of the structure with full strength designed hinge-joints. Thus, full strength design method is preferentially used for joint design in steel staggered truss structure.

3. The yield strength and elastic stiffness of steel staggered truss structure with load designed hinge-joints or full strength designed hinge-joints stay the same with the increase of compression ratio while the ultimate load bearing capacity of structure decrease with the increase of compression ratio.
4. Changing the strength grade of concrete has no influence to the load bearing capacity of steel staggered truss structure.

ACKNOWLEDGMENTS

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Hysteretic behavior study of Y-Eccentrically Braced Frames with end-plate connected replaceable links

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ABSTRACT: In order to investigate the hysteretic behaviors of Y-eccentrically braced steel frames with end-plate connected replaceable links, fourteen specimens of three series have been designed, concerning the arrangement of bolts, the number of bolts, the diameter of bolts, the thickness of end-plate of the link and the thickness of beam flange. The finite element software ABAQUS is adopted to simulate the hysteretic behavior of these series of specimens so as to analyze the effect of the relevant design parameters on the performance of hysteretic behavior, bearing capacity, lateral stiffness and the energy-dissipation capacity of Y-eccentrically braced steel frames in this paper. The results indicate that the behavior of Y-eccentrically braced steel frames with end-plate connected replaceable shear links is less affected by the connection. Only if the connections are so weak that joints fail earlier than the link will the behavior of the frame suffer a lot from the effect. In addition, the connection construction exerts a significant effect on the replaceable behavior of the link.

1 INTRODUCTION

Eccentrically Braced Frames (EBF) (Egor et al. 1988, Mastrandrea & Piluso 2009a, b, Prinz et al. 2009, Zhao & Gu 2005) are a lateral load-resisting system for building. Eccentrically braced frame systems are considered effective in earthquake design because they provide high stiffness at service load levels and exhibit excellent ductility and energy-dissipation capacity under severe load levels. Among all these EBFs, Y-eccentrically braced steel frames has the following unique properties as follows (Zhao & Gu 2006, Zhao et al. 2012, Yu et al. 2009): the Y-eccentrically braced frames links is set outside of the frame and the links are capable of dissipating most of the earthquake input energy with minimal damage to the essential structural components of the frame because most of damage is anticipated to occur within the links. Furthermore, unlike the links in other eccentrically braced frames, the links in Y-eccentrically braced frames is free of axial load effects because the vertical components of equal compression and tension brace forces cancel out with each other.

In previous researches on Y-eccentrically braced steel frames, links are weld together with frames and braces. However, experimental studies (Zhao et al. 2012, Yu et al. 2009) show that the failure of Y-eccentrically braced steel frames often occur when the weld of link and brace cracks. In order

to prevent such failure phenomenon, end-plate connection is adopted among link, brace and beam. In this way links can be replaced easily for seismic rehabilitation. Besides, the welding of key components can be performed in the factory so as to ensure its quality. Bolted connection of the replaced link can be finished on site, which saves a great amount of time and money. This paper employs ABAQUS software to systematically analyze the hysteretic behavior of Y-eccentrically braced steel frames with end-plate connected links. Results of this research can help verify the corresponding theory and provide an analytical method for the similar engineering structures.

2 FINITE ELEMENT MODEL

2.1 Constitutive relationship of steel

The constitutive model of steel will employ isotropic model provided by ABAQUS in monotonic loading while kinematic hardening rule is adopted in cycling loading in this paper. Under bilinear kinematic hardening rule, mises yield criterion and corresponding flow rule will be adopted. Young's modulus E equals 2.06×10^5 N/mm² and Poisson's ration μ equals 0.3.

All the material properties used in models presented in this paper confirm to the actual stress-strain relationship required by ABAQUS.

Material properties of steel are as follows: $\sigma_y = 235 \text{ N/mm}^2$, $\sigma_u = 420 \text{ N/mm}^2$, $\varepsilon_y = 0.114\%$, $E_s = 0.02E$ while the high-strength bolts (Chasten et al. 1992) index set as $\sigma_y = 940 \text{ N/mm}^2$, $\sigma_u = 1040 \text{ N/mm}^2$, $\varepsilon_y = 0.456\%$, $E_s = 0.02E$.

2.2 Meshing and boundary conditions

All models contain C3D8R solid units and Medial axis algorithm of sweep mesh generation technology is used to generate mesh with acceptable quality. The element mesh sizes are 100 mm, 80 mm and 80 mm for column, beam, and brace, respectively. Refined mesh of link, end plate, and flange of beam in the connection are 30 mm. The mesh for bolts are 10 mm. At the joint location, there should be contact relations involving three parts of the bolt: the contact between bolt head or bolt cap and beam flange or end plate; the contact between bolt bar and beam flange or end plate holes; the contact between end plate and beam flange. The normal behaviors of contact surface are hard contact, with coefficient of tangential Coulomb friction behavior. Friction coefficient between bolt head or bolt cap and beam flange or end plate is 0.413 while there being no friction coefficient in the contact between bolt bar and beam flange or end plate holes. All these contact settings are determined according to experiment results. Pretension P is applied at the center of bolt bar's cross section with bolt load. There are three steps when applying pretension force: firstly, use a smaller force in order to build the relations of specimens smoothly; secondly, exert to pretension P according to the requirements of study; finally, vary the force in order to reach the current length.

All the Degrees of Freedom (DOFs) of column base are restrained through coupling in order to simulate fixed end boundary condition. The translational DOFs of the top flange of beam will also be restricted to prevent out of plane instability. Apply axial force of $0.3N_y$ (N_y is the axial pressure

when the whole cross section yields) at the top of column, then couple the column with the area of beam's depth to one point. After that, enforce a lateral deflection of this point in order to implement monotonic loading and cyclic loading. Click the NLGEOM. ON button on ABAQUS to calculate the second-order effect, and the finite element model is shown in Figure 1.

3 LOADING SYSTEM

The loading system of finite element model employs ECCS fully loaded system. Firstly it analyzes one-way load curve for finite element model, then the general yield bending moment method is used to determine the yield point, in which the loading system for the specimen is as follows: use displacement loading system below loading control method, cyclic loading is multiplied by the yield displacement of yield loading according to $\Delta_y/4$, $\Delta_y/2$, $3\Delta_y/4$, Δ_y , $2\Delta_y$, $3\Delta_y$, $4\Delta_y$,..... Practice shows that the third basic loop curve overlaps with the second basic loop curve, so it can be summarized that each level displacement shall cycle twice until the specimen get damaged.

4 DESIGN OF SPECIMENS

Based on previous experiments (Shi 2015), and based on the BASE specimen, thirteen Y-eccentrically braced steel frames with end-plate connected shear links have been designed, which forms three series based on the arrangement of bolts, the number of bolts, the diameter of bolts, the thickness of end-plate of the link and the thickness of beam flange.

4.1 BASE specimen

The height of the one-story BASE specimen is 3.9 m and the span is 7.8 m. The length of the shear links is 750 mm. The specimen uses welded I-section steel, the steel grade of the plate is Q235B. The steel members used for beam, column, shear link, and brace are $H550 \times 300 \times 11 \times 18$, $H650 \times 650 \times 32 \times 40$, $H550 \times 250 \times 12 \times 12$, and $H380 \times 260 \times 12 \times 14$, respectively. In the middle of the shear links, there are three pairs of stiffeners. The thickness of these stiffeners are 12 mm and the spacing between them is 250 mm. The 30 mm thick end-plate is connected with beam and brace by twelve 10.9 M27 fiction type high strength bolts, both at the top and bottom. The geometrical dimension of the BASE specimen is shown in Figure 2, and the arrangement of bolts is shown in Figure 3(a).

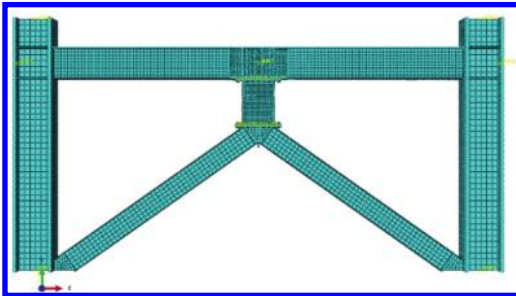


Figure 1. The mesh generation of finite element model.

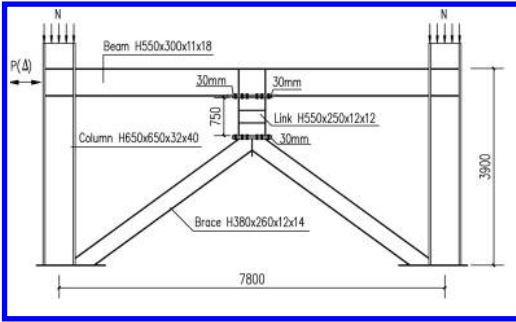


Figure 2. The dimension of BASE specimen.

Table 1. Series specimens of BOLT.

Specimens	BOLT1	BOLT2	BOLT3
The arrangement of bolts	M20 Fig. 3(b)	M22 Fig. 3(b)	M22 Fig. 3(a)
Specimens	BOLT4	BASE	BOLT5
The arrangement of bolts	M24 Fig. 3(a)	M27 Fig. 3(a)	M30 Fig. 3(a)

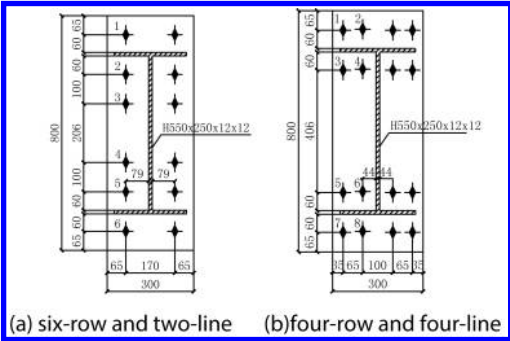


Figure 3. The arrangement of bolts.

4.2 Specimens: BOLT series

The design of the bolts in end-plate connection is essential. In order to investigate the influence of the diameter, number and arrangement of bolts on the performance, the specimens of BOLT series are designed. The level of the high strength bolts is 10.9 and the bolts have two different arrangements as shown in Figure 3. The other sizes are the same with the BASE specimen. Specimens BOLT1 and BOLT2 have bolts arranged in four-row and four-line with bolt designation as M20 and M22, respectively. Specimens BOLT3, BOLT4, and BOLT5 have bolts arranged in six-row and two-line with bolt designation as M22, M24 and M30, respectively.

Table 2. Series specimens of EP.

Specimens	EP1	EP2	EP3
End plate (mm)	18	24	27
Specimens	BASE	EP4	
End plate (mm)	30	36	

Table 3. Series specimens of BF.

Specimens	BF1	BF2	BASE
Thickness of beam flange (mm)	18	24	30
Specimens	BF3	BF4	
Thickness of beam flange (mm)	36	42	

4.3 Specimens: EP series

The thickness of end plate has a great influence on the hysteretic behaviors. If the endplate is too thin, it will cause large deformation and make it difficult to substitute the shear links. The EP series specimens only vary the thickness of the end plate on both sides of the shear links. The specimens are EP1, EP2, EP3 and EP4. The corresponding thicknesses of the end plates are 18 mm, 24 mm, 28 mm and 36 mm, respectively.

4.4 Specimens: BF series

In order to investigate the influence of different thicknesses of beam flange on the hysteretic behaviors, the BF series specimens are designed. The specimens in this series have different thickness of beam flange connected with the shear links. Corresponding partial thickness of the beam flange for specimens BF1, BF2, BF3 and BF4 are 18 mm, 24 mm, 36 mm and 42 mm, respectively.

5 PARAMETRIC ANALYSES

5.1 Specimens: BOLT series

Figure 4 is the hysteretic curves for specimens from BOLT series. The hysteretic curves of the BOLT series specimens is round and can almost coincide by two circles. The bearing capacity and stiffness do not have significant reduction under different circles. The specimens BOLT1, BOLT3 and BOLT4 are damaged due to tension failure or the shear failure of the bolts. The BOLT1 specimen achieve the displacement circulation of $4\Delta_y$, while the BOLT3 and BOLT4 reach the displacement circulation of $3\Delta_y$.

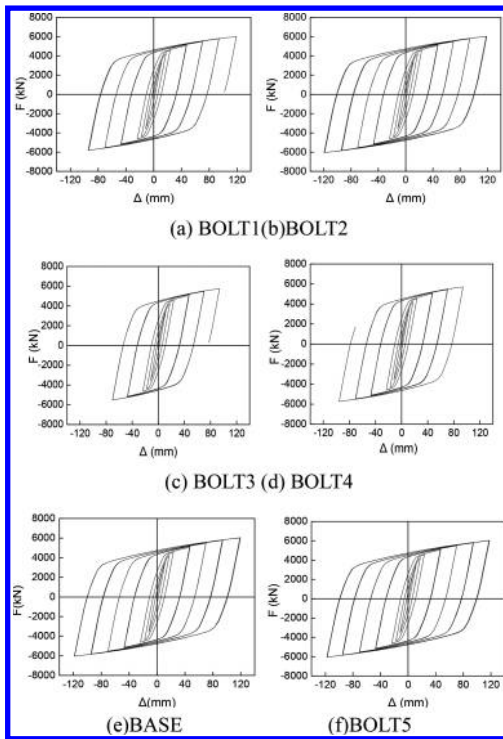


Figure 4. The hysteretic curve of the BOLT series specimens.

Besides, the other three specimens all finish the displacement circulation of $5\Delta_y$ and failed due to that fact that the stress of the web of the shear links reach the ultimate strength. The ultimate capacity of specimens BOLT1, BOLT2, BOLT3, BOLT4, BASE and BOLT5 are 5776.5kN, 6027.8kN, 5513.7kN, 5438.2kN, 6027.8kN and 6028.5kN, respectively. The plastic rotation angles of the shear link of BOLT2, BASE and BOLT5 all reach around 0.15rad which is larger than 0.08rad, the ultimate bearing capacity of the shear link reaches about 1580kN. The plastic rotation angle and the ultimate bearing capacity of these three specimens are quite similar, and the difference is within one percent. For BOLT1, the plastic rotation angle and the ultimate bearing capacity of the shear link are 0.118rad and 1457.2kN, calculated from the penultimate circle. For specimen BOLT3, the results are 0.088rad and 1309.5kN and for specimen BOLT4, the numbers are 0.087rad and 1284.6kN.

Figure 5 is the skeleton curves of the specimens within BOLT series and Figure 6 is the secant stiffness degradation curves for specimens within the BOLT series.

The total energy dissipation of specimens within BOLT series are shown in Figure 7.

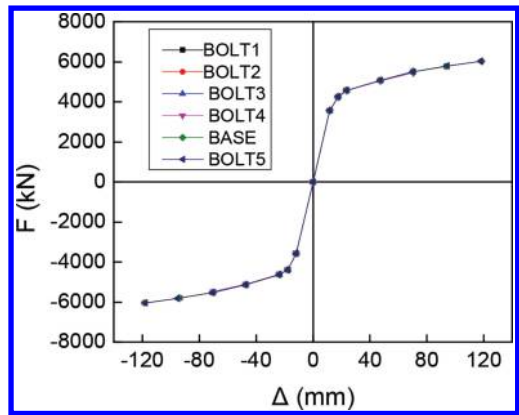


Figure 5. The skeleton curves of the BOLT series specimens.

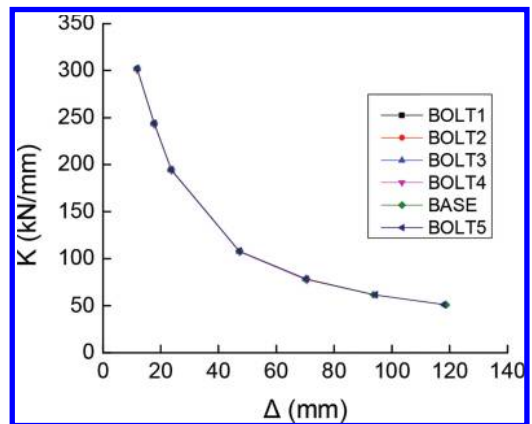


Figure 6. The secant stiffness degradation curves of the BOLT series specimens.

Specimens that experience more displacement circulation have better ductility and better energy dissipation ability. The total energy dissipations of specimens BOLT2 and BOLT5 are similar to that of BASE specimen. However, the whole energy of the BOLT1, BOLT3 and BOLT4 specimens is far less than that of BASE specimen due to the breakdown of the bolts.

5.2 Specimens: EP series

The hysteretic curves of specimens within EP series are shown in Figure 8. It indicates that the hysteretic curves of the EP series specimens are round and can coincide by two circles. The bearing capacity and stiffness do not have significant reduction during the different circles. All of the specimens finish $5\Delta_y$ displacement circulation and do not have much difference shown in the hysteretic curves.

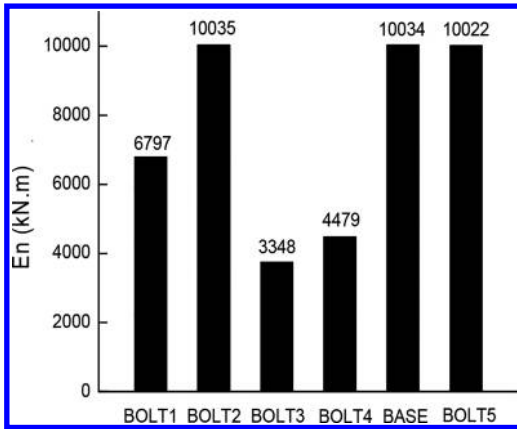


Figure 7. The total energy dissipation of the BOLT series specimens.

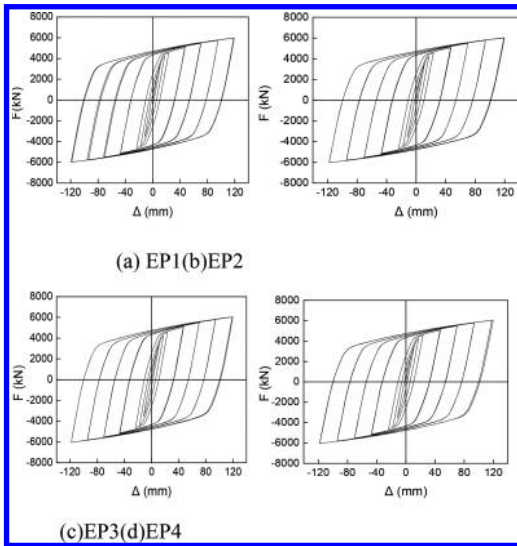


Figure 8. The hysteretic curve of the EP series specimens.

The stress of the web of the replaceable shear links reaches the ultimate strength and results in the structure break down. The ultimate capacity of the EP1, EP2, EP3, BASE and EP4 specimens are 5960.7kN, 6016.4kN, 6021.3kN, 6027.8kN and 6053.2kN, respectively. With the increase of the thickness of the end plate, the ultimate capacity grows. Comparing specimen EP1 with BASE specimen, the difference in the ultimate capacity is about 2 percent, which is very small. The plastic rotation angles of the shear link of every specimen reaches around 0.15rad which is far more than 0.08rad. The ultimate bearing capacity of the shear link of EP1, EP2, EP3, BASE and

EP4 are 1499kN, 1561.6kN, 1571.5kN, 1578.7kN and 1586.8kN, respectively. Similar to the whole function, with the increase of the thickness of the end plate, the ultimate capacity grows. When the end plate is thin, the amplification is fast. When it reaches a certain thickness, the amplification is quite slow. Compare EP1 and BASE specimen, the difference in ultimate bearing capacity is 5.3%. Therefore, the influence of the thickness of the end plate on the shear links is little bit more critical than that on the whole specimen, but the difference is not very significant. It is because when the end plate is thin, the deformation is huge and the capacity of anti-bending is greatly reduced, therefore, the range of the shear capacity turned out to be smaller.

Figure 9 is the skeleton curves of the specimens within EP series and Figure 10 is the secant

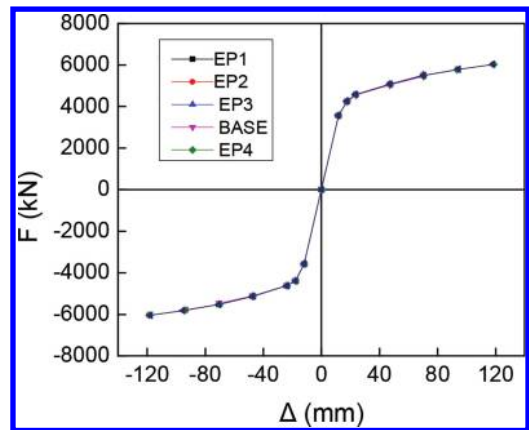


Figure 9. The skeleton curves of the EP series specimens.

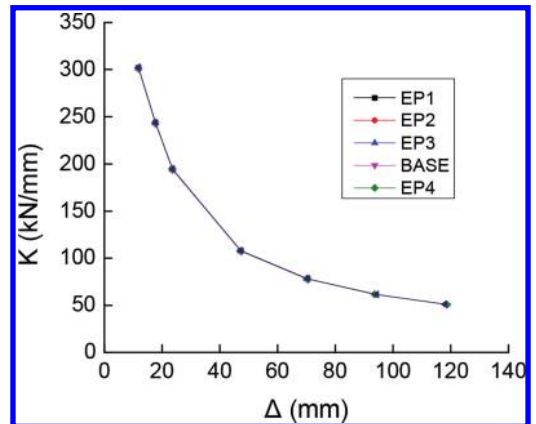


Figure 10. The secant stiffness degradation curves of the EP series specimens.

stiffness degradation curves for specimens from EP series.

The total energy dissipation for specimens within EP series is shown in Figure 11. The more the displacement circulation of the specimen is, the better the ductility is and the more the energy dissipation is. The total energy of the EP and the coefficient of energy dissipate is similar to the BASE specimen. The capacities of both of them are good. Changing the thickness of the end plate of the replaceable shear links has little influence on the energy dissipation.

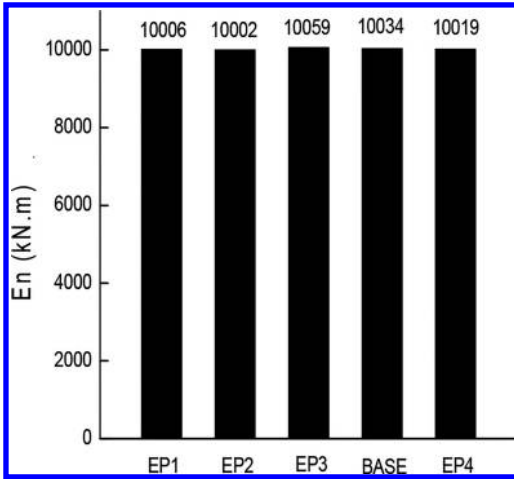


Figure 11. The total energy dissipation of the EP series specimens.

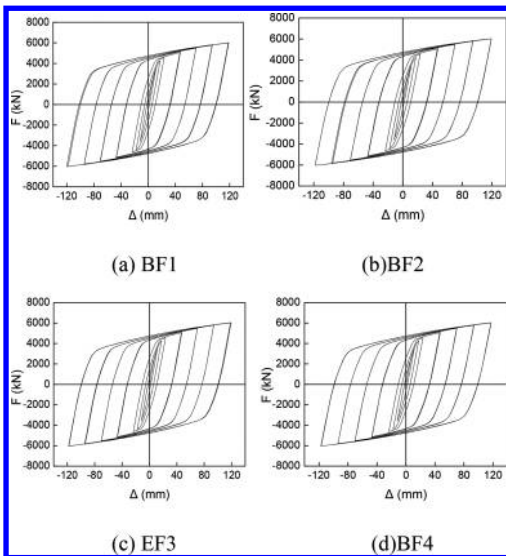


Figure 12. The hysteretic curve of the BF series specimens.

5.3 Series specimens of BF

The hysteretic curve of the specimens within EP series is shown in Figure 8. We can see that the hysteretic curves for EP series specimens is round, and almost can coincide by two circles. The bearing capacity and stiffness do not have significant reduction during the different circles. All of the specimens finish $5\Delta_y$ displacement circulation and do not exhibit much difference in the hysteretic curves. The stress of the web of the replaceable shear links reaches the ultimate strength and result in the structure break down. The ultimate capacity of specimens BF1, BF2, BASE, BF3 and BF4 are 5994.9kN, 6022.5kN, 6027.8kN, 6030.2kN and 6041.1kN, respectively. With the increase of the thickness of the flange of the beam, the ultimate capacity grows, but the growth is small. The plastic rotation angles of the shear link of every specimen arrives around 0.15rad which is far more larger than 0.08rad. The ultimate bearing capacity of the shear link of BF1, BF2, BASE, BF3 and BF4 are 1448.9kN, 1572.0kN, 1578.7kN, 1579.7kN and 1580.6kN, respectively. Like the whole function, with the increase of the thickness of the flange of the beam, the ultimate strength grows, but the growth is small.

Figure 13 is the skeleton curves of specimens within BF series and Figure 14 is the secant stiffness degradation curves of the BF specimens.

The total energy the BF series specimens dissipate is shown in Figure 15. The more the displacement circulation of the specimen is, the better the ductility is and the more the energy dissipation is. The total energy of the BF and the coefficient of energy dissipate is similar to the BASE specimen that the capacity of them are both good. Changing the thickness of the flange of the beam has little influence on the energy dissipation.

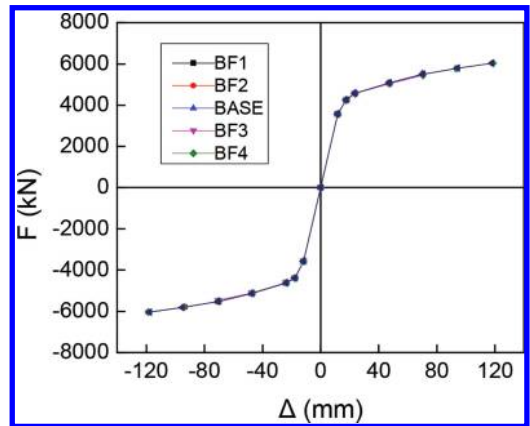


Figure 13. The skeleton curves of the BF series specimens.

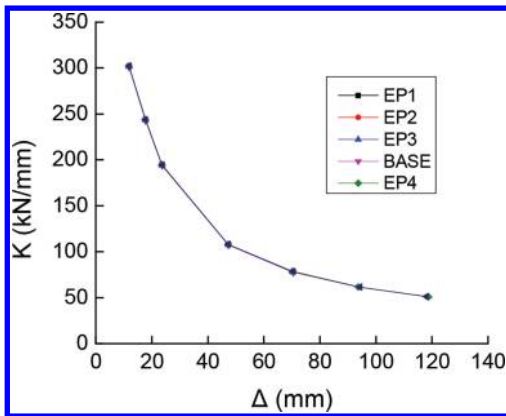


Figure 14. The secant stiffness degradation curves of the BF series specimens.

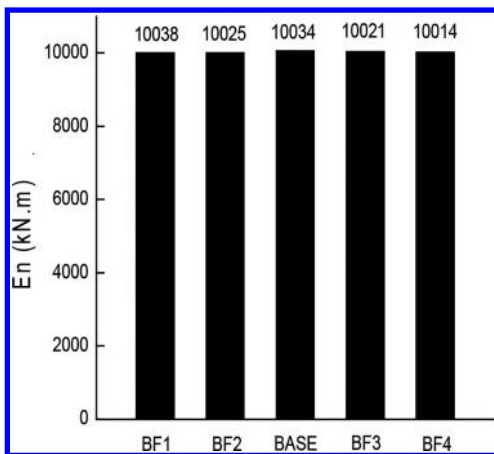


Figure 15. The total energy dissipation of the BF series specimens.

6 CONCLUSIONS

Using finite element program—ABAQUS, this paper analyze thirteen Y-eccentrically braced steel frame specimens from three series with different bolt diameter, bolt number, bolt arrangement, the thickness of end plate of the link and the thickness of beam flange and investigated their hysteretic behaviors with end-plate connected replaceable shear links. Some conclusions can be summarized as follows:

1. The diameter of the bolts has tremendous influence on the behavior of the beam. If the connection are so weak that joints fail earlier than the link, than it cannot meet the required 'strong connection-weak members' principle.
2. The thickness of the end plate of the link has little impact on the overall performance of the

specimens, but it does makes a big difference on the behavior of the replaceable link.

3. Similarly, thickness of the beam flange has little impact on the whole performance of the specimens, but makes a big difference on the behavior of the replaceable link. The thickness of beam flange is advised to be 1.0~1.2 times of the thickness of the end plate.
4. Based on the parametric analyses of the three specimen series, it is suggested that prying force coefficient shall be increased from 1.3 to 1.4 to enhanced the reliability of joints when calculating the ultimate capacity of bolts.
5. The influences of connection performance in Y-eccentrically braced steel frames with end-plate connected replaceable shear links turns out to be quite small, except the case that if the connection is so weak that it causes premature failure before the breakdown of the replaceable link.

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An initial concept design of an innovative flat-plate Solar Thermal Facade for building integration

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ABSTRACT: Compared to the conventional solar thermal collectors, Solar Thermal Façade (STF) is a prevalent implementation by incorporating a solar thermal collecting device into building façades. It has been greatly driven by a desire for aesthetic architecture, a practical demand for improved indoor thermal performance, and the simultaneous aspiration for on-site energy/thermal generation in a building. The proposed flat-plate STF is an optimal choice in both heat transfer performance and façade intensification, in addition with a high-tech expression, easy design assortment with the metal window frame and the exposed metal load-bearing section, flexible profiles and expressive textures. Meanwhile, it can provide a Domestic Hot Water (DHW) solution for a typical 3-member family apartment in a high-rising residential building in Shanghai, China. This paper aims to present a comprehensive study of STF building integration design from multi-angles of technical, functional, constructive and formal aspects based on the proposed STF system.

1 INTRODUCTION

In order to achieve the global carbon emission target, the high fraction of locally available renewable energy sources in energy mix will become necessary in addition to a significantly reduced energy demand. Solar energy is one of the most important renewable sources locally available for use in building heating, cooling, hot water supply and power production. Truly building integrated solar thermal facade systems can be a potential solution towards the enhanced energy efficiency and reduced operational cost in contemporary built environment.

According to the vision plan issued by European Solar Thermal Technology Platform (ESTTP), by 2030 up to 50% of the low and medium temperature heat will be delivered through solar thermal (ESTTP 2009). However currently, the solar thermal systems are mostly applied to generate hot water in small-scale plants. And when it comes to applications in field of solar space heating, large-scale plants in urban building projects, hotels and local heating networks, the insufficient suitable-and-oriented roof of most buildings may dictate solar thermal implementation. For a wide market penetration, it is therefore necessary to develop new solar collectors with feasibility to be integrated with building components. Such requirement opens up a large-and-new market segment

for the STF system, especially in future district or city—level energy supply.

STF is defined as the “multifunctional energy facade” that differs from conventional solar panels in that it offers a wide range of solutions in architectural design features (i.e., colour, texture, and shape), exceptional applicability and safety in construction, as well as additional energy production. It has flexible functions of heating/cooling buildings, providing hot water, power generation and improving the insulation and overall appearance of buildings. These STF technologies would boost the building energy efficiency and literally turn the envelope into independent energy plant, creating the possibility of solar-thermal deployment in high-rise buildings.

2 WORKING PRINCIPLE OF TYPICAL SOLAR THERMAL FACADE SYSTEM

The typical STF system is schematically shown in [Figure 1](#). The system normally comprises a group of modular STF collectors that receive the solar irradiation and convert it into heat energy, whereas the heating/cooling circuits could be further based on the integration of a heat pump cycle, a package of absorption chiller, modular thermal storage and system controller.

In case of unsatisfied weather conditions, a backup/auxiliary heating system is also integrated

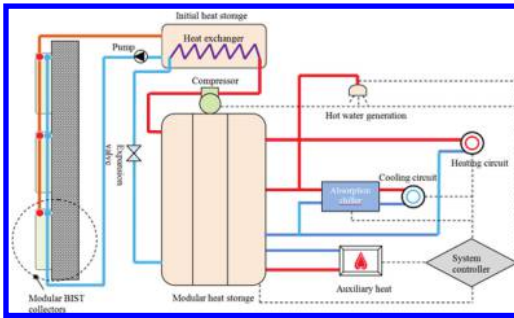


Figure 1. Schematic of modular solar thermal facade system for building services.

to guarantee the normal operation of system. In the typical solar thermal facade system, the overall energy source is derived from solar heat, which is completely absorbed by the modular STF collectors. This part of heat is then transferred into the circulated working medium and transported to the preliminary heat storage unit, within which heat transfer between the heat pump refrigerant and the circulating working medium will occur. This interaction will decrease the temperature of circulating medium and enables it absorbing heat in the facades for next circumstance.

Meanwhile in the heat pump cycle (compressor-condenser-expansion valve-evaporator), the liquid refrigerant will be vaporized in the heat exchanger, which, driven by the compressor, will be subsequently converted into higher-temperature-and-pressure, supersaturated vapour, and further release heat energy into the tank water via the coil exchanger (condenser of the heat pump cycle), leading to temperature rise of the tank water. Also, the heat transfer process within the coil exchanger will result in condensation of the supersaturated vapour, which will be downgraded into lower-temperature-and-pressure liquid refrigerant after passing through the expansion valve. This refrigerant will undergo the evaporation process within the heat exchanger in the initial heat storage again, thus completing the heat pump operation. When the water temperature in tank accumulates to certain level, then water can be directly supplied for utilization or under-floor heating system. For the cooling purpose, additional appliance of absorption chillers should be coupled with.

3 CATEGORIES OF SOLAR THERMAL FAÇADE TECHNOLOGIES

The solar thermal facade can be classified into air-, hydraulic- (water/heat pipe/refrigerant) and PCM-based types according to heat transfer medium.

Air based type is characterized by lower cost, but lower efficiency due to the air's relatively lower thermal mass. This system usually uses the collected solar heat to pre-heat the intake air for purpose of building ventilation or space heating. Hydraulic-based STF's are most commonly used building integrated solar thermal devices that enable the effective collection of the striking solar radiation and conversion of it into the heat for purpose of hot water production and space heating. The PCM-based type is usually operated in combination with air, water or other hydraulic measures that enable storing part of the collected heat during the solar-radiation-rich period, and releasing it to the passing fluids (air, water, or others) during the solar-radiation-poor period to achieve a longer period of STF operation. In general, the economical air type is good at anti-freezing/boiling and non-corrosive with simple structure but lower in heat capacity and with potential leakage and noise. Cost effective water type performs well in cold climates for its higher specific heat, but potential mineral deposits, possible leakage, freezing, corrosion and overheating is unavoidable. As compared, higher efficient refrigerant type is both smaller in storage volume and fluid volume, but its limitations lie in higher cost and unbalanced liquid distribution. Heat pipe type shows compact and super highest heat exchange ability while being low in hydraulic and thermal resistances, but the higher vacuum degree is a limitation in processing. PCM type aims to improve thermal comfort and building envelop with diversity building integration methods. Its disadvantages are difficult operation with complex behavior, diverse affection factors and sensitive heat injection. Therefore, further selection of these solar thermal facade technologies should be carried out depending on different application scenarios.

4 CONCEPTUAL DESIGN OF AN INNOVATIVE SOLAR THERMAL FAÇADE SYSTEM

The proposed STF system is designed to provide DHW for a typical 3-member family apartment in a high-rising residential building in Shanghai, China. This system is expected with lower cost, higher efficiency, façade integrative and aesthetical appearance to provide an alternative to an existing roof-mounted, conventional wall-hung evacuated-tube or flat plate solar water heating system.

4.1 Relevant standards for STF design

First of all, there are several relevant standards for STF design. European Standards EN 12975-1,

EN 12977-1 and EN12978-1 specify the requirements related to the product itself containing high temperature resistance, exposure, external thermal shock, internal thermal shock, rain penetration, impact resistance and mechanical load. And EN 12975 standard is updating to address the global concerns in evaluating the conventional and advanced solar thermal products (Zhang 2015). In addition, the solar thermal facade should provide protection from external conditions, as solar irradiation temperature, humidity, precipitation and wind in order to achieve an acceptable indoor thermal comfort (Roecker 2009) (Probst 2008). Additionally, there are statutory instruments, directives and standards for the facade application to possess the construction, hydraulic and hygiene characteristics. The integration constraints has illustrated in Figure 2.

4.2 Choose of absorber material

With regards to the material selection of the innovative solar thermal façade, it is cohered with the metal for its hi-tech appearance, strong intention and highly prefabricated feature inspired from the roll-bond absorber. The most popular constructive metals are steel, aluminium, copper, titanium-zinc and bronze. As a potential building component, the material should also pay special attention to the most common and significant physical phenomena. Thermal dilation is a movement that can be accommodated by appropriate forms of jointing and assembly. The corrosion is an alternation on the surface appearance of majority metals from environmental influences that requires higher maintenance costs. Meanwhile, the connection between water pipe and metal absorber also need to avoid a regenerative anti-corrosion layer which changes the appearance with the time, such as copper or a contact between a third group, such as iron and steel. In a consequence, stainless steel seems to be an optimal choice with combinations in both heat transfer performance and façade intension.

In another aspect of the unglazed absorber type, the proper mechanical joining is the important link to guarantee a good thermal bond through

processing techniques (Kalogirou 2004). The modern manufacturing techniques of cutting, folding, stamping and welding introduced by industry enable a super thin the full flow-through stainless steel absorber.

4.3 Configuration of the innovative STF

The basic configuration of the innovative STF is showed in Figure 3. The thermal absorber is made up by two parallel thin flat-plate metal sheets, one of which is extruded by machinery mould to formulate arrays of mini corrugations, while another sheet remains smooth for attaching building wall. A laser-welding technology is applied to join them together, forming up the built-in turbulent flow channels. Such unique compact structure engenders not only high heat transfer capacity but also convenience in rapid assembly and installation. If several absorbers are connected together as a complete larger area of building facade the question of the connecting arrangement arises. Therefore, a flexible connection arrangement scheme is proposed through

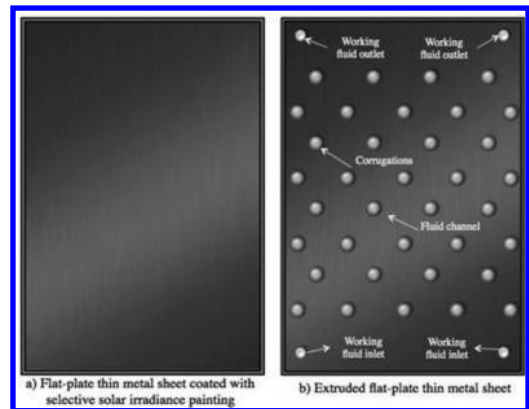


Figure 3. Elementary diagram of the innovative super thin flat-plate metal solar absorber.

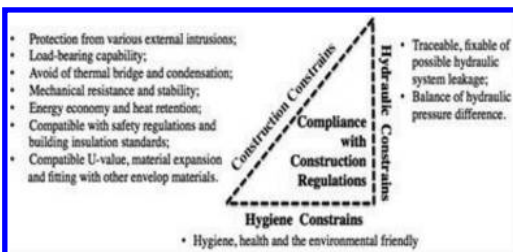


Figure 2. Integration constraints for STF.

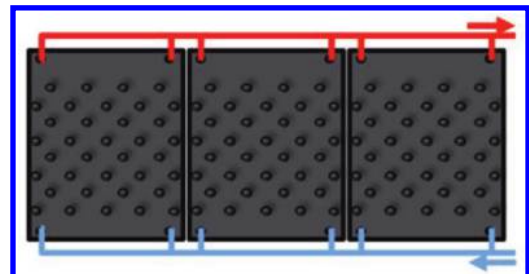


Figure 4. Parallel connection arrangement of the proposed STF.

two sets of inlet and outlet around the four corners of the flat plate panel. Basically, there are three kind of arrangements as parallel, series and combined.

In the case of parallel connection, all the distributing and collecting header pipes are connected as

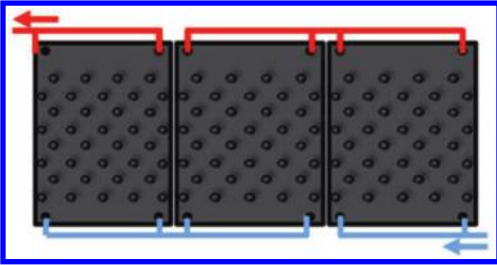


Figure 5. Series connection arrangement of the proposed STF.

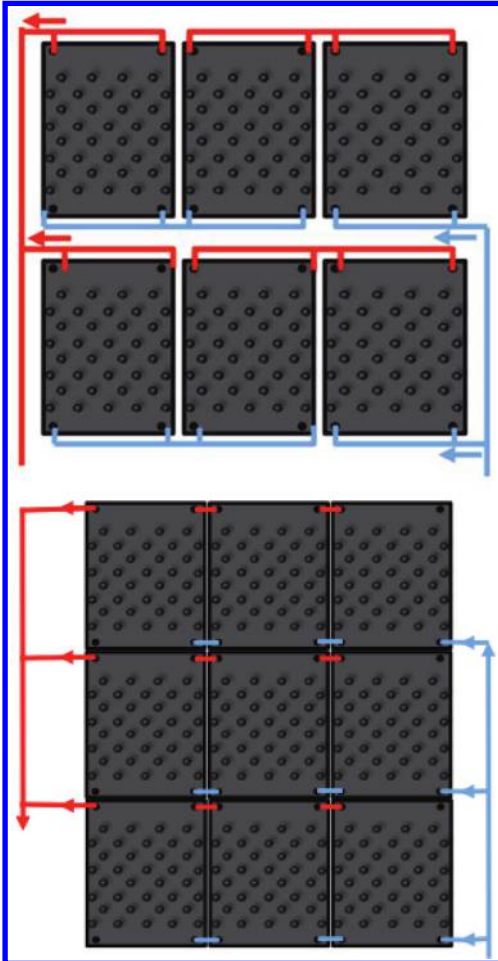


Figure 6. Combined connection of the proposed STF.

illustrated in Figure 4. In case of series connection, all the absorbers are connected successively as illustrated in Figure 5. And in the case of combined connection, the absorbers in each group are in parallel connection first, and then multiple groups are connected in series, making it possible to combine the advantages of both connection principles as illustrated in Figure 6.

Figure 7 demonstrates the conceptual application design of the solar thermal façade. It can be seen that the whole system is simply in structure with main stainless steel solar absorber, accessorial metal profiles, piping and insulation.

4.4 Reflexions of solar thermal façade applications

The proposed STF system, shown in Figure 8 is simple in composition with a metal absorber, a circulating water pump and an insulated water tank. The stainless steel absorber is thin to integrate into building envelope as metal cladding panel converting the absorbed solar energy into thermal energy.

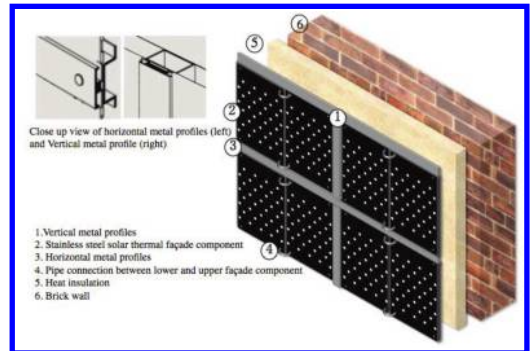


Figure 7. Conceptual design of the façade application with the close up view of horizontal and vertical metal profiles.

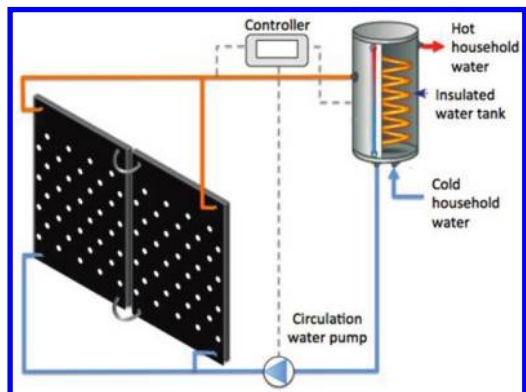


Figure 8. Conceptual system connections.

Next, the absorbed solar heat is transported to the pipe coil in the insulated water tank through circulation water pump to exchange heat.

5 CONCLUSION

This paper proposed the initial concept design of a metal cladding based STF technology that recently enjoys the popularity in building facade application for its high-tech expression, easy match with the metal window frame and the exposed metal load-bearing section, as well as flexible profiles and expressive textures. It inspires a promising direction for STF system owing to its simple structure, light and thin panel and optimal heat exchange performance. It is expected to break through the limitations of conventional STF, and achieve a broader market deployment.

ACKNOWLEDGEMENTS

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Sustainable composite structures

Driving architectural design with material innovation: A design research approach

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ABSTRACT: This paper presents outcomes of a design research investigation into the architectural implications of innovative uses of concrete. The issue of sustainability is addressed in a variety of ways at the level of architectural design, in response to recent material innovations. The study presented employs design as a method of investigation to develop new visions for future living and working. Results include new types of architectural space developed through the use of translucent concrete, high-strength concrete, fabric-formed concrete, fiber-reinforced concrete, printed concrete, insulated concrete forms and a variety of concrete surface treatments.

1 INTRODUCTION: CONCRETE FUTURES

1.1 *China's concrete future*

Ever since the beginning of the economic boom more than three decades ago, reinforced concrete (referred to as 'concrete' in the following) has dominated Chinese architecture like no other material. Concrete structures form the basis of China's large-scale urbanization process, with China consuming 59 percent of globally consumed cement in 2014 (Global Cement Report 2015). Until today, however, the use of concrete in China is remarkable more on the level of scale than on the level of architectural quality. Even though concrete structures are ubiquitous, their role in forming what could be called China's 'new vernacular' has not been adequately reflected in architectural design yet. In addition, the high speed of the building boom over the past four decades has resulted in a large number of buildings that are lacking in terms of their fit to contemporary functional requirements as well as their construction and material quality (Steadman 2013). As a result, more than half of China's residential buildings will require demolishing and rebuilding within the next twenty years (Qian 2010). Most buildings that employ concrete as their main construction material do not engage with their materiality in an innovative way as part of their architectural design—instead, concrete structures have become characteristic of unimaginative and architecturally lacking buildings. In Chinese architecture, concrete is typically hidden from view with elaborate care (Mayer 2011). This not only contrasts with the great potential of the material concrete to be used and developed in innovative ways as part of architectural spaces (Roth 2012, Koren & Hall 2012) but also causes waste and

prevents further developments in innovative as well as sustainable architecture and construction. The contemporary emphasis on economy and speed is however likely to change in the near future, as a consequence of an increasingly slow economic and urban development. Great ecological challenges have arisen as a result of China's economic success in the past decades (Mayer 2011), and design as well as construction methods will need to be reconsidered in the light of future requirements of sustainability and overall increased quality.

1.2 *The concrete futures design research project*

The study presented in this paper addresses the theme of innovative concrete architecture by focusing on the type of the medium rise tower that is common throughout China's new urban developments. The *Concrete Futures* project task was the architectural design of an innovative mixed-use 100 m tall tower, utilizing concrete as the main construction material. Design proposals for the tower explored the mutual relationships of typologic, spatial, structural, technical, and programmatic aspects and functional requirements within the broad framework of the material concrete. In addition, design proposals considered the specific conditions of the local Chinese context. Proposals were designed for either a site in Suzhou, within a newly developed urban context, or a dense urban site in Shanghai with a rich cultural and historical context.

1.3 *Research method*

The study adopted a design research approach, as described by Koskinen et al. (2011) and Laurel

(2003). As outlined by Downton (2003:55), design research (also called design-driven research) proceeds through applied design. In addition to design proposals and personal knowledge, design research also produces collective knowledge in the form of research results that can be made explicit in writing. Design research employs design-based forms of rigour that concentrate on frequent examination of and reflection on interim results. One of the main differences between design research and science is that design research processes are specific and not replicable (Downton 2003:99)—results can be tested for quality but not repeated.

The study comprised ten students working on the design theme—outlined in section 1.2 above—over the course of two semesters, as part of a BEng Architecture final year project. Students developed individual design proposals in response to the overall brief calling for a new architectural vision based on innovative uses of concrete. All design proposals addressed architectural and technological/structural ideas in an integrated manner. Students initially focused on material properties and uses of concrete as well as on existing case studies. This initial research phase was then extended into a design research process that continued research on precedents into applied explorative design processes, taking into account students' choice of architectural direction.

Proposals were required to address a breadth of requirements and to respond not only to technological aspects of concrete construction, but also to the particulars of the functional programme and site context. This process can be described as one in which ideas are mutually generated between architectural and technological considerations. Throughout the study, the design research process was pursued in a multi-modal manner, employing among others models, drawings and text. A series of nine interim reviews throughout the design development—many of them joined by external reviewers—provided a rigorous context for proposal development.

1.4 *Scope and limitations*

The *Concrete Futures* project focused on future architectural potentials of concrete. To enable a broader range of visions, design proposals were not limited to conform to current building codes. The project outcomes are thus to be understood as visionary prototypes that raise as many (architecturally productive) questions as they provide answers. Limitations of this study arise primarily from its relatively short duration of two academic semesters. Due to the nature of design research studies, results are relatively specific, but lead to a variety of potential future explorations, as outlined

in the final section of the paper. Results and findings made as part of this project demonstrate not only integrated design, taking into account a variety of future technical and social challenges, but also develop new questions regarding sustainable architectural futures of Chinese concrete construction.

2 ARCHITECTURAL IMPLICATIONS OF MATERIAL INNOVATION

Even though concrete pervades the construction industry in China, construction practice and innovative architectural uses of the material tend to lag behind overseas construction approaches (Mayer 2011). This is due to various factors, among them are the limited quality of workmanship provided by a transient unskilled workforce and the sheer speed at which buildings are required to be designed and completed by clients (Campanella 2011, Chung et al. 2002). These conditions strongly affect the architectural design of building types primarily relying on concrete as construction material—such as the medium size tower building of 25–30 floors that this study addresses. With the present and likely continuing future slow-down of the economic boom and greater affluence of the urban middle-class population (Mayer 2011), a change in expectations can however be projected that will increasingly emphasize quality rather than quantity. The study presented here explores how the architectural type of the medium size tower could be both challenged and further developed through innovative uses of concrete, and how such innovations can also lead to more sustainable futures. In this context, the interplay of material, technology and architectural design is taken to be a driving force in generating new architectural spaces, new spatial experiences and new patterns of use and inhabitation.

3 SUSTAINABILITY IN ARCHITECTURE

3.1 *Between technical and cultural perspectives on sustainability*

The term 'sustainability' has been interpreted in various ways within the field of architecture. In a broad manner, sustainable architecture seeks to address problems generated by short-term thinking in design and construction processes, to prevent negative impacts of current developments on future potential developments and society at large (Brundtland Report 1987). From this broad aspiration, different takes on sustainability have

developed, based on different ideological underpinnings (Jarzombek 2006). Most contemporary approaches to sustainability in architecture focus on building technology, often under the heading of 'green architecture'. This has led to new building codes and international standards, such as 'LEED' (Leadership in Energy & Environmental Design). As Jarzombek (2006) however discusses, sustainable building technologies tend to be employed not only based on ecological, but often also economical aims, and may not always serve the long-term perspectives they are intended to support. The long-term effect of sustainable technologies in building construction is still debated, involving at times conflicting ethical, engineering, and political orientations.

Extending the purely technical focus of sustainable design, other perspectives on sustainable architecture have been proposed that address a broader cultural framework in the question of how humans relate to nature and the environment (McGrath 2013). The movements of New Urbanism and New Classical Architecture for example arose from sustained criticism of modernist approaches to architecture and urbanism, and focus on providing alternative spaces for living that claim long-term social sustainability. The study presented here addresses sustainability at both levels: sustainability motivates the invention of new types of material, but also their integration into architectural schemes that anticipate and accommodate future inhabitants' needs and expectations, as outlined by McGrath (*ibid.*). Sustainability is thus understood not necessarily as implying an exclusive focus on energy, but includes a broader perspective that integrates material, social and environmental aspects.

3.2 Sustainable concrete futures

While the materiality of concrete has been integrated with architectural design more explicitly in recent Chinese architectural practice, these advances tend to be primarily aesthetically motivated (see for example Wang 2012). This study does not exclude, but also extends beyond an emphasis on aesthetics and focuses on recent technological innovations in concrete construction and material research, exploring new architectural types potentially resulting from such innovations. Aspects of concrete construction addressed in the following include construction technology, material science, structural design and building performance. A variety of concrete construction technologies were explored, such as high performance concrete, fiber reinforced concrete, precast concrete, exposed concrete structures, ICF (Insulated Concrete Forms), fabric-formed

concrete and 3D printed concrete. Extending this technological aspect, design proposals in addition engaged innovative and sustainable uses of concrete in a broader social and cultural context. This approach allows not only the exploration of architectural potential inherent in existing material innovations, but also generates new questions and visions that may in turn lead to yet further material innovations.

In this study, individual design proposals adopt a range of perspectives on sustainability. Some focus on social aspects, developing forms of urban living that enable better use of social as well as environmental resources in the long term. One design proposal creates a hub for innovation to support future innovative technology development, thus enabling better designed future design and technology. Some proposals provide improved quality and comfort for simple and resource-efficient living styles, in particular within urban contexts. Yet other design proposals focus on long-term use of buildings and fitness for future requirements.

4 RESULTS

The study produced a wide range of results that demonstrate the potential of material-focused design to generate innovative types of architectural spaces. In the following, central aspects of these types are listed and illustrated with architectural design proposals. To fit the scope of this paper, the following sections concentrate on these particular aspects. Individual projects are thus presented only in excerpts and are not introduced in their entirety.

4.1 Sculptural space

Contemporary concrete structures in China are typically constructed in rectilinear assemblies, in the form of frame structures composed of columns and beams, laterally stabilized by occasional cores and shear walls. This architectural vocabulary follows modernist ideals of efficiency and economy, but also imposes certain limitations on the types of space made available to human experience. In the search for alternatives to rational modernist architectural design, the sculptural potential of reinforced concrete was explored already early on, both in architecture and engineering. Examples can be found in the works of Saarinen and the later works of leCorbusier, or the works of Pier Luigi Nervi and Sergio Musmeci. Sculptural approaches to architectural and engineering design were however continuously hampered by the intense technical effort required in the construction process of sculptural concrete.

In recent years, advances in and proliferation of digital fabrication techniques have enabled new possibilities for the design and construction of sculptural concrete, such as the TailorCrete research project (TailorCrete 2015), which aims for this new technology of constructing sculptural concrete to “initiate a transition from the rectangular monotony of today’s industrialised concrete buildings (...), to new industrialised unique concrete structures without the need for expensive and labour-intensive manual construction processes” (ibid.).

Another technique that promises to enable sculptural customization of concrete buildings is 3D printing. The Chinese company WinSun, located in Suzhou, has recently made great strides in increasing the scales of buildings they print (Davison 2015). WinSun integrates sculptural and structural capabilities of the new production technique with sustainable features of the material, such as saving material and cost during construction as well as using recycled materials. The concrete printing mixture used by WinSun for 3D printing (Fig. 1) for example is made from recycled rubble, fibreglass, steel, cement and binder, and takes 24 hours to dry.

The *Lightwell* design proposal for a residential tower with integrated shopping arcade in Shanghai demonstrates how the new sculptural possibilities of concrete can generate new qualities of architectural spaces (Fig. 2). The irregular sculptural shapes of the internal atrium walls are not merely decorative, but also structural, similar to the architecturally performing exposed sculptural concrete structure designed by Arata Isozaki for the Himalayas Centre in Shanghai. Difficult to achieve with contemporary Chinese construction methods, such uses of concrete may become more commonplace with the spreading of new technologies such as that developed by WinSun.

4.2 Experiential concrete

In addition to the possibility of creating sculptural spaces (as outlined in the previous section),



Figure 1. 3D printed concrete by Chinese company WinSun.



Figure 2. The *Lightwell* design proposal for employing sculptural internal spaces to enhance high-density urban living.

new spatial qualities of concrete buildings can also be generated by increased consideration of human experience. Made available at a more immediate sensory level, concrete can thus contribute to elegant yet simple types of architectural spaces. Exposed concrete is still rare in China, but can be expected to be adopted more frequently in the future with improvements in construction technology, material quality and skill levels of the construction workforce. In the *Outward and Inward*—看与被看 design proposal, these premises are adopted as a basis to explore potential future architecture. In the spirit of the multi-dimensional design approaches employed in classic Chinese scholar gardens, the design proposal demonstrates how concrete architecture can provide similarly rich spatial experiences while maintaining a simplicity that enables future adaptability and long-term use. The proposal features large open (and adaptable) spaces that allow for cross-ventilation and multi-functional use (Fig. 3).

The proposal adopts design and construction strategies employed in the recently completed ‘Vertical Forest’ experimental housing in Milan, Italy, with the aim of re-introducing greater amounts of greenery into urban and high-rise living. Employing exposed concrete (as illustrated in Fig. 4) requires increased effort during the process of construction but also maintains adaptability and saves resources by reducing the overall requirements for claddings and interior finishes.

While exposed concrete is not yet widely accepted in contemporary Chinese residential architecture, it has the potential to become accepted more widely in the future, following the upgrading of construction technology. This broader appreciation of both simplicity and quality in concrete architecture can for example be found in Japanese as well as Swiss architecture.



Figure 3. Creating simple yet natural environments in a reduced vocabulary of exposed concrete architecture.



Figure 4. Engaging simplicity as a hallmark of architectural quality and elegance.

4.3 A new thinness

The *Living in ambiguous fields* design proposal (Fig. 5) focuses on recovering a human scale within urban high-rise residential architecture. New materials such as high-strength concrete and fiber-reinforced concrete allow for significantly reduced thickness of structural elements. The primary aim for economic and ecological advantages driving the innovation processes of these new materials are complemented by new architectural effects. Slabs, beams and walls can not only be constructed in less massive, more elegant shapes, but can in addition be exposed (see previous section 4.2).

For the *Living in ambiguous fields* design proposal, a typical medium-rise tower as found in China's urban context was reconsidered to take advantage of the now possible thinness in concrete construction. The tower is dissolved into several small-footprint towers, similar to the 'pencil towers' found in Hong Kong and Manhattan (Clines 2015). Each of the towers contains two-floor apartments connected by shared courtyards. In contrast to conventional high-rise urban residential buildings,

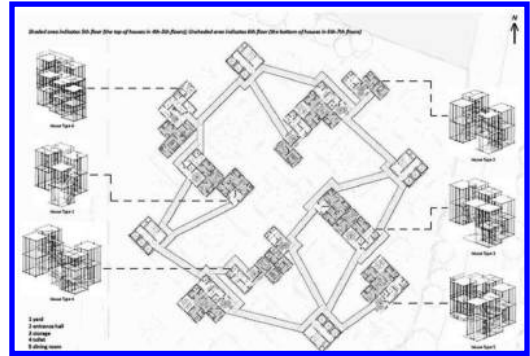


Figure 5. Employing high-strength concrete to create dense yet human-scale living environments.

thin columns, beams and floor plates are designed to generate the spatial experience of living in a small individual house. This contrasts with the experience of living in an apartment within a large housing block that has an overbearing structural presence.

4.4 New transparencies

The integration of concrete and glass fibers has recently introduced translucent concrete as a construction material at the building scale. LiTraCon and other manufacturers, produce a variety of translucent concrete blocks and panels for use in design and construction. While translucent concrete is already in use, it still adds considerable expense to a building, which is likely to change in the future. Among architects, this new material is of great interest as it fundamentally changes the heavy and opaque spatial presence of conventional concrete.

The *Vertical Suzhou Living* design proposal (Fig. 6) employs translucent concrete throughout a residential tower. The atmosphere created by the material is used to mark the transition between public and private spaces as well as the transition between interior and exterior spaces.

4.5 Integration of construction and material properties: ICF at new heights

ICF (Insulated Concrete Forms) is a construction material as well as a system for construction offered by a variety of manufacturers. ICF consists of two sheets of EPS that are connected with plastic parts. In the construction process, the empty core of ICF modules is filled with regular reinforced concrete, with the EPS acting as lost formwork. ICF is increasingly used for its advantages in sustainable and economical construction, but has not

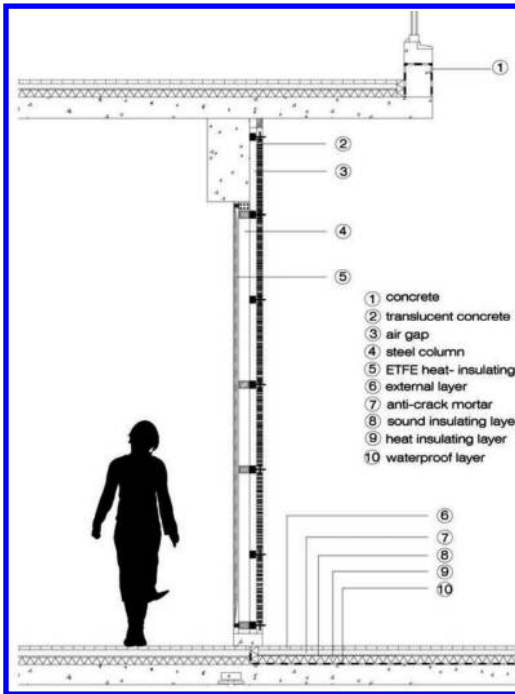


Figure 6. Translucent concrete employed within conventional concrete frame structure.



Figure 7. High-rise applications of ICF employing curved walls.

been adopted in China yet as building codes are not as stringent in insulation requirements as overseas codes. While ICF is not entirely new, the material has not yet been explored architecturally at the scale of tall buildings. *The Waves* design proposal (Fig. 7) illustrates how ICF could be employed in the context of urban Chinese residential buildings. The proposal focuses on how to extend the formal vocabulary of ICF through curved shapes, while maintaining the other advantages of the material. Integrating curves into ICF construction will make

the material more attractive to architects by demonstrating that the material is capable of accommodating both technical and aesthetic design aims.

4.6 Engaging the building frame architecturally

In Chinese architectural design, structures are typically designed somewhat separately from building form and spatial experiences. As most buildings consist of highly repetitive column frame structures, architects simply accommodate the regular column patterns into their floor plan layouts. Overseas examples (Baumberger 2011) however show that the integration of structural and architectural design can produce more integrated, more sustainable and more interesting buildings and architectural spaces.

In the *Creative Community Tower* proposal (Fig. 8), a building is differentiated into functional zones through its structure, which also acts as architectural device. The proposed tower provides a productive environment for creative professionals to live in, work and share ideas within the community and with the public. A variety of functional zones are designed to cater for the needs of creative individuals as well as companies. The concrete building structure is designed to provide a suitable framework and host for these various functional zones while also performing structurally. The entire structure is exposed and more irregular than conventional concrete structures, although it follows an overall pattern that is determined through the distribution of functions on each floor. Within one floor plan, the building structure changes from closed walls in areas where privacy should be protected, to shorter walls enabling increased spatial exchange to columns that enable a flexible floor plan. While this approach renders the building very specific, it also



Figure 8. Exposed concrete structure as a spatial design tool.

contributes a unique character that may be sought by future users and inhabitants.

The *Casting Nature* design proposal also addressed the integration of structure and architecture within the scope of architectural design. In this case, the focus was on newly developed strategies to cast concrete in fabric formwork (Orr et al. 2011). Fabric formed concrete is studied primarily for its promise of sustainability and material savings, while maintaining construction quality. Fabric-formed concrete can produce concrete building elements that are optimized for performance, yet also interesting in terms of their spatial and aesthetic effects (ibid.). Inspired by the potential to integrate these two aspects, the *Casting Nature* design proposal (Fig. 9) developed a tower that features organically twisting columns. These columns are constructed through fabric-formed concrete and are exposed in the interior of apartments. The smooth and natural shapes of the resulting building interiors impart a new sense of nature to the residential apartments.

4.7 A new concrete vernacular

With the vast majority of buildings throughout China constructed in concrete, it would be appropriate to say that China is developing its own specific type of concrete vernacular. At this point in time, these buildings are planned almost exclusively at the large scale, without taking the human scale and living conditions into account other than in terms of economy and statistics. Several proposals designed for the *Concrete Futures* project extended this quite limited scope by giving particular attention to the experiences of inhabitants. These aspects of sustainable growth are likely to shape China's urban future, and deserve as much attention as the more technical innovations discussed in previous sections.



Figure 9. Curved tree-like columns creating an organic overall impression.

In the *Collage Tower* proposal (Fig. 10), local spatial patterns found in the traditional Shanghai Lilong housing are adopted to create more varied living spaces and rich communities within a high-rise tower. In the *New Shanghai Habitat* (Fig. 11), the linear Lilong patterns typical for Shanghai were transformed into a new type of high-rise structure.

In addition, particular attention was given to the interiors of the concrete building. A variety of surface finishes on the exposed concrete structure serves to distinguish public and private zones within the building, much of which is shared among different groups of inhabitants. The variety in surface finishes marks inhabitation zones of different groups in a subtle manner. It is also reminiscent of the sensual richness found in the disappearing



Figure 10. Creating an enhanced sense of community at a variety of scales for a residential tower in Shanghai.

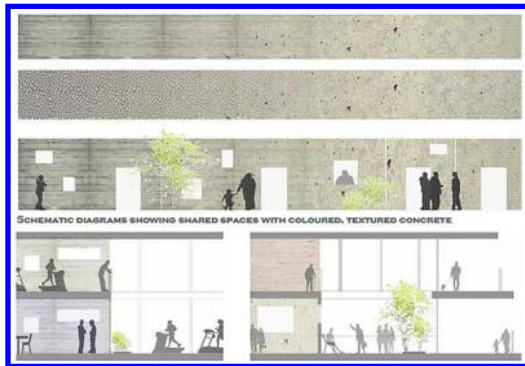


Figure 11. Using a variety of concrete surface treatments to express different degrees of public and private.



Figure 12. Simple yet elegant exposed concrete in urban community housing for rural migrants to the cities.

traditional Lilong housing that features a rich mix of materials.

The *Housing Diversity* design proposal (Fig. 12) addresses the challenges rural migrants to the cities face, and proposes a building that allows migrants to gradually grow into, and adopt the skills necessary for, their new urban lifestyles. The building is designed such that both local residents (mostly elderly) and migrants may live in different yet intermingling communities. The proposal makes use of exposed concrete surfaces to provide a simple yet elegant backdrop for urban dwellers from a more basic village living background. It also accommodates families of migrants, which is important when addressing the challenge and consequences of urban migration in China.

5 SUMMARY AND OUTLOOK

This paper presents a variety of effects of concrete material innovations on architectural design. The design research study reported on here adopts a range of approaches to sustainability, some focusing more on technical aspects, some focusing on wider social issues. Both approaches consider long-term perspectives, but may have different short-term priorities. Concentrating on the potentials of the material concrete in the future of Chinese urban architecture and construction, the resulting design proposals offer a range of visions for future concrete architecture, specifically addressing the medium-rise tower.

Aspects explored by the design proposals presented above include several effects of and architectural opportunities arising from new modes of designing with concrete. Among them are: sculptural concrete, experiential concrete, a new thinness of concrete structures, new transparency in concrete, curved shapes for medium-rise towers constructed with ICF, considering concrete structures as part of the architectural design, and finally, the

conception of a new concrete vernacular for Chinese urban residential spaces.

Adopting a design research method, the study produced design proposals as well as reflections and evaluations of these proposals. Design proposals were intended to raise challenging questions that require further investigation, both in terms of design and science. Among these questions are: How can dense yet high-quality living spaces be provided for economically disadvantaged urban dwellers? This is highly relevant as China's urbanization is to increase further in the near future. In terms of the material concrete, questions remain regarding how to engage architecturally with a material that is used throughout China, but is typically hidden? How can concrete return to the Chinese architectural stage? Both questions aim at more sustainable futures, and seek to further an increased awareness—and appreciation of—new possibilities in concrete architecture.

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Finite element analysis on fire resistance behaviors of PEC columns with restrained thermal elongation

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ABSTRACT: Partially Encased Concrete column (PEC) is a combination column whose bearing capacity and local stability can be both increased as hollow steel sections filled with concrete, however, due to partial steel exposure to air, PEC columns easily buckle and have a decreased bearing capacity under high temperatures. To study the fire resistance behaviors of PEC columns, a nonlinear finite element analytical model of restrained partially encased concrete columns under standard 4-side fire exposure is developed. Through comparing with simulated values and test values, the accuracy of the proposed analytical model has been verified, and the fire resistance behaviors of PEC columns have been analyzed using the finite element method. Meanwhile, the moment at column end effects on the fire resistance behaviors of restrained PEC columns has been analyzed.

1 INTRODUCTION

PEC column is a combination column with section steel and concrete whose hollow steel sections are partially encased with concrete. Composite steel and concrete structures are very popular because bearing capacity and local stability can be both increased as hollow steel sections filled with concrete. Compared to steel structures, PEC columns are the efficient structures to increase the usable floor areas. Meanwhile, the partially encased concrete columns also can be used in forcing and rebuilding steel columns, so it can be observed that these combination columns have the great application prospect.

Due to partial steel exposure to air, PEC columns are easy to buckling and decreasing bearing capacity under high temperature, the fire resistance behaviors of PEC columns have aroused more and more concern. Wainman et al. (1994) studied PEC columns fire resistance behaviors by using experimental method, and the research results of PEC columns offer reference for European Standard. Eurocode4-1-2 and DIN4102-4 gave the fire resistance design contents which expressed as the diagram form. Ali et al. (2004) experimentally studied on the fire resistance behaviors of 99 high and normal strength concrete elements. The parametric study investigated the effect of restraint degree, loading level and heating rates on the performance of concrete columns subjected to elevated temperatures with a special attention directed to explosive spalling. Correia et al. (2011) experimentally studied the behaviors of

composite columns made of partially encased steel sections subjected to fire, and the parameters studied were the load level, the axial and rotational restraint ratios and the slenderness of the column, and they found that for low loading levels the stiffness of the surrounding structure had a major influence on the behavior of the column subjected to fire. Increasing the stiffness of the surrounding structure led to reductions in the critical times. The same behavior was not observed for the high loading levels. Meanwhile, the fire resistance behaviors of restrained columns are getting more and more attention. Benmarce et al. (2002) experimentally studied on fire resistance of 12 axial constraint high-strength concrete columns. Huang et al. (2007) performed an experimental study of the axial restraint effecting on fire resistance of four unprotected encased I-section composite columns, and the real-sized columns were subjected to concentric axial force at a load ratio of 0.7 at normal ambient temperature, and different degrees of axial restraint were investigated. Huang et al. (2008) numerically studied on the fire resistance of embedded I-section composite columns designed according to EC4 Pt.1.2. These columns were subjected to axial compression forces and four-face uniform heating, based on numerical analyses, they found that under high loading levels, columns with small cross-sections failed to meet the fire resistance as suggested by EC4 Pt.1.2. Wu Bo & Li Yihai (2009) performed an experimental studied on fire performance of eight axially restrained Reinforced Concrete (RC) columns under a combination

of two different load ratios and two different axial restraint ratios. The eight RC columns were all concentrically loaded and subjected to ISO834 standard fire on all sides. As the results of the experiments had shown, the stiffness of the axial restraint and loading level play an important role in the fire behaviors of both high-strength concrete and normal concrete columns. Wu Bo et al. (2010) studied on the performance of 12 axially restrained RC columns (four for each type of cross section, i.e., +, -, T- and L-shaped cross section) under fire conditions, and they found that restrained RC columns behaved quite differently to isolated ones. No typical 'runaway' failure of isolated RC columns was observed in the fire test.

It can be known that studies on the fire resistance behaviors of PEC columns are imperfect, the research results only draw a few qualitative conclusions, it is very necessary to further study on fire behaviors of PEC columns for better application. In this paper, we analyze the moment effecting on fire resistance performance of restrained PEC columns, and compare to test results, the correction of finite element analytical model has been verified.

2 FINITE ELEMENT MODEL

The finite element analytical model of restrained PEC columns under standard 4-side fire exposure has been developed, and using the proposed model to calculate the temperature fields based on transient heat transfer, finally, analyze the mechanical properties of PEC columns.

1. Thermal performances of materials. The thermal conductivity coefficients and specific heat of materials proposed by EC4 (1994) have been used to calculate the temperature field. The unit weight of concrete is considered as constant quantity, which the value is 2300 kg/m^3 , and the unit weight value of steel is 7850 kg/m^3 . The plastic damage model and the isotropic elastoplastic model are chosen for concrete and steel to analyze the mechanical properties, respectively. The expressions proposed by EC4(1994) are used as the stress-strain relationship of concrete and steel under high temperature.
2. Element type and mesh generation. Eight-node solid element (DC3D8) and two-node truss element (DC1D2) have been respectively used for concrete & steel and reinforced bars when doing the temperature field analysis. During the mechanical performance analysis, Eight-node reduced integrated solid element (C3D8R) and two-node truss element (T3D2) have been respectively used for concrete & steel and reinforced bars. Mesh generation and node numbering should be same during these two stages, and we used the structured meshing technology in mesh generation.

3. Interface processing. We assume that the interfaces of concrete and steel have no temperature change during 3D temperature conductivity analysis, and there is no bond slip between concrete and steel during the mechanical performance analysis.
4. Boundary condition. The temperature of PEC columns and environment are also 20°C before heating. The rigid connection is adopted between column and foundation, and the displacements of X & Y directions are zero during the temperature field analysis. The axial constraint can be carried out by using spring element.

3 EXPERIMENTAL VALIDATION

3.1 Text survey

We performed an experimental studied on fire performances of two axial restrained PEC columns under a combination of same load ratios and two different moments at the top of column. The ideal pin connection was adopted at the top of column and the rigid connection at the bottom. The axial constraint was carried out by steel beam. The magnitude, direction and action point of load at the top of column should be keeping constant.

The numbers of the specimens are labeled as PEC-K010-L50 and PEC-K010-L50-M. Where K010 means the axial restraint ratio is 0.1, and L50 means the load ratio is 0.5. The two columns are of a nominal height 4250 mm and of typical cross section as shown in Figure 1, in which a $250 \times 250 \text{ mm}$ square section encased a $\text{HW}250 \times 250 \times 9 \times 14$ section. Hollow steel sections are partially encased concrete, and the four longitudinal direction reinforcing bars HRB335 Φ 16 are embedded in the concrete, whose cover thickness is 40 mm. The cross direction reinforcing bars HPB235 Φ 8@100/150 are weld on the steel flanges whose weld length is 70 mm, and the distance of these bars at the ends range within 500 mm is 100 mm. Steel end plates are welded at both ends of specimens.

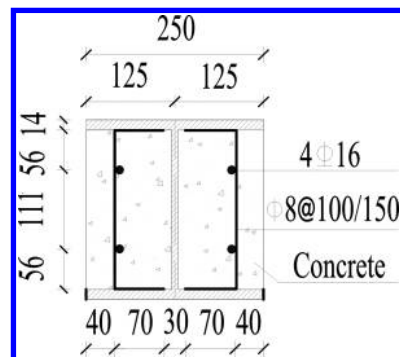


Figure 1. Cross-section of PEC column.

Table 1. Label number of specimens and text parameters.

Label number	Sectional dimension/mm	Steel ratio	Slenderness ratio	Load ratio	Moment
PEC-K010-L50	250 × 250	0.103	41.2	0.5	No
PEC-K010-L50-M	250 × 250	0.103	41.2	0.5	Yes

Table 2. Mechanics performance of steel.

Steel	<i>t</i> /mm	<i>f_y</i> /MPa	<i>f_u</i> /MPa	<i>E_s</i> /MPa	<i>ν</i>
Web	8.53	280	465	1.99 × 10 ¹¹	0.313
Flange	13.67	281.7	453	2.13 × 10 ¹¹	0.299

Table 3. Mechanics performance of concrete.

Text block	<i>f_{cu}</i> /MPa		$\overline{f_{cu}}$ /MPa		$\overline{f_{cu}}$ /MPa
Text 1	25.33	25.73	24.55	25.2	23.35
Text 2	18.82	22.65	23.09	21.5	

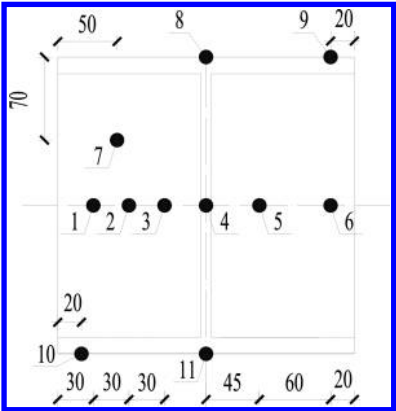


Figure 2. Location and numbering of thermocouples in PEC columns.

A summary of the material properties of the steel and concrete are shown in Table 2 and 3. Where *t* means thickness, *f_y* means yield strength, *f_u* means ultimate strength, *E_s* means elastic modulus, *ν* means Poisson ratio, *f_{cu}* means cubic compressive strength, $\overline{f_{cu}}$ means the average values of cubic compressive strength.

3.2 Temperature field

11 temperature measuring points were set at the middle of column to monitor the section temperature distribution, the distribution of the measuring points as shown in Figure 2. To ensure

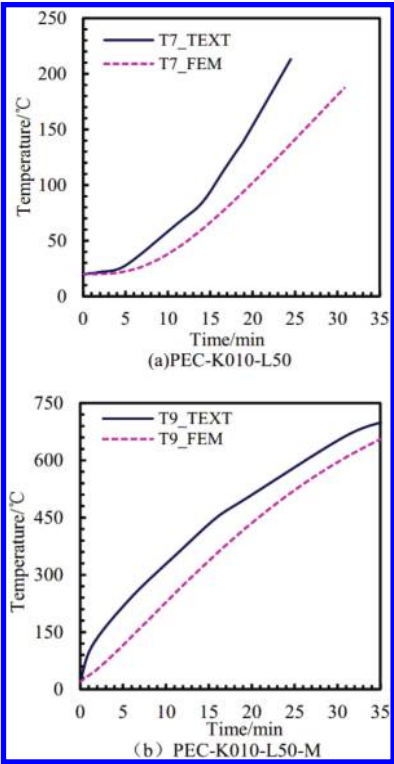


Figure 3. Development of T7 temperature of PEC-K010-L50 (a), Development of T9 temperature of PEC-K010-L50-M (b).

the thermocouples in the correction position, we embed the thermocouples after concrete hardening finished. K-thermocouple had been applied in this text, and it is of 3 mm diameter and of 1100°C limit text temperature.

The measured temperature values of T7 in PEC-K010-L50 and T9 in PEC-K010-L50-M are shown in Figure 3 (a, b). They show that the texting temperature values are general greater than simulated, while the measured values agree well with simulated values on the whole. Yet we find out the measured values at T7 developing faster than simulated values as shown in the Figure 3 (a), it may be the concrete cracking inducing to accelerate the temperature rising in the later stage of text. In general, the comparison can draw a conclusion that

the thermal performances of materials proposed by EC4 (1994) can simulate well the temperature field of composition structure.

3.3 Fire performance

The measured and simulated fire resistance of PEC-K010-L05 without applied moment are both 30 min, and the measured and simulated fire resistance of PEC-K010-L05-M applied moment are 35 min and 32 min, respectively. The results show that the measured values agree well with simulated values, and the comparison can draw a conclusion that the stress-strain relationship of concrete and steel under high temperature proposed by EC4 (1994) can simulate well the fire resistance of composition structures. Through above analysis, we find that fire resistance of the column applied moment greater than which the column have not, the main causes may be as follow: (1) The load of column applied moment is smaller when the load ratios are same, and the smaller applied load is useful to increase the fire resistance of columns; (2) The fire resistance of columns could be decrease because second-order effect produced by moment. The above factors coupling effects cause the result

which fire resistance of the column applied moment greater than which the column have not.

3.4 Axial deformation analysis

The axial deformation *versus* times of axial restrained PEC columns are shown in Figure 4 (a, b). The axial deformation appears thermal expansion firstly. With temperature increment the expansion deformation gradually reduced, and expansion deformation reduced more and more rapidly due to the materials degradation. The results show that the measured values agree well with simulated values.

4 CONCLUSIONS

1. The nonlinear finite element analytical model of restrained PEC columns under standard 4-side fire exposure is developed. Through comparing with simulated values and test values, the availability of proposed analytical model has been verified;
2. The fire resistance performance of column without applied moment is different from which applied moment. The fire resistance of the column applied moment is greater than which have not applied moment.

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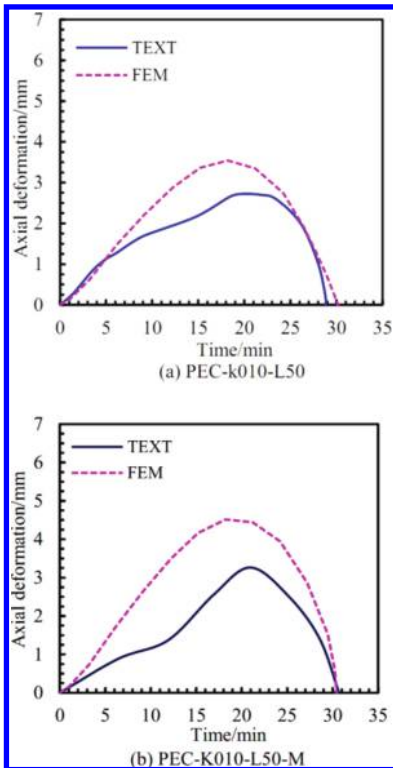


Figure 4. Axial deformation at the top of specimens.

A review on Agricultural Fibre Reinforced Concrete

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ABSTRACT: The construction industry has undergone rapid development during the last few decades. This significant construction growth has substantially increased the demand for construction materials. Among all materials used in the construction industry, concrete is the most significant. Billions of tonnes of naturally occurring minerals are mined for the production of concrete which has left a big gap in terms of sustainability of the construction industry. To overcome this drawback, the search for durable and sustainable construction materials is currently needed. The world of concrete with fibres has been in continuous research and also in use. However, manmade fibres though enhanced the properties of traditional concrete, are not sustainable and economical. This paper presents a brief review of fibres from agriculture wastes that have been in research and application as potential next generation construction materials. The micro-structure and properties of Agriculture Fibre Reinforced Concrete (AFRC) are described and some current possible applications are mentioned. Various methodology, emerging trends as well as variations in modification techniques for agricultural fibre modified concrete are observed. Several recently published articles and technical papers dealing with such fibre modified concrete are critically reviewed.

1 INTRODUCTION

Concrete is the most widely and commonly used building material in civil engineering all over the world. Its annual energy consumption exceeds that of all the other engineering materials. These days carbon emission from construction sites due to use of cement is a global issue. On the other hand, people's desire to live eco-environment is continuously increasing. A huge portion of the world's carbon emissions is coming from the concrete industry. Concrete industry is constantly looking for new ways to create lighter more ecological friendly products.

Concrete is strong in compression, however, as a very brittle material, has low strain capacity in tension and consequently low toughness. Savastano et al. (2009) in his investigations showed cracks develop whenever loads give rise to tensile stresses exceeding the tensile strength of concrete. Adding fibres to concrete matrix has been long recognised as a way to enhance the energy absorption capacity and crack resistance of the plain concrete. Concrete is typically reinforced with steel or synthetic fibres like carbon, glass and natural fibres. But not only steel is highly expensive, has high energy consumption and it comes from a non-renewable resource;

unprotected steel is also highly corrosive which might lead to infrastructure deterioration.

Natural fibres such as those derived from jutes, hemp, coir, corn, straw or pines and coconut are commonly used in concrete. In this review, we intend to study AFRC and its application to the ordinary concrete.

2 FIBRE REINFORCED CONCRETE

Fibre Reinforced Concrete (FRC) is Portland cement concrete reinforced with randomly distributed fibres. In FRC, short or discrete fibres are dispersed and distributed randomly in the concrete during mixing, and thus improve concrete properties in all directions. FRC is cement based composite material that has been developed in recent years. It has been successfully used in construction with its excellent flexural-tensile strength, resistance to spitting, impact resistance and excellent permeability and frost resistance. It is an effective way to increase toughness, shock resistance and resistance to plastic shrinkage cracking of the mortar. Fibre is a reinforcing material possessing various material or geometrical properties. They can be circular, triangular or flat

in cross-section. The fibre is often described by a convenient parameter called “aspect ratio”. The aspect ratio of the fibre is the ratio of its length to its diameter. The principle reason for incorporating fibres into a cement matrix is to increase the toughness and tensile strength and improve the cracking deformation characteristics of the resultant composite. For FRC to be a viable construction material, it must be able to compete economically with ordinary reinforcement.

3 HISTORY OF FIBRES IN CONCRETE

While fibres such as horse hair and straw were commonly used in ancient times, the first modern alternative was the use of asbestos fibres in the early 1900's. The concept of using fibres in order to reinforce matrices weak in tension is more than 4500 years old. Ancient civilizations used straw fibres in sun dried mud bricks in order to create a composite with increased toughness, i.e. a matrix with a better had to overcome the major deficiencies of concrete, which were the low tensile strength and the high brittleness (RTA 2006). A French engineer, named Joseph Lambot, in 1847 came out with the idea of adding con resistance to cracking and an improved post cracking response (Naaman 1985). Since Portland cement concrete started to be used widely as a construction material attempts were made to use fibres for arresting cracks. Engineers had to overcome the major deficiencies of concrete, which were the low tensile strength and the high brittleness.

The development of fibre reinforcement for concrete was very slow before 1960's. Until then there were some papers describing the basic concept of using fibres for reinforcement in concrete mixes but there was no application. Nevertheless, research on glass fibres had been conducted in USA, UK and Russia in early 1950's. Actually, in Russia glass fibres were not only under research but were also used in the construction industry.

However, this kind of fibres was found to be prone to alkaline attacks. In late 1950's Portland Cement Association started investigating fibre reinforcement (ACI 544.1 R 1996).

Although patents from the last century refer to the use of natural plant fibre as a component of building materials made from cements and plasters in natural fibre (mainly wood fibre) as a reinforcement for fibre cement has mainly taken place in the last 10–20 years. Unfortunately the large fibre cement manufacturing companies are the real custodians of the history of the fibre cement development and, from the obvious gap in the literature; they have released very little information about the use of natural fibres in cement.

Fibres from agricultural by-products or waste, such as wheat straw, soy stalk and rice husk, have received considerable attention due to their nature as a by-product of food. Wheat and rice straw, rice husk, rice husk ash, coir, corn stalk, banana fibre and pineapple leaf fibres are lingo-cellulosic biomass that have been successfully utilized to reinforce polymeric matrices. The classification of natural fibres have been compiled by Ni (1995) and illustrated in Figure 1.

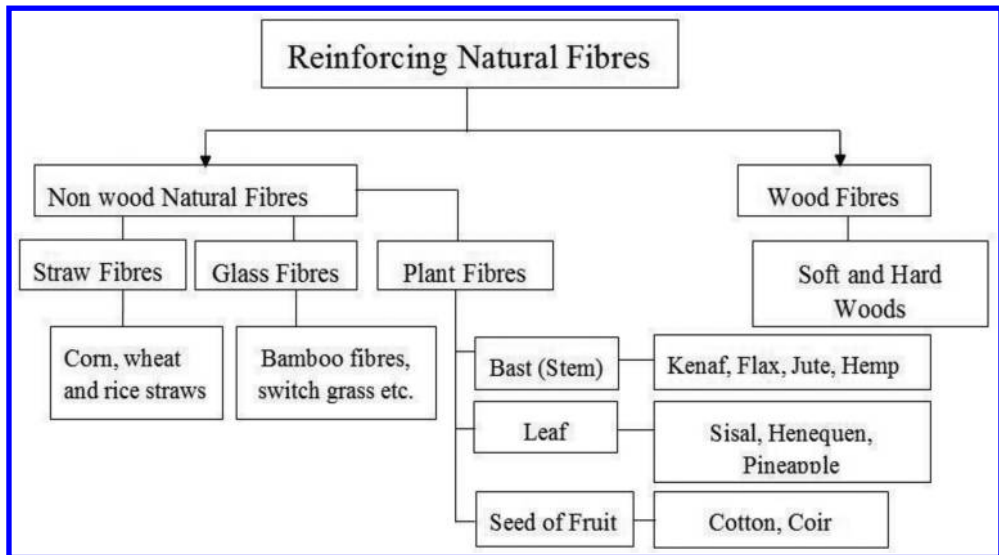


Figure 1. Classification of natural fibres (Ni 1995).

4 MICROSTRUCTURE OF AGRICULTURAL FIBRE

The components of natural fibres are cellulose, hemicellulose, lignin, pectin, waxes and water-soluble substances. The cellulose, hemicellulose and lignin are the basic components of natural fibres, governing the physical properties of the fibres.

The mechanical properties of natural fibres depend on its cellulose type because each type of cellulose has its own cell geometry which determine the chemical properties. Cellulose content is an important parameter, because in chemical pulping, the pulp yield corresponds to the cellulose content of the raw material.

Morsy (2011) mentioned that cellulose is the basic structural component of all plant fibres whereas hemicelluloses is made up of chains of sugars. They comprise a group of polysaccharides (excluding pectin) bonded together in relatively short, branching chains and remains associated with the cellulose after lignin has been removed.

Lignin is the compound that gives rigidity to the fibre. Natural fibres could not attain great heights or rigidity without lignin. Lignin is complex hydro-carbon polymer with both aliphatic and aromatic constituents.

5 NEED OF USE OF AGRICULTURAL FIBRES IN CONCRETE

Rapid population increment and economic expansion today have led to an increasing demand for high quality value-added forest products, such as pulp and paper products. Rice wastes are residue production in significant quantities on a global basis. According to the National Bureau of Statistics in China NBS (2014), the annual rice and wheat production for 2014 was estimated more than 206.5 and 126.2 million tonnes respectively. Among these large quantities of agricultural residues, only a minor portion of the residues is reserved as animal feed or household fuel. However, a huge quantity of the remaining straw and stems is not used as industrial raw materials and is burnt in the fields or on the side of the road. The pollution therefore is a serious problem by burning agricultural residues in this area. The use of agricultural residues in pulping and papermaking has, therefore, many advantages including reducing the need for disposal or environmental impact. (Xiao et al. 2001).

6 REVIEW OF LITERATURES

There have been very few researches on mechanical use of agriculture straw fibres and few

researches are focused on chemical properties and addition of chemical for acetylation and alkali-zation treatment methods. It is understood that the chemical transformation of raw straw fibres to concrete suitable fibres is a major challenge. Table 1 shows the chemical composition of some common natural fibres compiled and tabulated below.

Table 2 summarizes the properties of some common agricultural fibres.

6.1 Microstructure of fibres

Xiao et al. (2001) investigated the structural features and physio-chemical and thermal properties of the lignin and hemicellulosic preparations, isolated with 1 M Sodium Hydroxide (NaOH) at 30°C for 18 hours from dewaxed maize stems, rye straw, and rice straw, using a combination of several destructive and non-destructive techniques as mentioned earlier. The chemical composition and thermal stability of their residues, mainly celluloses, were also reported.

Treatment of dewaxed maize stems, rye straw, and rice straw resulted in dissolution of 78.0, 68.8, and 82.1% of the original lignin, and 72.1, 72.6, and 84.6% of the original hemicelluloses, respectively. The thermal analysis of the polymers showed that hemicelluloses degraded in first place, while lignin showed less degradation, and therefore, its structure was more stable. Cellulose, on the other hand, showed an important degradation process, mainly between 250 and 330°C, and its thermal stability was lower than that of lignin, but higher than that of hemicelluloses.

Table 1. Chemical composition of some common natural fibres in percentage by weight.

Material	Cellulose	Hemicellulose	Lignin
Rice husk	45.0	19.0	19.5
Barley husk	47.0–48.0	24.0–38.0	16.0–19.0
Rice straw	38.3	28.0	14.9
Wheat straw	34.5	21.3	17.5
Oat straw	44.0–53.0	27.0–38.0	16.0–19.0

Table 2. Properties of natural fibres.

Fibres	Specific gravity	Tensile strength (MPa)
Sisal	1.370	347–378
Coconut	1.177	95–118
Bamboo	1.158	73–505
Jute	1.020–1.040	250–350
Rice straw	–	48

6.2 Treatment of raw fibres

Fahim et al. (2012) published the potential usage of virgin Low Density Polyethylene (LDPE) reinforced with different concentrations (2%, 5% and 6% by weight) of treated rice straw with different lengths (2 mm, 4 mm and 6 mm). The use of rice straw as reinforcement in composites with 20% and 30% by weight was successfully prepared by extrusion and compression moulding. The two treatment methods used for rice straw were alkali and acidic treatments. The removal of impurities and waxy substances from fibre surface avoided creation of rougher topography after treatment and improved the quality of fibre. Also content of hemicellulose and lignin decreased so increase effectiveness of fibre due to dispersing of fibre in matrix was seen. The reinforcing material was embedded in the matrix material to enhance tensile and flexural behaviours of the synthesized composite. The result of investigating these two mechanical properties, using statistical analysis and design of experiments, showed an enhancement in the mechanical properties of the virgin polymer composite compared to the virgin polymer. The flexural stress of the composite increased three times the virgin flexural stress, while the tensile stress increased eight times the original tensile stress.

Pan et al. (2009) studied the effects of alkalization, acetylation, and Maleic Anhydride grafted Polypropylene (MAPP) treatments on the thermal and chemical properties of the wheat straw fibre. Investigation was conducted using thermogravimetric analysis, infrared spectrophotometer, X-ray diffraction and scanning electric microscopy techniques.

Wheat straw fibre-polypropylene (PP) composites were prepared to investigate the effects of wheat straw fibre content (10, 20, 30, 40, and 50 wt.%), fibre size (9, 28, and 35 mesh), and Maleic Anhydride grafted Polypropylene (MAPP) concentration (1, 2, 5, and 10 wt.%) on the static and dynamic mechanical properties of the wheat straw fibre-PP composites in this study. It was found that by adding wheat straw fibre into PP matrix, the tensile modulus and strength of the composites increased gradually, whereas the tensile elongation at break decreased because of the stiffness of the wheat straw fibre. With increasing MAPP concentration, the PP composites showed an increase in tensile strength, the highest tensile strength occurred with the concentration of MAPP up to 10% weight.

6.3 Effect of agricultural fibres in concrete

Ramaswamy et al. (1983) studied the behaviour of concrete reinforced with jute, coir and bamboo

reinforced concrete. Fibre microstructure was studied at first. Compressive strength of various percentages of fibres was carried out and the results were presented. Impact tests, modulus of rupture and split cylinder tensile strength tests were also performed on the FRC.

Deformational properties such as stress strain behaviour along with creep and shrinkage on sustained compressive load was observed. The results of various experiments showed vegetable fibres can be used with advantage in concrete. By adopting a suitable method of proportioning of the concrete mix and by gradual, alternate incorporation of fibres and water, homogeneous mixes can be obtained for small amounts of fibre additions. Improvement in impact strength of over 25% and increased ductility under static loading and considerably lower shrinkage characteristics of the order of 50% to 70% compared to those of plain concrete, are noted as positive features of vegetable fibre additions. No serious adverse effects of fibre additions have been noted.

Savastano et al. (2009) studied the performance of the vegetable fibres relevant to civil construction applications. The availability of the fibres and their extraction are closely related to the manufacturing and processing of the fibrous raw materials for different uses. The paper also contains an example of the application of vegetable fibres as reinforcement in cement-based composites for building and infrastructure construction.

In this paper, he compiled the study of mechanical properties of vegetable fibres such as jute, coir, sisal, banana and bamboo. For the case study, sisal fibre was used to prepare cement composite pads reinforced with 8% of fibres. Three point bending test was performed and modulus of rupture and modulus of elasticity was obtained for different conditions of exposure and age of the samples. The results showed significant benefits of such fibre reinforced composites.

Vajje & Krishnamurthy (2013) studied the addition of natural fibres viz. Jute, Sisal, Hemp and banana in concrete. Strength tests such as cylinder compressive strength and workability by slump tests were performed.

It was seen that slump was decreasing with the addition of fibres. More the fibre-cement ratio more is the decrease in slump due to absorbency of water by fibres. Hence the use of proper super plasticizer which does not affect other properties except workability is recommended for higher fibre-cement ratios from this result. The addition of fibres increased compressive strength with 0.5% fibre-cement ratio and little increase for 1% of fibre-cement ratio compared to plain concrete. But at 1.5% of fibre-cement ratio, though plasticizer was added, the compressive strength decreased

compared to plain concrete. Also as the percentage of fibre increased, sudden brittle failure of specimen was seen.

Yalley & Kwan (2013) showed in their research experimental studies on the use of coconut fibre as enhancement of concrete. The addition of coconut-fibres significantly improved many of the engineering properties of the concrete, notably torsion, toughness and tensile strength. The ability to resist cracking and spalling were also enhanced. However, the addition of fibres adversely affected the compressive strength. When coconut fibre was added to plain concrete, the torsional strength increased (by up to about 25%) as well as the energy-absorbing capacity, but there is an optimum weight fraction (0.5% by weight of cement) beyond which the torsional strength started to decrease again. Similar results were also obtained for different fibre aspect ratios, where again results showed there was an optimum aspect ratio (125). An increase in fibre weight fraction provided a consistent increase in ductility up to the optimum content (0.5%) with corresponding fibre aspect ratio of 125. Overall the study has demonstrated that addition of coconut fibre to concrete leads to improvement of the toughness torsion and the tensile stress of the concrete.

Merta & Tschegg (2013), studied the fracture energy of concrete reinforced with natural fibres of hemp, elephant grass, and wheat straw. Chopped fibres of hemp, wheat straw, and elephant grass with 40 mm length were added to the concrete matrix as fibre reinforcement. The fibres content was 4.5 kg/m^3 which resulted in a fibre percentage of 0.19% by weight. The fibres were used as they come from nature without any kind of preparation ensuring in such a way a low cost building material. They tested it uniaxially with the Wedge Splitting Test (WST) method.

The addition of fibres was found to improve the fracture toughness of plain concrete. The most distinctive increase in the fracture energy has been observed by hemp reinforced concrete (up to 70%). The reason is fibre's high tensile strength and of the fibre's fineness. Straw and elephant grass fibres increased the fracture energy of concrete solely up to 2% and 5% respectively. The reason for that is believed to be the combination of high surface roughness of the straw fibres (resulting in good bond with the concrete matrix) and of low tensile strength of the fibres. By elephant grass FRC the low surface roughness of the fibres results in pure bond with the concrete matrix and fibre pull-out failure without any stress transfer. Also, it was noted that the splitting tensile strength of the fibre reinforced specimens was up to 4%, 7%, and 8% lower compared to unreinforced concrete

specimens for hemp, straw, and elephant grass fibres respectively.

6.4 *Durability and thermal properties with concrete*

Bouasker et al. (2014) in his research used four types of straw: three wheat straws and a barley straw. In the study, the morphology and the porosity of the different straw aggregates was studied by SEM in order to understand their effects on the capillary structure and the hygroscopic behaviour. The physical properties such as sorption-desorption isotherms, water absorption coefficient, pH, electrical conductivity and thermo-gravimetric analysis were also studied. The sorption-desorption isotherms showed that the straw is an excellent hydric regulator that improves hygrothermal comfort of the building. The straw present a high water absorption coefficient compared with that of other vegetable fibres. At 20°C , straw absorbed an average weight of water three times larger than its own weight. This value reached four times for barley straw. This fact makes straw more sensitive to water.

In conclusion, it was found that natural straws had a very low bulk density, a high water absorption capacity and an excellent hydric regulator. The introduction of the straw in the water tends to make the environment more basic; this observation can slow carbonation of the binder matrix in the presence of the straw. However, the skin of the straw particles seems to be smooth. Unfortunately, this property has an adverse effect on the mechanical strength of final materials based straw fibre.

Chabannes et al. (2014) explored the use of raw rice husk as natural aggregate in a lightweight insulating concrete. Specimens made out of whole rice husks and a lime-based binder was manufactured by mixing and mechanical tamping. Both thermal and mechanical properties of the final concrete materials were studied in comparison to hemp concrete designed with the same process. The effects of an outdoor exposure of the specimens on the mechanical strength are discussed and the binding matrix mineral phases considered. Thermal measurements showed that rice husk concrete can compete with hemp concrete in terms of thermal insulation with a dry thermal conductivity ranging from about $0.10 \text{ Wm}^{-1} \text{ K}^{-1}$ to $0.14 \text{ Wm}^{-1} \text{ K}^{-1}$ depending on the mix proportioning. Compression test results on a defined mixture intended to be used as a filling material in a wall timber frame highlighted lower mechanical performances for rice husk concrete with an average compressive strength of $0.33 \pm 0.03 \text{ MPa}$ at 60 days compared to $0.48 \pm 0.02 \text{ MPa}$ for hemp concrete.

Akturk et al. (2015) studied the use of raw rice husk in high strength concrete to prevent spalling, improve mechanical, physical and thermo-physical properties of concrete after high temperature treatment. The paper investigated the usability of RRH, an agro waste and sustainable material, as an alternative to Polypropylene (PP) fibres in high strength concrete exposed to high temperature. Therefore, a series of tests was performed to determine and compare the performance of HSC mixes containing different amounts of PP and rice husk.

The PP fibers were used at three different dosages, i.e., 0.2, 0.4, and 0.5% and the proportions of raw rice husk were 0.5, 1.5, and 3% by the weight of the cement. Cylinders and prisms were cast to test the properties of concrete. For mechanical properties, compressive and splitting tensile strength was performed whereas for physical and thermo-physical properties, ultrasonic pulse velocity tests, water absorption tests and void content was determined. Thermal conductivity of concrete was determined using a guarded hot plate apparatus. The test results showed when rice husk was used at 3% of cement by weight it can prevent explosive spalling and can be used in alternative of polypropylene. Due to lower thermal conductivity of rice husk mix, the mechanical properties of the concrete slightly decreased. Up to 300°C, the CO and CO₂ emissions of rice husk mix were similar to those of polypropylene mix. The increase in the temperature resulted in a significant decrease in the mechanical properties of all of the mixes. Also, compared with the polypropylene mixes, the addition of rice husk slightly decreased the mechanical properties of the concrete.

7 CONCLUSIONS

The available literature data is mostly related to the research on vegetable and natural fibres. Further investigations are needed in order to clarify several aspects that current knowledge does not represent. As an example only recently has the agriculture fibres got the proper attention. Since the main reason for fibre degradation relates to alkaline degradation, much more research is needed about the chemical interactions between the cement matrix and the agricultural fibres. The right treatments to improve fibre and cement matrix compatibility are still to be found. The same could be said about the variation on fibre properties thus controlling quality methods are needed in order to ensure minimal variations on the properties of agricultural straw fibres. Durability related issues also deserve further investigations. Concrete structures reinforced with

rice and wheat fibres are a promising field towards a more sustainable construction and applications.

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Flexural test of Tee steel PEC beam

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ABSTRACT: To investigate vertical load-bearing capacity of Tee steel PEC beams, test results of two simply supported Tee steel PEC beams under vertical loads are presented. The procedure and method of test was introduced. The carrying capacity and failure mechanism, deformation capacity, mid-span sectional strain distributions were analyzed. The result indicates that the Tee steel PEC beam has good carrying capacity and deformation ability under vertical loads. Tee steel PEC beam specimens with full shear connection exhibit cross sectional strains that meets the flat section assumption. To avoid brittle failure of Tee steel PEC beams, the composite beam should have a larger steel ratio. It is rational that the plastic neutral axis is located in the flange of the concrete plate. Specimens with higher steel ratios have a higher bending strength and stiffness.

1 INTRODUCTION

In 1980s, Partially Encased Composite (PEC) beam (EN1994-1-1 2004) was proposed in Europe, it can be used for residential buildings, office buildings, and other high-rise frame structure. The fundamental form of PEC beam is to pour concrete between flanges of steel section so that it can enhance the capacity of the beam to resist fire and corrosion and lower the costs. Initially, when calculating the carrying capacity of the PEC beam, steel section was taken into account but without the consideration of concrete section. Researchers later discovered that this design not only improves the capacity of fire resistance of the members, but also improves the carrying capacity and stiffness of the composite member (Kindmann & Bergmann 1993). Since concrete between flanges of steel beam provide lateral support for steel plates that can help prevent local buckling, the local stability and overall stability of the beam can be ensured in normal use stage. Due to the participation of concrete portion, under the same span, PEC beam is lower than ordinary steel and concrete composite beam, thus reduced the height of the structure. Under the condition of meeting the requirements of the use function of buildings, reduce the cost of maintaining structure, decoration and pipeline facilities. In recent years, the research and engineering application of PEC structure are becoming more and more popular in foreign countries (EN1994-1-1 2004, Kindmann & Bergmann 1993, Piloto et al. 2013, Chen & Wang 2010, Konstantinos et al. 2013).

The “I” shape steel beam is generally used in the PEC beam. In order to reduce the amount of steel and make full use of the compressive capacity of concrete and the tensile capacity of steel, Young K. Ju and Sang-Dae Kim (Ju & Kim 2005, Ju et al. 2007) proposed Tee steel PEC beam, which consists of inverted Tee steel, stirrup, longitudinal bar, horizontal shear stud bolt, and precast concrete. Tee steel beam and precast concrete share the construction load. After pouring the concrete floor in the upper part, the Tee steel beam was finished and it was subjected to various loads in normal use stage. In the positive bending moment region of Tee steel PEC beam, the concrete bear the compression and the steel part withstand the tension, which is of great importance to fully utilize the material properties of steel and concrete therefore reduce the cost. Wrapped by concrete, the capacity of fire resistance and corrosion resistance of steel beam get improved significantly. The shear connectors were placed horizontally to prevent the longitudinal splitting cracks happening in the floor slab. Tee steel beam was embedded in concrete, the connection force between them was so strong that the number of shear connector can be minimized.

To investigate vertical load-bearing capacity of Tee steel PEC beam, two simply supported Tee steel PEC beams were tested under vertical loads. Procedure and method of test was introduced. The carrying capacity and failure mechanism, deformation capacity, mid-span section strain distribution were analyzed.

2 EXPERIMENTS

2.1 Test specimens

According to the code or standard, the geometry size and section of the Tee steel PEC beam were presented in Figure 1, in which the beam has a depth of 350 mm and a span of 3.6 m. The flange plate and web of the specimen SCB-1 have thicknesses of 10 mm and 8 mm, respectively (steel ratio is 3.73%). The flange plate and web of the specimen SCB-2 have thicknesses of 8 mm and 6 mm, respectively (steel ratio is 2.88%). Steel used in the specimen is Q235B. The concrete floor has a width of 800 mm and a thickness of 80 mm, which all meet the requirements of effective width and floor thickness made in «Technical specification for steel reinforced concrete composite structures» and «Code for design of concrete structure». The longitudinal bars and transverse bars of concrete slab are arranged according to structural requirements and the design strength of concrete is C30.

2.2 Material properties

In order to ensure the material properties of concrete, the standard 150 mm × 150 mm × 150 mm

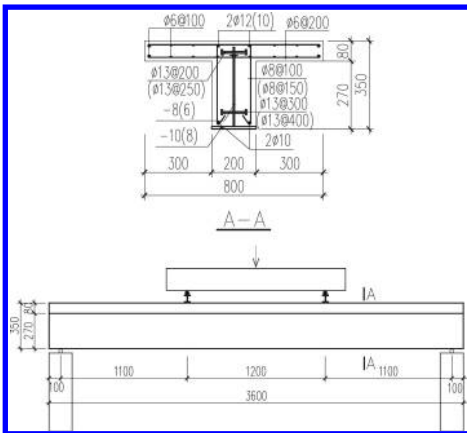


Figure 1. The geometry size of the specimen.

concrete cubes was prepared in the process of pouring composite beam, and was cured under the same temperature environment with the specimen in the room. The measured compressive strength of concrete specimens are in accordance with the relevant provisions of the national standard of «Standard for test method of mechanical properties on ordinary concrete» (GB/T 50081-2002), the compressive strength of concrete test cube was 29.05 MPa for SCB-1 and 31.12 MPa for SCB-2. The bars and steel plates were sampled according to the relevant provisions of the national standard of «Metallic materials—Tensile testing at ambient temperature» (GB/T 228-2002) and «Steel and steel products—Location and preparation of test pieces for mechanical testing» (GB/T 2975-1998). The diameter of the steel bar specimens was 10 mm and 12 mm, respectively, and the thickness of the steel plate was 6 mm, 8 mm and 10 mm respectively. The results of the test of steel materials are shown in Table 1.

2.3 Test setup of specimen

Tee steel PEC beam specimen was simply supported and fixed hinge support was achieved by welding steel bar on the steel plate, sliding hinge support was used by using rolling steel bars. The test was carried out under the counter force frame, 100 tons of hydraulic jacks were used to load the specimen through the distribution of two points, in order to pass the force uniformly to the composite beam and concrete flange plate, two lateral loading beam with strong stiffness were placed under the mat. The loading method was shown in Figure 1 and the test loading photo was shown in Figure 2.

2.4 Layout of measurement points

The strain gauges were arranged at the mid-span section of the bottom flange and the web of the steel beam and the strain rosette was arranged at the web of the shear span. To monitor the strain of the stirrup, two strain gauges were placed at the stirrup which was in the same section with the strain rosette of the steel web. In order to measure

Table 1. Material properties of steel.

Steel	Thickness or diameter (mm)	Yield strength (f_y /MPa)	Ultimate strength (f_u /MPa)	Modulus of elasticity ($\times 10^5$ N/mm ²)	Elongation (δ /%)
Web	6	273.5	403.4	2.3	19.6
Flange web	8	250.9	387.0	2.7	20.7
Flange	10	334.22	444.9	2.4	20.6
Bar	10	345.4	494.7	2.1	25.2
Bar	12	309.17	475.9	2.3	28.1



Figure 2. Test setup of specimen.

the variations of deflection during the loading process, a dial indicator was set at both ends of the beam and three displacement meters were set at the mid-span and loading points.

3 TEST PROCESS AND FAILURE MECHANISM

3.1 Test procedure description

In the early loading stage, the SCB-1 specimen was in the elastic state, and the concrete was intact, the load and deflection curve was linear. As the load approached to 100 kN ($P/P_u = 7.1\%$), the first vertical crack was found near the loading point of the specimen. As the load approached to 180 kN ($P/P_u = 11.4\%$), a vertical crack was found at the shear span near the loading point. As the load approached to 280 kN ($P/P_u = 40\%$), a longitudinal crack was found at the top of the plates of the pure bending region, at the same time, a longitudinal crack was also found at the concrete flange of left shear span. When loaded to 400 kN ($P/P_u = 57.14\%$), the cracks in the side of the beam become significantly wider. When loaded to 460 kN ($P/P_u = 65.7\%$), diagonal cracks of shear span become wider and almost all cracks formed at this load level. When loaded to 480 kN ($P/P_u = 68.5\%$), the width of diagonal cracks of shear span has reached 0.3 mm. When loaded to 500 kN ($P/P_u = 71.42\%$), the deflection of the mid-span section increased significantly, and the slope of the displacement load curve became smaller. When loaded to 653 kN ($P/P_u = 97.14\%$), the maximum crack width has reached 1.93 mm, as shown in Figure 3. When loaded to 674 kN ($P/P_u = 100\%$), displacement meter readings increased rapidly, the top of the concrete flange near the loading point of the specimen was crushed which led to the end of testing. The temperature bars set under the surface of this flange near the plate edge was also bent and protruded, as shown in Figure 4. Figure 5 is the overall deformation of the SCB-1 specimen and the distribution of the cracks in the half span beam as shown in Figure 6.

The first vertical crack was found near the supports of the SCB-2 specimen at load level of



Figure 3. SCB-1 crack.



Figure 4. Crushing of the concrete flange.



Figure 5. Overall deformation of SCB-1.

100 kN. As the load approached to 200 kN ($P/P_u = 44.7\%$), the strain of the bottom steel plate in the middle span is more than 1753 microstrain and the measured value of the steel yield strain reaching this value shows that steel has begun to yield.



Figure 6. Crack distribution of SCB-1.



Figure 8. Crack distribution of SCB-2.



Figure 7. Overall deformation of SCB-2.

As the load approached to 380 kN ($P/P_u = 85\%$), the crack width in pure bending section near the support reached 0.31 mm. As the load approached to 430 kN ($P/P_u = 96.20\%$), the crack width in shear span part near the support reached 1.23 mm and the deflection of the middle span reached 21 mm. As the load approached to 447 kN ($P/P_u = 100\%$), the maximum deflection reached 44.84 mm, at this time in the lower flange of the steel beam in pure bending section near beam loading was pulled down. At the same time, the upper part of the concrete flange plate is crushed, beam began to fracture. The overall deformation of the specimen is shown in Figure 7, the distribution of the beam cracks in the SCB-2 shown in Figure 8.

3.2 Failure mechanism analysis

The test specimens has experienced a longer period from the yield period to the failure period when compared to normal composite beam Two test

specimens are all failed in normal section bending, the ultimate load capacity was larger, the deflection was larger. Crack was firstly observed in the pure bending section near the loading point, it developed to the full width before being damaged. However, the flexural failure mode of the two specimens were different, the steel beam of the specimen SCB-1 was subject to tensile yielding, the damage of the composite beam was caused by the crushing damage of concrete in compressive zone and the damage has obvious signs. The steel beam in pure bending section of SCB-2 was strained to damage first, then, the concrete in compressive zone was crushed. Though the plastic rotation of the specimen occurred, the failure mode of the specimen was similar to the brittle failure of the reinforced concrete beam, which was more harmful. The bottom flange of the steel beam was exposed, and it was far from the neutral axis, which was subjected to the maximum tensile stress, and once the bottom flange was strained open, the specimen will break down immediately. In order to avoid the brittle failure, the bottom flange of the Tee steel PEC beam should take higher steel ratio, so as to ensure bottom flanges of the steel beam was not strained to damage before the concrete being crushed.

4 TEST RESULT ANALYSIS

4.1 Load deflection curves

From the test process and test results, the stress state of composite beam can be divided into elastic, elastic-plastic, and plastic stages. The load deflection curves of two specimens are shown in Figure 9.

1. Elastic stage

In the early stage of loading, deformation is small, composite beam had good performance,

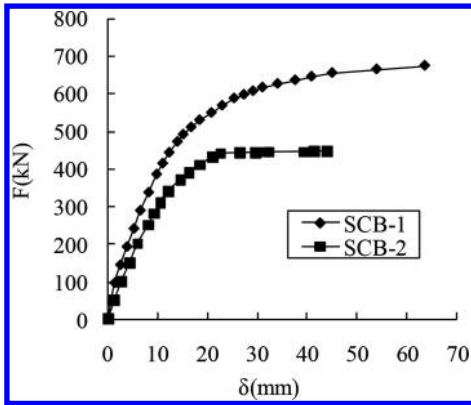


Figure 9. Load deflection curves.

mid-span deflection curves of two specimens are linear, the strain and stress of steel and compressive concrete were linear, strain of beam flange plate and the adjacent tensile concrete were similar. As the load approached to 60 to 70 percentage of the ultimate load, specimen begun to yield, which is the critical point between the elastic state and elastic-plastic stage of the composite beam. The normal use limit state should be controlled in the stage.

2. Elastic-plastic stage

With the increase of the load, the deformation of composite beams was increased faster than the load, the load deflection curves started non-linear behavior, stiffness of the composite beam degraded due to the cracking of concrete portion, the strain of the cracked section increased and the interaction between the steel plate and the concrete has been destroyed. The concrete at the top of the plate was beginning to enter the plasticity stage, and the strain of the bottom flange of pure bending section of the Tee steel beam was about to reach the yielding value, steel beam entered the elastic-plastic stage. With the development of the plastic zone of Tee steel beam section, neutral axis moved upwards continuously. The effective working section of composite beam was decreased, and the stiffness of composite beam was obviously decreased. The nonlinear characteristic of the load and deflection curve was more and more obvious, and when the load reached about $0.95P_u$, the composite beam entered the plastic stage.

3. Plastic stage

With the increase of load, the deformation of pure bending section of the composite beam was increased significantly, the load deflection curves was close to a horizontal line. When the load reached the limit value, the majority of the steel

beam has entered the plastic deformation state due to the tensile yielding. Concrete in the compression zone was also shown to be plastic, the compressive strain reached the limit value, the damage of the Tee steel PEC beam caused by the crushed failure of the top concrete in mid-span section or tensile failure of the bottom steel plate.

4.2 Strain distribution along the vertical cross section of Tee steel webs

The strain variation along the centerline of the composite beam was monitored by 6 strain gauges in the vertical section of the cross section of Tee steel webs from top to bottom. Strain variation of different load levels are shown in Figure 10 and 11. For specimen SCB-1, the measured yield strain of the bottom plate is 1689 microstrain. Before the load reached 77.14% of the ultimate load, which means in the elastic stress stage, the strain was approximated to a linear distribution. It begun to enter the plasticity when the neutral axis was at the height of 225 mm. After that, the strain of the lower flange of the steel beam grew rapidly, the strain distribution was not linear, but the upwards movement of the neutral axis was not obviously. For specimen SCB-2, the measured yield strain of the bottom plate is 1753 microstrain, and the changes of the neutral axis was also not obvious in the elastic plastic stage. Near the

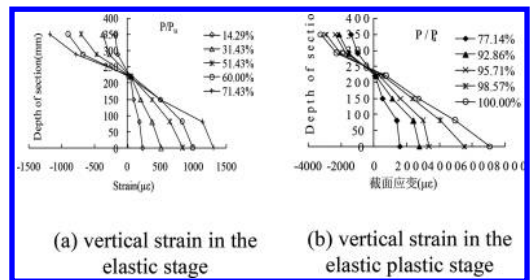


Figure 10. The distribution of the vertical section of SCB-1 specimen.

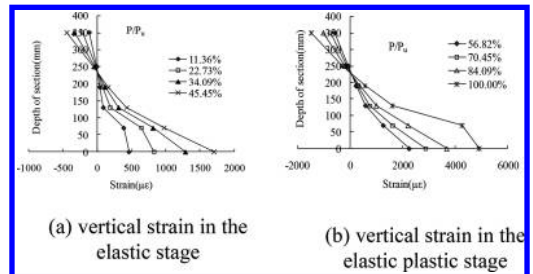


Figure 11. The distribution of the vertical section of SCB-2 specimen.

section height 235 mm the thickness of steel flange and web of specimen SCB-2 was small, the depth of compression zone was relatively small, and axial height was bigger than specimen SCB-1. In general, the distribution of strain was basically consistent with the flat section assumption.

4.3 Strain of web of steel beam and bar

The shear strain of the steel beam was measured by three-dimensional strain rosette and the strain of the stirrup was measured by strain gauge. From Figure 12, and Figure 13, the deformation of the stirrup was small at the early loading stage, and the stirrup began to play a role when the specimen entered the elastic-plastic stage. Before the steel yielding, concrete stress was small, so the stress of stirrups was also small. Compared to stirrup, the steel web bears more shear. During the entire process of loading, the shear strain of the web was bigger than that of the stirrup. At the moment the specimen was damaged, the two stirrup strain measurements of the specimen were smaller than the yielding strain.

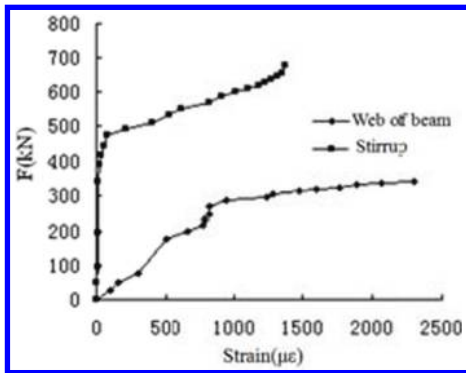


Figure 12. SCB-1 shear-strain curve.

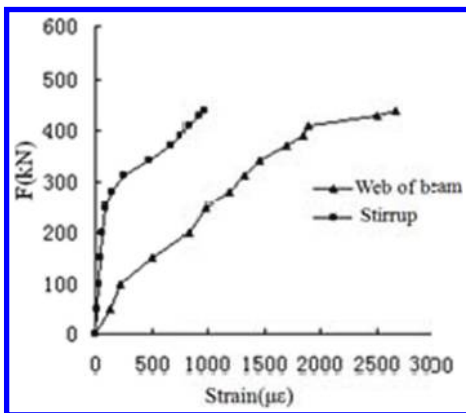


Figure 13. SCB-2 shear-strain curve.

5 CONCLUSIONS

1. Tee steel PEC beam has superior mechanical properties, good load carrying capacity, high stiffness, good deformation ability, and good ductility.
2. Distribution of cross sectional strain of T steel PEC beam specimen with full shear connection consistent with flat section assumption.
3. The Tee steel PEC beam should have larger steel ratio, which helps avoid the occurrence of brittle failure of the composite beam.
4. Plastic neutral axis resides in the flange of concrete beam, the tensile area of the Tee steel beam is large, which makes full use of materials.
5. Compared with specimen with lower steel ratio, the specimen with higher steel ratio has higher bending strength and stiffness.
6. The amount of slip of specimen is small, steel beam and concrete can work together under normal use conditions.

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Sustainable construction technology & BIM

The feasibility of retrofitting existing office buildings to combat energy consumption due to future climate change in three key regions of China

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ABSTRACT: As China goes through the present phase of rapid urbanisation, there is an urgent need to reduce the energy consumption per unit area in buildings in order to mitigate a sudden surge in total energy usage in buildings. At the same time, buildings are expected to last longer, so they will also need to account for effects of future climate change. Much of China's existing buildings are not considered energy-efficient, and retrofitting these buildings to perform in the future would be a much less energy-intensive strategy than demolishing them and building new replacements. This paper aims to investigate the feasibility of retrofitting existing office buildings in three different climate regions of China. These include the "Cold" region in the north, which includes Beijing; the "Hot Summer Mild Winter" region in the south, which includes Guangzhou, and two regions from the "Hot Summer Cold Winter", which includes cities such as Shanghai and Nanjing. Using data from the climate model, HadCM3, Test Reference Years are generated for the 2020s, 2050s and 2080s, for various IPCC future scenarios for these cities, and using computer simulation, the feasibility of retrofitting office buildings to perform in the face of climate change in these regions is investigated. For the "Cold" region, simulation results suggest that energy reductions of up to 30% with improvements to the building enclosure is possible by retrofitting current existing office buildings to the current Chinese National Standard for the region. Retrofitting in the "Hot Summer Mild Winter" region could reduce energy consumption by 45%, but the effects of retrofitting in the "Hot Summer Cold Winter" region are less profound. Requirements for combating possible increases in energy consumption from heating and cooling demands due to future climate change was also investigated, and it was found that with the exception of Shanghai, by simply improving U-Values of the building envelope to the new national standards, the effects of future climate change can be nullified up to the 2080s. Further improvements to having similar standards as developed countries such as England and Wales will have less significance and may not be able to nullify the effects of climate change from the newly retrofitted office buildings. However, this could be achieved by involving other energy-saving strategies and installing suitable energy-saving technologies for the regions.

1 INTRODUCTION

Since China opened its markets in 1978, there has been rapid growth and development in its economy. As a direct result, energy consumption has also increased significantly over this period. Similar to other parts of the world, the building sector accounts for roughly a third of the total energy consumption. Researchers such as Wu (2003), Zhou (2007) and Fridley et al. (2008) have suggested that for the whole of China, the building sector is responsible for about 27.5% of the national total energy consumption. Zhong (2007) have shown that as mass urbanization continues, and more buildings to be constructed in the next 20 years, this percentage could rise to 40%. As living standards improve, occupants will also demand higher levels of comfort, which will further increase heating and cooling demands. By the end of 2010,

the whole of China has over 43 billion square metres of constructed area, however, only 4–5% of which meet the national building energy standards, and the other 95–96% are classified as "intensive energy consumers" (Zheng 2007, Hong 2009).

In China, the lifespan of many buildings is short even by modern standards. The high turnover rate has resulted in an existing building stock that is fairly young. Zhu & Lin (2004) projected that by 2015, half of China's existing building stock will have been built after 2000. In 2010, Chen Huai, director of the policy research centre at the Ministry of Housing and Urban-Rural Development, stated that "Only those homes built after 1999 are likely to be preserved in the longer term" due to perceived safety and functionality deficits in older buildings (Qian 2010).

Effects are being made by the government to reduce the amount of energy consumed by

buildings in China with more and more stringent building codes for residential, commercial and office buildings. However, despite the comparatively high rates of construction and demolition, most buildings in China do not comply with the latest national building codes. There have been many studies on the improvement of standards and design strategies for new buildings in China and other nearby countries with similar climates, however, relatively few studies have been conducted on the situation for the retrofit of existing office buildings.

This paper aims to investigate the benefits of retrofitting existing office buildings to the standard of the current building code in three major cities (Beijing, Shanghai and Guangzhou) that are situated in three different climate zones in China, and see if retrofitting to the current standard can nullify effects of future climate change in these regions in terms of energy consumption until the end of the century. Data from the climate prediction model, HadCM3, are used in the analysis of the impacts of climate change in the 2020s, 2050s and 2080s.

1.1 Climate zones in China

There are five main climatic zones in China: “Severe Cold”, “Cold”, “Hot Summer and Cold Winter”, “Moderate”, and “Hot Summer and Warm Winter”, as shown in Figure 1. They have distinctive characteristics, and therefore separate national building codes. This paper will investigate the situation in the “Cold” zone by using weather data for Beijing, the “Hot Summer and Cold Winter” zone with the city of Shanghai, and the “Hot Summer and Warm Winter” zone with the city of Guangzhou.

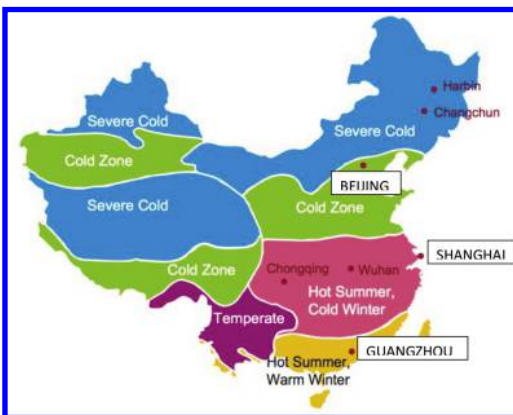


Figure 1. Climate zones in China and cities investigated.

2 CLIMATES IN BEIJING, SHANGHAI AND GUANGZHOU

The cities of Beijing, Shanghai and Guangzhou were selected to represent situations in three different climate zones in China. The buildings in each zone have very different heating and cooling needs. For example, Guangzhou in the “Hot Summer Warm Winter” zone has very little heating demand, as its winters are relatively mild. Shanghai is in the “Hot Summer, Cold Winter” zone, with days exceeding 40 deg C in the summer, as well as cold sub-zero days in winters. However, as Shanghai is located on the coast, it is generally slightly milder than locations in the same region which are more inland. Beijing also has hot summers, especially around August and early September, and winters are extremely harsh too. The changes in the next 100 years due to climate change for the three climate zones are also significantly different.

2.1 Current climates

Long running series of real observed data from weather stations in the three cities were not available for this study, so in-depth study of the characteristics of different weather parameters could not be conducted. However, data can be extracted from the existing Test Reference Years, from the Energy Plus program. Figure 2 shows the average monthly range and values for daily maximum and minimum temperatures for Beijing, Shanghai and Guangzhou.

2.2 Future climate change

The Hadley Centre model (HadCM3) (Gordon 2000) is used to provide future climate data for this study. HadCM3 is a global climate model developed at the Hadley Centre of the Met Office in the UK. It is a Coupled Atmosphere-Ocean General

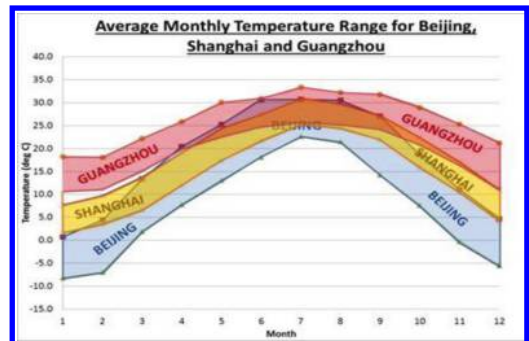


Figure 2. Current monthly average temperature range for Beijing, Shanghai and Guangzhou.

Circulation Model (AOGCM), in which the globe is divided into grid boxes, each measuring $2.50^{\circ} \times 3.75^{\circ}$. The gridboxes used in this paper are gridbox numbers 1952 (which encloses the area with latitude from 38.75°N to 41.25°N , and longitude from 114.375°E to 118.125°E , and includes the city of Beijing), 2337 (which encloses the area with latitude from 28.75°N to 31.25°N , and longitude from 118.125°E to 121.875°E , and includes the city of Shanghai), and 2623 (which encloses the area with latitude from 21.25°N to 23.75°N , and longitude from 110.625°E to 114.375°E , and includes the city of Guangzhou).

Figure 3 shows the extent of the three grid-boxes. Unlike weather data from typical weather years, HadCM3 only provide daily values for parameters such as maximum, minimum and average temperatures, humidity, wind speed and downward short-wave flux (solar radiation), based on 4 main future scenarios on carbon emissions, A1F, A2, B2 and B1 (Nakicenovic 2000). For example, the A2 scenario describes a very heterogeneous world where slow and fragmented economic growth. Changes in monthly temperature range is assumed, together with a continuation of population growth and continued increase in CO_2 emission into the twenty-first century (WMO 2001).

2.3 Compilation of future test reference years for beijing, shanghai and guangzhou

Due to the unavailability of extended periods of real historical weather data for the cities, future Test Reference Years (TRYs) were constructed using the

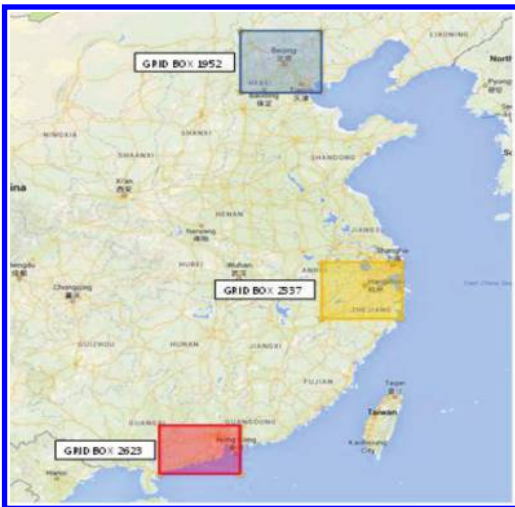


Figure 3. Geographically extents of the gridboxes 1952, 2337 and 2623.

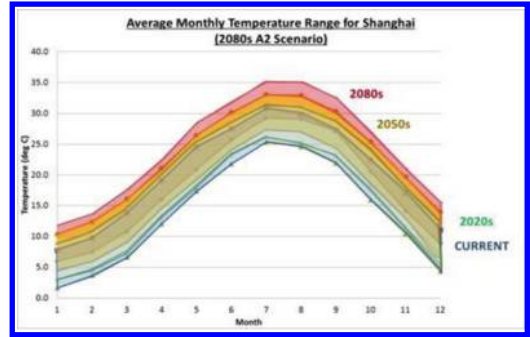


Figure 4. Changes in average monthly temperature range in the TRY for Shanghai under the A2 scenario.

“morphing method” (Belcher et al. 2005), which uses differences between monthly averages from “historical periods” and “future periods” in climate models, and impose these onto existing Test Reference Year. For this study, data from HadCM3 are separated into four periods: 2000s (which includes data from 1990–2009); 2020s (which includes data from 2010–2039); 2050s (which includes data from 2040–2069) and 2080s (which includes data from 2070–2099), and the average monthly temperatures under the A2 scenario from resulting TRYs for Shanghai is shown in Figure 4.

3 METHODOLOGY

In order to run a large number of simulations quickly, a simple 2nd Order Simulation model (Chow & Levermore 2007) was used with current and future TRYs from Beijing, Shanghai and Guangzhou were inputted as weather files, the average heating and cooling demands were calculated for a series of rectangular-shaped buildings of various sizes (from 1000 m^2 to 10000 m^2 in total floor area). The glazing applied is for 60% of the façade areas, and three levels of U-value settings were also used, based on:

- The current specification of existing buildings in the three cities (before retrofitting)
- The current national standard for new buildings in the three cities (after retrofitting)
- The high-specification building with similar standards to the Building Regulations in England and Wales (DCLG 2006) (further improvements).

This is summarised in Table 1. Verification of results from the 2nd Order Simulation model was conducted by comparing samples of the results with one of the leading commercial software

Table 1. Physical Properties of the three office types to be simulated for Beijing, Shanghai and Guangzhou.

	Beijing		Shanghai		Guangzhou		High-spec
U-value (W/m ² K)	Existing building	Current Reg (J12070-2012)	Existing building	Current Reg (JGJ- 134-2010)	Existing building	Current Reg (DBJ15-51- 2007)	Current Reg for England and Wales
Windows	2.7	1.8	3.2	2.8	5.8	3.0	1.8
Roof	0.54	0.35	0.8	0.8	1.0	0.9	0.2
Ext. walls	0.76	0.40	1.34	1.0	1.5	1.1	0.35
Ground	0.53	0.45	1.5	1.5	1.5	1.5	2.2
Glazing used	Double- glazing	Double-glazing (low-e)	Double- glazing	Double- glazing	Single- glazing	Double- glazing	Double-glazing (low-e)

simulation packages, IES VE, and the results were very similar in cases with and without direct solar gains. This was to ensure that similarities in results were not a result of cancellation of over-prediction in air temperature with under-prediction in solar gains or vice versa.

As for the actual HVAC systems, in this analysis, a natural gas boiler is assumed for heating, and a chiller for cooling. For the calculation of primary energy consumption, a ratio of 2.579 is applied to electricity for cooling.

3.1 Internal gains

In an occupied office building, there will be lighting, equipment and occupants that will increase the heat inside the room. In this study, an extra 30W/m² has been added in the simulation as internal gains.

3.2 Internal comfort range

For the series of simulations, the comfort temperature range was set between 19°C and 24°C, such that if the internal temperature falls below 19°C, heating will be supplied and if it reaches 24°C or above, cooling is activated. As well as testing for energy demands with retrofitting, a second parallel run of simulations was conducted using a bigger comfort temperature range of 15°C to 28°C (4°C either side of the initial range) to see the effects of greater occupant tolerance.

3.3 Application of external shading

One of the key passive energy saving strategies for reducing cooling demand is the application of external shading. This is also tested in the thermal performance modeling to verify its effect on office buildings in the three regions under current and future climates.

4 RESULTS AND DISCUSSION

4.1 Potential savings in heating/cooling demands

Figure 5 shows the differences in heating, cooling and total primary energy consumption between existing, retrofitted and high-spec buildings in Beijing, Shanghai and Guangzhou (from definitions in Table 1) under the current climate.

It can be seen that with the exception of Shanghai, retrofitting to the latest standards are highly effective, with a reduction of total primary energy of 30% in Beijing, and 44% in Guangzhou. Retrofitting only reduced total primary energy consumption in Shanghai by 1% (and cooling loads actually increased slightly as a result of retrofitting), and this is mainly due to the fact that there difference between the current building specifications and the previous one is not that great. However, if office buildings in Shanghai are to improve their office buildings further, to the standards similar to the Building Regulations in England and Wales, the reductions from the existing building values are close to 30%. The current standards for Beijing are already very close to the Building Regulations in England and Wales, thus there is very little effect in improving them further. For Guangzhou, further improvements suggest a further 25% reduction from the retrofitted buildings (58% from the current buildings), and so it is a worth further investigation on the feasibility of such applications.

4.2 Effects of future climate change

Figure 6 shows the heating and cooling demands for Existing Buildings, Retrofitted Buildings, and High-Spec Retrofitted Buildings in Beijing for the 2000s, 2020s, 2050s and 2080s, under the two extreme IPCC future scenarios (A1 and B1), together with the medium-high A2 scenario. Results for Shanghai and Guangzhou are summarised in Table 2 and Table 3 respectively.

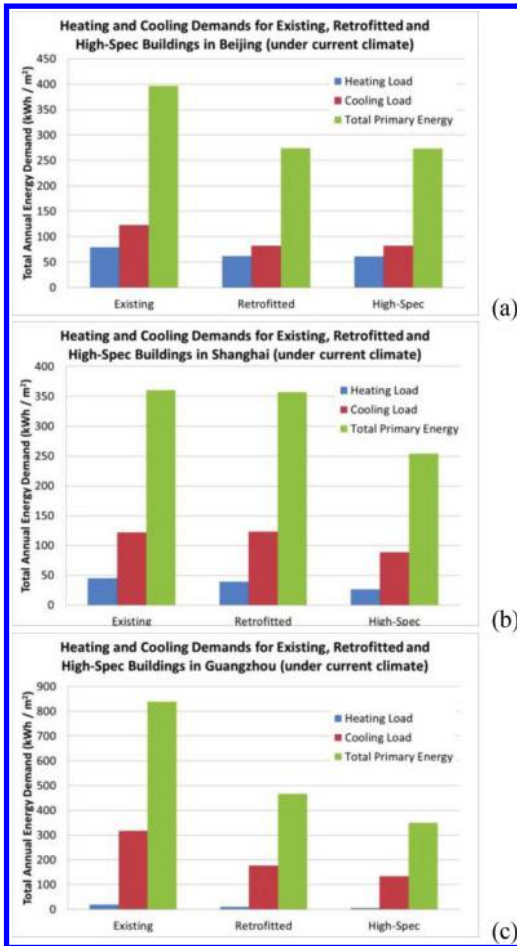


Figure 5. Comparisons of heating and cooling demands for Beijing (a), Shanghai (b) and Guangzhou (c) under the current TRY.

With future climate data, cooling demand rises sharply in all three cities, accompanied by a less severe reduction in heating demand (except in Guangzhou where there is virtually no heating demand currently). With the existing building specification, total primary energy consumption for heating and cooling could rise to between 450 kWh/m² to 500 kWh/m² by 2080s in Beijing depending on the future scenario, whereas for the retrofitted or high-spec building, this range falls to 320 kWh/m² to 360 kWh/m², which is lower than the primary energy consumption for existing buildings under the current climate, showing that retrofitting alone would nullify the effects of future climate change.

For Shanghai, the primary energy consumption rises from around 360 kWh/m² to between

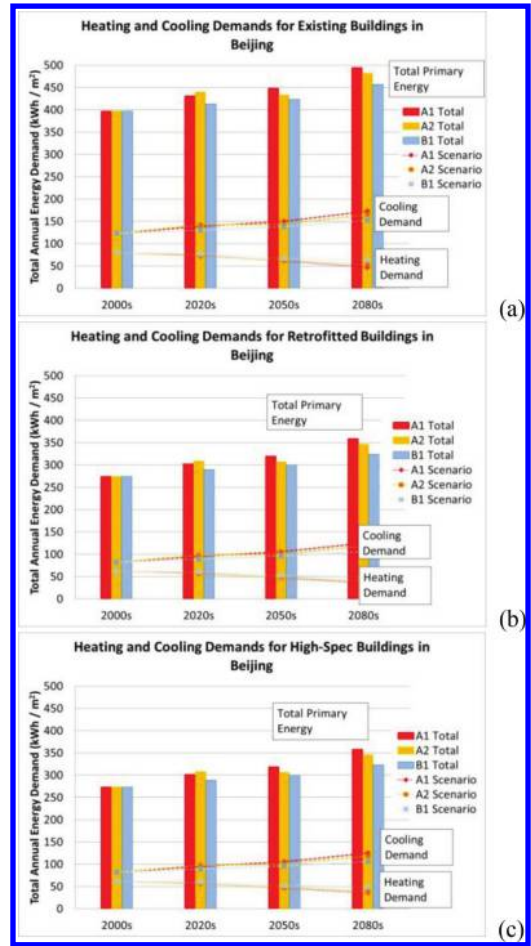


Figure 6. Heating and cooling demands for existing (a), retrofitted (b) and high-spec (c) office buildings in Beijing under future climate change.

428 to 507 kWh/m² by 2080s. However, the effects of retrofitting are not evident (possibly due to the relatively high previous standards), but improving to the standards in England and Wales will almost nullify the effects, with the consumption reaching 367 kWh/m² for 2080 under the A1 scenario.

For Guangzhou, the current primary energy consumption with existing buildings due to heating and cooling is approximately 840 kWh/m², and can be almost wholly attributed to cooling demand. This rises to 925 to 1050 kWh/m² by 2080s. With retrofitted buildings, this is reduced to between 540 to 640 kWh/m² by 2080s, which represents a 25–35% reduction from present values with existing buildings. Further improvements to the England and Wales standards further reduces this to between 410 and 490 kWh/m², representing

Table 2. Heating and cooling demands for existing, retrofitted and high-spec office buildings in Shanghai under future climate change (loads in kWh/m²).

	Heating load	Cooling load	Total primary energy
<i>Existing</i>			
Current	45.24	122.14	360.24
2020s A1	38.30	128.24	369.03
2020s A2	39.43	135.42	388.68
2020s B1	42.47	128.59	374.10
2050s A1	29.72	153.35	425.21
2050s A2	31.90	153.54	427.88
2050s B1	35.64	141.45	400.44
2080s A1	19.27	189.05	506.83
2080s A2	24.41	177.39	481.90
2080s B1	31.37	153.98	428.48
<i>Retrofitted</i>			
Current	39.17	123.19	356.88
2020s A1	33.04	128.54	364.54
2020s A2	33.93	135.42	383.18
2020s B1	36.64	129.10	369.59
2050s A1	25.70	152.48	418.95
2050s A2	27.24	152.68	421.00
2050s B1	31.05	141.43	395.80
2080s A1	16.31	185.83	495.57
2080s A2	20.80	175.12	472.43
2080s B1	27.05	153.11	421.92
<i>High spec</i>			
Current	26.63	88.20	254.10
2020s A1	21.85	93.83	263.84
2020s A2	22.83	98.03	275.65
2020s B1	24.84	93.50	265.98
2050s A1	16.48	112.89	307.62
2050s A2	17.89	111.46	305.35
2050s B1	20.45	103.22	286.65
2080s A1	9.60	138.68	367.26
2080s A2	12.73	129.15	345.81
2080s B1	17.34	112.01	306.21

Table 3. Heating and cooling demands for existing, retrofitted and high-spec office buildings in Guangzhou under future climate change (loads in kWh/m²).

	Heating load	Cooling load	Total primary energy
<i>Existing</i>			
Current	19.80	317.30	838.12
2020s A1	14.93	332.38	872.14
2020s A2	14.41	328.92	862.69
2020s B1	16.56	332.47	874.00
2050s A1	7.19	357.08	928.10
2050s A2	7.64	354.38	921.59
2050s B1	11.96	351.81	919.28
2080s A1	1.16	406.71	1050.07
2080s A2	2.98	379.52	981.76
2080s B1	8.63	355.80	926.24
<i>Retrofitted</i>			
Current	10.33	177.31	467.61
2020s A1	7.20	190.29	497.96
2020s A2	6.89	188.01	491.77
2020s B1	8.28	188.81	495.22
2050s A1	2.72	209.66	543.43
2050s A2	3.02	207.63	538.50
2050s B1	5.60	202.67	528.29
2080s A1	0.15	246.36	635.51
2080s A2	0.78	228.14	589.15
2080s B1	3.73	207.68	539.34
<i>High spec</i>			
Current	4.76	133.43	348.88
2020s A1	3.20	144.53	375.94
2020s A2	2.75	142.83	371.11
2020s B1	3.65	142.58	371.36
2050s A1	0.62	160.62	414.86
2050s A2	0.77	158.99	410.81
2050s B1	2.26	153.89	399.14
2080s A1	0.00	190.06	490.16
2080s A2	0.15	176.44	455.19
2080s B1	1.15	158.49	409.90

a 49–59% reduction. Guangzhou is the only city in this study where there are significant reductions with subsequent improvements in the building specifications. For Beijing, improving from the current standard and the England and Wales standards have little effect, and for Shanghai, retrofitted existing buildings to the current code have represents little improvement.

4.3 Other possible changes to reduce energy demand

As mentioned previously, two further parallel set of simulations was run, one with an increased temperature comfort range of 15°C to 28°C, and the second with applied external shading. From Figure 7,

it can be seen that the effects of extended the comfort temperature range is similar to the effects of retrofitting the building in Beijing, and similar to retrofitting the building to the England and Wales standards with a total primary energy reduction of approximately 30%. The effect of applying external shading is slightly less significant, with a reduction of roughly 20%.

For Shanghai, as seen earlier, the effects of retrofitted to the current code is minimal, but further improvement to the England and Wales standard will have a similar effect as extending the comfort temperature range. The effect is similar to Beijing with a reduction around 30%. External shading also performs similarly to Beijing, reducing total primary energy consumption by about 20%.

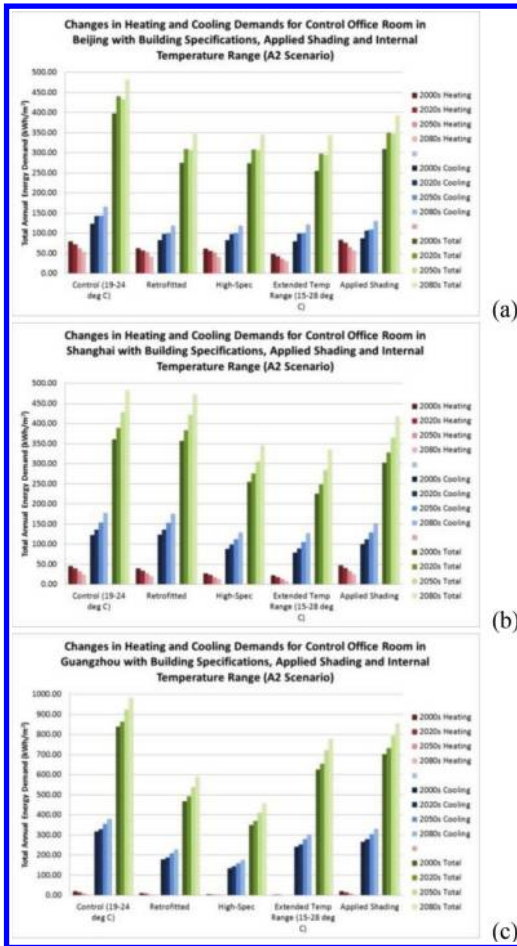


Figure 7. Comparison of heating and cooling demands for existing “control” office building, with retrofitted, high-spec, increased internal comfort temperature range and applied external shading for Beijing (a), Shanghai (b) and Guangzhou (c).

In Guangzhou, the extension of comfort temperature does not perform as well as retrofitting (25% reduction compared with 40% with retrofitting to the latest code), and retrofitting to the high-spec (England and Wales) standard can potentially reduce total energy consumption to less than a half.

The effects with applying external shading is significant, but is not as high as extending the comfort range. Similar to Shanghai and Beijing, the reduction with applied external shading is approximately 20%.

In all cases, the extension of comfort temperature range alone would be able to nullify the effects of future climate change up to 2080s, the

application of external solar shading can also nullify the effects of climate change up to 2080s for Beijing (with the exception of the A1 scenario), but will not be able to do so in Shanghai, not even for 2050s (with the exception of the B1 scenario). For Guangzhou, the effects of climate change in 2050s can be nullified by external shading alone, but by 2080s, it would not be able to do so (with the exception of the less severe B1 scenario).

4.4 Combination of different strategies

The effects of combining different energy-reducing strategies were also tested to see the full extent of the potential for energy reduction in the future.

Figure 8 shows that for Beijing, the combination of retrofitting (to either the latest code or

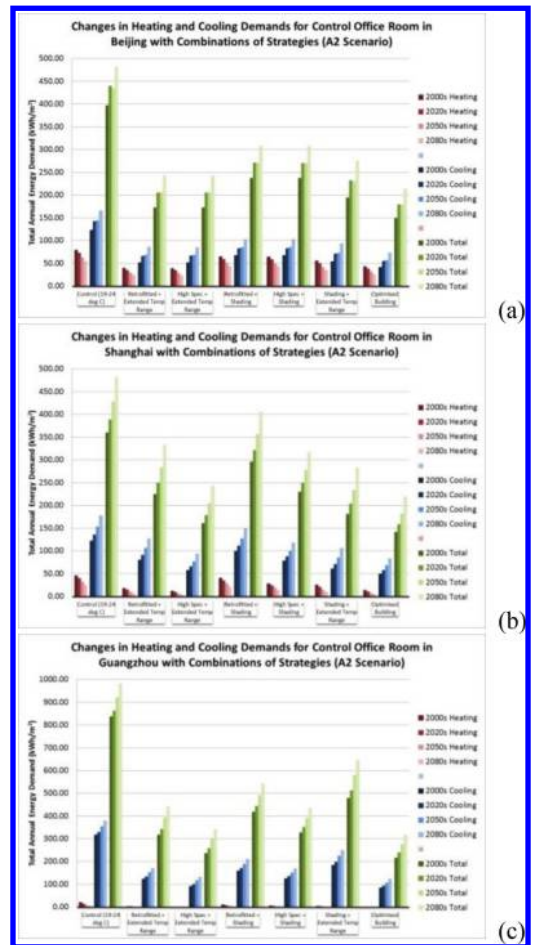


Figure 8. Comparison of heating and cooling demands for existing “control” office building, with combinations of energy-reducing strategies for Beijing (a), Shanghai (b) and Guangzhou (c).

to high-spec), together with an extended comfort temperature range could reduce total primary energy consumption by about 50%. Combining retrofitting with external shading is slightly less effective, with a reduction of 40%. If the building is not retrofitted, but shading is applied together with an extended comfort range, the reduction is about 45%. With the “optimised” office building, where the U-Values were set with those for the “high-Spec” building, the comfort temperatures set at the wider 15°C to 28°C range, and direct solar gains were excluded by means of external shading, the reduction could be as much as 63% (under current climate) to 54% (comparing under 2080s A1 scenario).

For Shanghai, it was seen that retrofitting alone has limited effects, thus combination of strategies with retrofitting is similar to the effects of those strategies alone. However, combining these strategies with a high-spec office does provide added improvements in reduction. For example, external shading with a high-spec office would provide approximately 35% reduction in total primary energy consumption, whereas a high-spec office with an extended comfort range reduces 50% of the total energy. Shading and extended comfort temperature range for a non-retrofitted office results in a 40% reduction. With an “optimized” office, the reduction could be as high as 60% (under current climate) to 53% (comparing under 2080s A1 scenario).

For Guangzhou, the reduction in energy consumption could be very high. Retrofitting (to the latest national code or to the England and Wales standard) together with either external shading or the extension of comfort temperature range nearly all produced reductions in total primary energy consumption by more than 50% (with the exception of retrofitting to national code and external shading, which had a reduction of approximately 45%). Shading and extended comfort range together reduced consumption by roughly 40%. The “optimized” office in Guangzhou could provide reduction as high as 75% (under current climate) to 67% (comparing under 2080s A1 scenario).

5 CONCLUSIONS

The effects of retrofitting to the current building code in the three cities in this study depended highly on the building specification of the existing building. In the case of Shanghai, the existing specifications are already very close to the current building code, thus further retrofitting to the latest code has very little effect. For Beijing and Guangzhou, the required U-Values for key elements are significantly lower in the current codes, thus, for these

two cities, the effects of retrofitting is clear. For the case of Beijing, the latest code is already very close to the England and Wales building standards, thus further improving to these standards have limited effects, whereas Shanghai can benefit from another step change in building specification.

Guangzhou is the only city in this study where there are significant improvements with each subsequent retrofit. One reason for this is that the current building stock are still have relatively high U-Values, and the current code could still be improved further to the standard in England and Wales.

With the exception of Shanghai, retrofitting to the current code will reduce total primary energy consumption to an extent that even by 2080s, the requirement for heating and cooling would be lower than that for the current climate with existing buildings. This suggests that retrofitting to the current code is able to nullify the effects of future climate change by the end of this century. The effects appear to be greatest in Guangzhou, partly because there is currently very little heating demand in Guangzhou, so the “benefit” of heating load reduction with future warming does not apply there. Hence the effects of buildings with higher specification become more significant.

The effects of applying external shading and extending the comfort temperature range were also investigated in this paper, and it was found that in Beijing and Shanghai, the effects of extending the comfort temperature range to 15°C to 28°C (from 19°C to 24°C) to an existing building would be similar to retrofitting the building to the standard in England and Wales. This extended comfort range is already in place across much of China, especially in residential buildings, and is one method of reducing energy consumption in most places, and some offices are also adapting to this approach, especially in the south of the country, although retrofitting still performs better than extending the comfort range for Guangzhou. Applying external shading also has a strong positive effect, albeit less so than extending comfort range or retrofitting, and can reduce energy consumption by around 20%.

Combining all the energy-saving strategies (retrofitting, extending comfort range and applying external solar shading) to provide an “optimum” building could reduce total primary energy consumption by between 40% and 75%. The reduction is more significant when comparing under current climate and also more significant in Guangzhou than in Beijing or Shanghai.

Although the effects of climate change were investigated in depth for this study, effects of future urban heat islands due to further urbanisation were not taken into account. This will make

the quest for producing low-energy buildings much more challenging, and further work is required to fully investigate and understand the effects of passive strategies on energy consumption in office buildings in China.

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Dynamical analysis of a substation structure under short-circuit current loadings

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ABSTRACT: The past several decades have seen a rapid increase of open-air substations to satisfy energy demands, which is partly contributed by up-to-date knowledge and development on short-circuit mechanical effects in substations from a structural engineering viewpoint. Substation structures with flexible conductors are complex, and nonlinear dynamics analysis is required. In this paper, ADINA will be used for such purposes with considerations of geometry nonlinear behaviour of conductor cables under short-circuit loadings. Focus is placed on the investigations of effects of droppers and spacers, together with pinch effects for twin conductors considering the contact nonlinearity.

1 INTRODUCTION

The substation structure comprises the support structure, and cable conductors, which is used for electric power transmission. The support structure is usually made of concrete and the cable is made of copper or aluminium. The short circuit currents (or fault currents) for flexible cable conductors in AC (alternating current) system cause electromagnetic field around the conductors, which produce tensile forces on the cables and consequently forces on the members of the support structure. Such substation structures are subject to self weight and environmental loadings as well as electromagnetic forces caused by electricity current. The electromagnetic effect may be critical especially under short-circuit conditions, although the duration of short-circuit is usually less than one second. Due to geometry-nonlinearity of cables, analysis of such structures is complex involving nonlinear structural dynamics. The traditional method in the past for design of such structures was based on empirical equations under static (quasi-static) conditions for simple linear models without considering geometry-nonlinearity, e.g. BS EN 60865-1:1994, Borhaug et al. (1971) and Bergeron (1994).

The linearity may not lead to a conservative solution for geometry-nonlinearity problems, which is not the case for material-nonlinearity problems, see Cook et al. (2001). Also, the static analysis cannot present real situation. Therefore, the analysis of such structures should be based on complex nonlinear dynamics. Real physical testing was performed for a number of substation structures under short-circuit loadings and the data for structural behaviour were documented. CIGRE Study

Committee (1996). Numerical simulations were suggested for design of such structures, since the short-circuit effects were dominant. Experimental investigations of electric and magnetic fields were performed by Charalambos et al. (2012). Hooman et al. (2013) proposed dynamic analysis for the wind load of an overhead transmission with fluid-structure interaction analysis method and finite element method. They showed that the proposed method could estimate the conductor motion and the cable tensions more accurately than the traditional quasi-static method for wind, as is the case for the investigation of wind in this paper.

The ADINA software with large deformation were employed to investigate the behaviour of the substation structures under dynamical short-circuit conditions in this paper. Focus was placed on the effects of droppers, spacers and pinches, since there is very limited information available in the literature for these.

2 THEORETICAL BACKGROUND

In general, the short current phenomenon occurs during the transient variation of voltage. The current change causes electromagnetic change, which would cause electromagnetic force to conductor. For single conductor system, the conductor would suffer the electromagnetic force caused by other conductor system with different current phases. This would lead to the conductor suffering from tension forces, drop forces, pinch forces etc in a magnetic field where relevant, see BS EN 60865-1:1994.

A three-phase AC current system is used for the substation in this study, as is the practical case in

the real situation. The symmetrical fault (short-circuit current) is used, which means that the three phases are balanced by themselves (the summation of the currents from all the three phases are always zero) and so there is no earth lines involved, see Glover et al. (2011). The unsymmetrical fault is more complex and complies with many practical conditions like thunderstorm. However, the symmetrical fault also occurs under high voltage especially with the mutation of the current acting on the conductor system such as the open and close, and this has been taken as the design guidance for a number of substation structures for National Grid UK.

The Finite Element Method (FEM) is used in this study for numerical simulations. FEM is an approximate numerical method for solving differential equations. The basic idea of FEM is discretization, i.e. to divide the domain of interest into a number of elements called finite elements, and for classical FEM, this is with the help of interpolation functions, which are applied after the transformation from strong forms (differential equations) to weak forms (integral equations). FEM is the most widely used numerical method for engineering applications. In this study, the conductor exhibits highly geometry-nonlinearity. So the current configuration has to be used in order to consider this, compared against a linear structure where the initial undeformed configuration is used. FEM software ANDIA was used to perform such simulations. Detailed theory behind FEM can be referred to Bathe (1996).

The calculation of short-circuit forces on conductors can be found in well-documented guide books, see CIGRE Study Committee (1996) and ADINA online manuals (2013). Considering two electric conductors traversed by short-circuit currents $i_1(t)$ and $i_2(t)$ and placed in a medium of magnetic permeability μ_0 , each differential element dr_2 of one conductor exerts on a differential element dr_1 of the other conductor a magnetic force given by Grassman's formula (see Fig. 1):

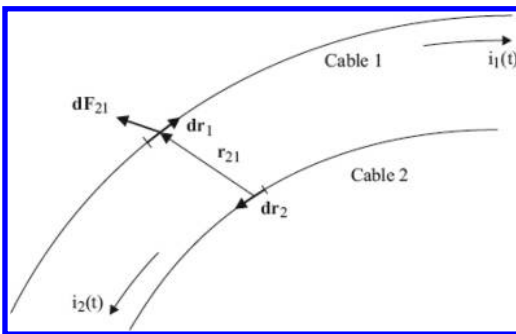


Figure 1. Conventions for electromagnetic loading.

$$dF_{21} = \frac{\mu_0}{4\pi} i_1(t) i_2(t) \frac{dr_1 \times (dr_2 \times r_{21})}{\|r_{21}\|^3} \quad (1)$$

3 NUMERICAL SIMULATIONS

3.1 General description and simulation parameters

This research was based on a practical National Grid project in UK: Assessment of Blyth 275 kV Substation Structure. The voltage for the substation power system is 275 kV. The duration for the short-circuit current is 0.3 s duration and the magnitude of the current is 31.5 kA (RMS value), as recommended by the National Grid Standard such as NGT3.1.4 (2014).

Time function of the short circuit is described by

$$f(t) = f^*(t) \sqrt{\frac{\mu_0}{4\pi}} \cdot \sqrt{2} \cdot I \cdot \left[\sin(\omega t - \phi + \alpha) + \exp\left(\frac{-t}{\tau}\right) \cdot \sin(\phi - \alpha) \right] \quad (2)$$

where $f(t)$ is the actual short circuit current at time t , $f^*(t)$ is the input multiplying factor, μ_0 is magnetic permeability in (volt-seconds)/(meters-amperes), I is the RMS value of short circuit current, ω is the angular frequency in degrees/unit time, ϕ is the phase angle in degrees, α is the impedance angle in degrees and τ is the time constant (unit of time). In the simulations presented in this paper, $\mu_0 = 1.26 \times 10^{-6}$, $I = 31.5$ kA, $\phi = 18000$ and $\tau = 0.081$, as recommended in practice.

The general properties for the copper material used for cable conductors and droppers are with a density of 8900 kg/m³, Poisson ratio of 0.3 and Young's modulus of 1.05×10^{11} N/m². It is taken as an elastic material in the simulations.

The general properties for the C30 concrete used for the support structure are with a density of 2400 kg/m³, Poisson ratio of 0.2 and Young's modulus of 3.0×10^{10} N/m². It is taken as an elastic material in the simulations (material nonlinearity of concrete is not considered).

The support structure is a standard HS43 substation structure with the height of 13 m (the distance between of the end supports is 60 m). The size of cross beam is 0.381 m $\times$ 0.533 m (1.25 inch $\times$ 1.75 inch). The size of column is 0.457 m $\times$ 0.457 m (1.5 inch $\times$ 1.5 inch) at the bottom with varied section along the height. The flexible conductor with copper material would be used. Cable conductors are modelled using large-deformation tension-only trusses, while beams and columns for the support structures are modelled

using Euler–Bernoulli beams. The boundary of the restraint is treated as fixed to the ground (pad foundations were not included in the simulations). A dynamical analysis (dynamic-implicit) was carried out where Rayleigh damping is used. The verification of the model is done by comparing the theoretical hand calculations of the cable conductors with those obtained from ADINA for a simple scenario. In the simulations that follow, the gravity loading will be applied for all the time, but the short-circuit will be applied with a duration of 0.3 s. More details can be found in Jiang (2015).

The simulations presented were for twin-conductors, where each phase comprises two bundled conductors and the RMS value of I becomes half of the specified 31.5 kA. Four models were presented as shown in Figure 2 to Figure 5, where

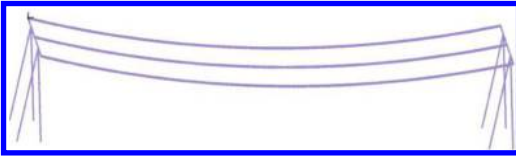


Figure 2. Substation structure without droppers or spacers.

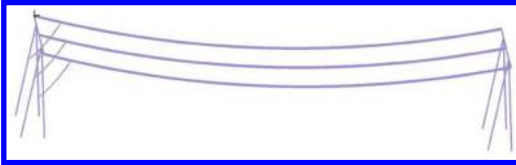


Figure 3. Substation structure with droppers but without spacers.

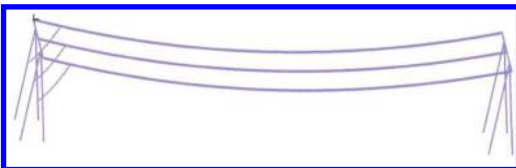


Figure 4. Substation structure with droppers and spacers.

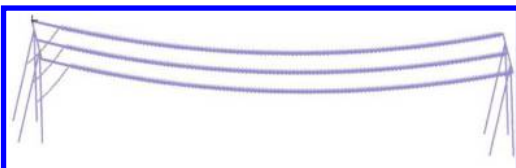


Figure 5. Substation structure with droppers, spacers and contact pairs.

the layout of the substation structure without droppers, layout with droppers, layout without droppers but with spacers (three spacers were placed every 14 m along each twin-conductor, and modelled as rigid elements), layout with droppers, spacers and contact pairs (contact pairs to mimic pinch effects). In contact pairs, if there are two nodes sufficiently close (distance smaller or equal to 10^{-6}) together, then they will stick together as one node and otherwise they depart. The gravity loading and snow and wind loadings were added to all the cases.

3.2 Simulation results

The reported results presented only show the cable tension at the end connecting the support beam (the first phase). The cable tension is an important index for the design and checking of the conductor itself, and it could also induce the relevant other indexes like bending moment for the beams and columns of the support structure. More details can be found in Jiang (2015).

Figure 6–Figure 9 show the cable tension forces for the four cases respectively. It can be seen from Figure 6 and Figure 7 that the inclusion of the droppers makes the peak value of cable tension larger, which is reasonable since this decreases the sag (results not shown in this paper, see Jiang (2015)). From Figure 7 and Figure 8, it can be seen that the inclusion of droppers makes the peak value of cable tension smaller, thus improving design performance. Further, the inclusion of contact pair makes the cable tension even smaller, which

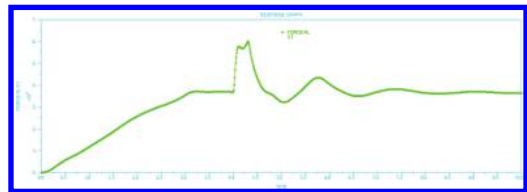


Figure 6. Cable tension force for the system without droppers or spacers.

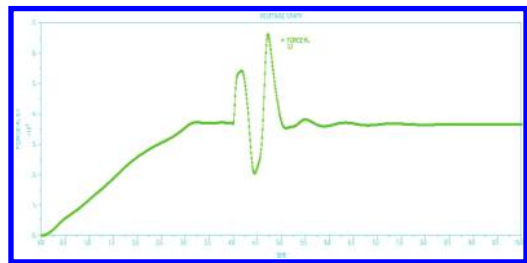


Figure 7. Cable tension force for the system with droppers but without spacers.

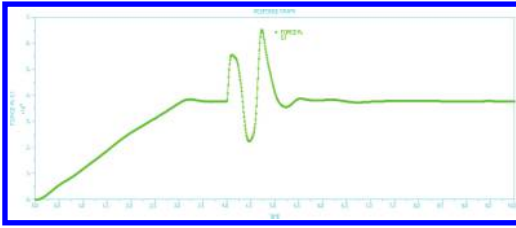


Figure 8. Cable tension force for the system with droppers and spacers.

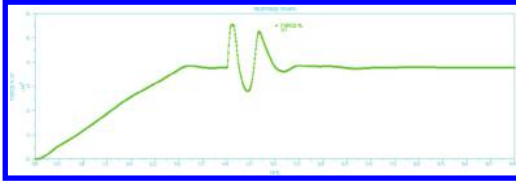


Figure 9. Cable tension force for the system with droppers, spacers and contact pairs.

indicates that the design of such structures without considering the pinch effects will make a conservative solution. In modelling the pinch effects, some nodes of the cables stick together as one unit and will depart when the electromagnetic forces between the nodes decrease to a certain level. More details are presented in Jiang (2015).

4 CONCLUDING REMARKS

Simulations have been performed for substation structures with flexible conductors considering geometry nonlinearity and contact nonlinearity using ADINA. The effects of droppers and spacers, pinches for twin conductors have been investigated. It is found that the inclusion of the droppers increase the peak value of cable tensions while decreasing the cable sag at the mid-span and so reasonable balance as to whether to use droppers should be made. It is found that inclusion of spacers will improve the design performance. From the simulation results, it is found that the pinch effect will decrease the cable tension force and therefore it will lead to a conservative solution if is ignored in the design consideration. All the simulations were based on balanced short-circuit conditions

(symmetric fault). Further work will be explored to investigate the more challenging unbalanced short-circuit conditions (unsymmetric fault).

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Investigation and calculation of energy consumption in modern prefabricated building

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ABSTRACT: Based on the theory of life cycle assessment, the project of prefabricated housing in the whole life cycle is divided into four stages: designing, production and construction, using and maintenance, and demolition and abandonment. Combined with the carbon emission characteristics in each stage, carbon emissions calculation models which give a convenient to make empirical study on energy-saving emission reduction benefits for the project of prefabricated housing are established. Using these calculation models, this paper makes a comparative carbon emissions analysis of a project of prefabricated housing which is in Shenyang City, Liaoning Province, China with a traditional residential project whose engineering scale is similar. The conclusion is that the energy-saving emission reduction benefits for the project of prefabricated housing are remarkable.

1 INTRODUCTION

With the rapid development of industrialization and urbanization, China has become one of the countries in the world which have the most new buildings and the most production of building materials. While these achievements, a lot of energy and building material are consumed, which show a surge trend in carbon emissions and environmental destruction. Compared to the traditional form of construction, prefabricated construction form has elevated the quality, shortened the construction period, saved resources and energy, and promoted environmental protection, etc. Its standardization of architectural design, industrial components production and assembly of construction mechanization, scientific organization and management have improved production of traditional Chinese architecture, promoted product refinement and standardization to achieve industrialization of construction, expanded labor productivity so as to ensure the overall quality and function of the building, reduce the consumption of resources and energy, achieve green energy saving benefit.

On July 22, 2015, France has approved the new Energy Bill (Energy Transition for Green Growth Bill, 2015). Under the bill, France's carbon tax rate will be 56 euro per ton of carbon dioxide in 2020, and by 2030 will continue to increase to 100 euro per ton. In France, a carbon tax on fossil fuels such as natural gas, oil and coal. According to the Intergovernmental Panel on Climate Change statistics, construction industry consumes 40% of global energy, and accompanied with 36% of emissions of

greenhouse gases, which has become one of the top three global greenhouse gas emissions (Malmqvist et al. 2010). If the building energy consumption is effectively controlled, China will be easier to achieve by 2020 carbon dioxide emissions per unit of GDP dropped 40% to 45% target (United Nations Environment Program, 2009). Construction of building carbon emission calculation model is to make qualitative comments about low-carbon and green buildings upgrade to quantitative evaluation.

2 RESEARCH STATUS OF LCA THEORY AND RESEARCH FRAMEWORK

Whole life cycle assessment theory is the evaluation tool which based on the entire whole life cycle of a product system considerations, from the extraction and processing of raw materials, to production, packaging, marketing, use, reuse and product maintenance to recycling and final waste disposal (United Nations Environment Program).

2.1 Foreign research status of whole life cycle assessment theory

Overseas for building carbon emissions research work carried out earlier, which has made substantial progress in the field of low-carbon architecture. But foreign researches mainly focused on carbon emissions of building materials and energy and using and maintenance phase, etc. Most of them remain in the qualitative analysis level and lack of quantitative analysis.

John Holmberg made a comparative analysis of the characteristics and using range of whole life cycle assessment theory and the input-output analysis theory. It is concluded that whole life cycle assessment theory applies to research of product and carbon accounting of a single building (Nassen et al. 2007).

Researches on building carbon emission model are mostly based on life cycle theory. The whole life cycle of the building was divided into four stages by Leif respectively for building materials production, construction, operation and demolition (Guavsson et al. 2010). Catarina put forward that the phase of construction accounted 20% for the total carbon emission, and the reduction space of carbon emission in the phase of use and maintenance of construction is very large (Chen 2010).

2.2 Domestic research status of whole life cycle assessment theory

There are also many Domestic research on carbon emission throughout the whole life cycle of build-ings, which has multiple phase partitioning methods.

For example, the carbon footprint of the residence model was studied by Yu Ping in Peking University, which was studied by the whole life cycle assessment theory and was joined with the concept of circulation at the same time, so the whole life of residence is divided into five stages respectively for processing of raw materials, construction, use and maintenance, demolition and disposal (Ping 2011).

The carbon footprint of residence in China were studied by Zhu Yan of Tsinghua University, structuring the building carbon emission calculation model based on the theory of whole life cycle in five stages from building materials production to dismantle, which was combined with the characteristics of Chinese architecture (Zhu & Chen 2010).

Residential energy consumption and carbon emission in cold region in China was researched and analyzed by Wang Songqing. The whole life cycle is divided into five stages, and the conclusion that building in the using phase of the energy consumption and carbon emission accounted for more than 80% of the total, and building materials production reached more than 10% of total was drawn by empirical analysis (Wang 2007).

2.3 Research framework

Based on LCA theory, the issue of resource consumption and environmental benefits of prefabricated housing from the design phase to the demolition stage involved a quantitative and qualitative evaluation, in addition to the construction and use phase of building materials and energy

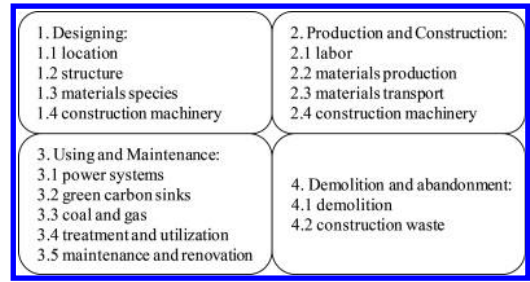


Figure 1. Research ideas about whole life cycle prefabricated houses.

consumption, including production, processing and transportation of building materials demolition phase of construction waste disposal and so on. Research ideas as shown in Figure 1.

As mentioned earlier, most of scholars divided whole building life cycle into five stages, who defined the production and construction as two phases. However, these articles did not consider the carbon emissions of machinery and equipment during the stage of production. Compared with the views of these scholars, whole building life cycle is divided into four phases in this paper, in which the production and construction stages are merged into a single stage. On the basis of existing researches and theories, this can more fully consider the carbon emissions of machinery and equipment during the stage of production and construction, avoiding the accounting error of carbon emissions of production phase in existing literature, and effectively improve the accuracy of the calculations of carbon emissions.

3 THE VARIOUS STAGES OF CARBON EMISSION CALCULATIONS MODEL

Prefabricated houses in the whole life cycle of the total carbon emissions is shown in Equation 1.

$$C_{tot} = C_{de} + C_{con} + C_{use} + C_{rem} \quad (1)$$

where C_{tot} = total building carbon emissions; C_{de} = design phase carbon emissions; C_{con} = production and construction phase carbon emissions; C_{use} = use and maintenance phase carbon emissions; C_{rem} = demolition and abandonment phase carbon emissions.

3.1 Carbon emissions calculation models of designing phase

The actual carbon emissions subsequent stages are deeply influenced by building location, structure,

types and kinds of construction machinery, programs, etc. However, a lot of literature and field research shows that prefabricated houses in the design phase of direct carbon emissions is small, so at this stage carbon emissions in the life-cycle are negligible. Therefore, this stage carbon emissions calculation model is not established.

3.2 Carbon emissions calculation models of production and construction phase

This phase carbon emissions are mainly composed of four parts and as shown in Equation 2.

$$C_{con} = C_{wlo} + C_{bmp} + C_{tra} + C_{mac} \quad (2)$$

where C_{wlo} = labor of carbon emissions; C_{bmp} = building materials production of carbon emissions; C_{tra} = building materials transport of carbon emissions; C_{mac} = carbon emissions from construction machinery.

3.2.1 Carbon emissions of labor

During the construction phase, workers in addition to the on-site construction work, their space also includes living area and office area. In the living area of energy including electricity and gas categories, reflected in the canteen and other aspects of gas consumption as well as air conditioning, television, lighting and other energy consumption. Energy consumption of office area is mainly reflected in computers, printers, lamps and other energy consumption. For calculation purposes, this is only part of the calculation of carbon emissions generated by electricity consumption (Ye 2012), as shown in Equation 3.

$$C_{wlo} = Q_e \times CE_{ele} \quad (3)$$

where Q_e = per capita of power consumption; CE_{ele} = electricity per unit of carbon emission factor, calculated with Table 1 Northeast Power Grid's (Yin 2012).

Table 1. China power grid electricity carbon emission factor.

Power distribution	Electricity carbon emission factor
North China power grid	0.781
Hainan power grid	0.731
Northeast power grid	0.724
East China power grid	0.683
Northwest power grid	0.643
Central China power grid	0.580
China southern power grid	0.577

3.2.2 Carbon emissions of building materials production

Carbon emissions generated by the production of building materials are influenced by building materials and quantity and as shown in Equation 4.

$$C_{bmp} = \sum_{i=1}^n m_i \times p_i \times (1 + \beta_i) \quad (4)$$

where i = the number of types of building materials; m_i = the amount of building materials; p_i = building materials carbon emissions per unit; β_i = the i -th building material loss factor. Table 2 (Bing 2012) illustrates the part of emissions per unit of building materials. Table 3 (Bing 2012) illustrates some of loss coefficient of building materials.

3.2.3 Carbon emissions of building materials transportation

In the process of horizontal transportation of building materials and construction waste, the consumption of fossil fuels by and means of transportation can be calculated by Equation 5.

$$C_{tra} = \sum_{j=1}^n e_m \times E_{mj} \times (T_{mj} + T'_{mj}) \quad (5)$$

Table 2. Carbon emissions per unit of building materials.

Name of materials	Unit	Carbon emissions per unit
Concrete grade C30	m ³	312
Steel	kg	2.792
Template	t	2792
Insulation	m ²	207.91
Masonry	m ³	183.56
Roofing sheet	m ²	12.34
Roof tiles antifreeze	m ²	232.45
Waterproof coating	m ²	35.89
Plastering	m ²	0.206
Exterior paint	m ²	33.56

Table 3. Loss coefficient of building materials.

Name of materials	Loss factor
Sand	0.05
Gravel	0.05
Cement	0.05
Glass	0.05
Steel	0.1
Concrete	0.05
Wood	0.07

where e_m = m kind of carbon emissions in energy conversion factor; E_{mj} = unit of machine-team of j-kind machinery's m energy consumption; T_{mj} and T_{mj}' = the number of machine-team of transportation of building materials and construction waste from the j-kind tool consumes m-kind energy.

3.2.4 Carbon emissions of construction machinery

Calculation method is shown in [equation 6](#). It should be noted that the amount of the carbon emissions of the materials transporting period has been calculated already, therefore the amount of the carbon emissions of the level transporting machinery is not calculated again.

$$C_{mac} = P_{mj} \times N_{mj} \times E_{mj} \quad (6)$$

where P_{mj} = carbon emissions conversion factor of m-kind energy-consuming from j-kind machinery; N_{mj} = the number of machine-team of m-kind energy-consuming from j-kind machinery in the construction and installation phase.

3.3 Carbon emissions calculation models of using and maintenance phase

This phase carbon emissions are mainly composed of five parts and is shown in [Equation 7](#).

$$C_{use} = C_{ele} + C_{cag} + C_{gre} + C_{sad} + C_{mat} \quad (7)$$

where C_{ele} = carbon emissions of electric energy consumption; C_{cag} = carbon emissions of coal and gas consumption; C_{gre} = green carbon sink capacity; C_{sad} = carbon emissions of water consumption and water treatment; C_{mat} = carbon emissions of maintenance and renovation of the building materials.

3.3.1 Carbon emissions of electric energy consumption

Calculation method is shown in [equation 8](#).

$$C_{ele} = \sum Q_k \times CE_{ele} \times t_{ope} \times n_{ope} \quad (8)$$

where Q_k = k-th electric power consumption per unit time; t_{ope} = electric annual operating time; t_{ope} = building life.

3.3.2 Carbon emissions of coal and gas consumption

Calculation method is shown in [equation 9](#).

$$C_{cag} = \sum (CE_{coa} \times q_{coa} + CE_{gas} \times q_{gas}) \times n_{ope} \quad (9)$$

where CE_{coa} and CE_{gas} = per unit of carbon emission factor from coal and gas, calculated with [Table 4](#) (*China Energy Statistical Yearbook and Listing of*

Table 4. The main energy carbon emission factor listing.

Types	Unit	Containing energy	Carbon emission factor
Raw coal	kg	20908	2.06
Crude	kg	41816	3.07
Natural gas	m ³	38931	2.18
Diesel fuel	kg	42652	3.16
Fuel oil	kg	41816	3.13
Kerosene	kg	43070	3.15
Coke oven gas	m ³	16726	0.74
Gas	m ³	5227	0.23

national gas guide from IPCC); q_{coa} and q_{gas} = the annual average consumption of coal and gas.

3.3.3 Green carbon sink capacity

Calculation method (Yin 2012) is shown in [equation 10](#).

$$C_{gre} = K_{gre} \times S_{str} \times n_{ope} \quad (10)$$

where K_{gre} = the amount of carbon in per unit time by per unit area of green absorption; S_{str} = green building area.

3.3.4 Carbon emissions of water consumption and treatment

Calculation method is shown in [equation 11](#).

$$C_{sad} = (CE_{sup} \times Q_{sup} + CE_{dra} \times Q_{dra} + CE_{sew} \times Q_{sew}) \times n_{ope} \times CE_{ele} \quad (11)$$

where Q_{sup} , Q_{dra} and Q_{sew} = water consumption, water discharged to municipal sewers and sewage from treatment station; CE_{sup} , CE_{dra} and CE_{sew} = corresponding to per unit of carbon emission factor.

3.3.5 Carbon emissions of maintenance and renovation of building materials

Calculation method is shown in [equation 12](#).

$$C_{mat} = \sum CE_{mat} \times Q_{mat} \times (1 - \eta_{mat}) \quad (12)$$

where CE_{mat} = carbon emission factor of building materials; Q_{mat} = the amount of building materials; η_{mat} = recovery factor of building materials.

3.4 Carbon emissions calculation models of demolishing and abandoning phase

This phase carbon emissions are mainly composed of two parts and is shown in [Equation 13](#).

$$C_{rem} = C_{dem} + C_{aba} \quad (13)$$

Table 5. Construction waste production rate coefficient.

Structure type	Steel and concrete			
	Steel	concrete	Brick	Hybrid
Scrap	29.2	18	1.4	13.8
Waste concrete sand	651.3	1494.7	482.2	894.3
Waste bricks	217.1	233.8	384.1	400.8
Combustible waste	7.9	25	37.2	25
Cullet	2.6	1.7	1.8	1.7
Total	908.1	1773.2	906.7	1335.5

where C_{dem} and C_{aba} = demolition and abandonment phase carbon emissions.

3.4.1 Carbon emissions of demolition phase

Calculation method is shown in equation 14. (Wang 2007)

$$C_{dem} = C_{con} \times 90\% \quad (14)$$

where C_{con} = producing and constructing phase carbon emissions.

3.4.2 Carbon emissions of abandonment phase

Calculation method is shown in equation 15.

$$C_{aba} = \delta_{was} \times L_{tra} \times CE_{too} \times W_{dis} \quad (15)$$

where δ_{was} = construction waste production rate coefficient, calculated with Table 5 (Teng 2011); L_{tra} = transporting distance; CE_{too} = unit carbon emission factor of transportation; W_{dis} = transportation of weight and load.

4 EMPIRICAL ANALYSIS

The 17# building of Shenyang Vanke Spring River adopted the framework developed by Japan Kashima, its columns, beams, laminated floor, inner walls, external walls, balconies, staircases and other structures components were prefabricated, then transported to the constructing site, and then in the field hoisted and installed. The framework used sandwich to maintain the heat preservation from the outside, which effectively played the role of the heat preservation by the outside walls; prefabricated housing's flexibility also embodies the advantages in the aspect of resisting the earthquakes. Besides, its industrial production and construction highlight the characteristics of energy saving, emission reduction and other green areas.

The durability of prefabricated housing's holding system has greatly extended the time of utilization and saved a lot of constructing land when

compared to the monolithic reinforced concrete structure. At the same time, a lot of materials that are stronger and lighter used, which have better characteristics of anti-seismic, so that the building floors can be more and more land can be saved.

Previously established carbon emissions models are applied to Vanke Spring River 17#. Then, make a comparison between prefabricated housing with similar construction scale of traditional residence in various stages of carbon emissions. Next part will be described in detail some of the comparative analysis of a representative.

4.1 Analysis of production and construction phase

Through the research summary, we can conclude that the carbon emissions in the producing and constructing phases of 17#, Vanke Spring River, labor accounted for 2.2%, building materials production accounted for 75.5%, building materials transportation accounted for 4.2% and 18.1% of construction machinery using.

In addition, through the study of literature, interview of experts, the measure of data, the steel, concrete and other building materials consumption per unit area can be concluded. Comparing with experiential consumption per unit area of traditional residence, the saving amount per unit area can be obtained. Furthermore, by this mean of referencing to data in "The Price of per Machine per Team in Construction Engineering in Liaoning Province", the carbon emission of construction machinery can also be calculated.

4.2 Analysis of using and maintenance phase

According to the survey data show that Vanke Spring River 17# average summer temperature indoors is 23°C, the average winter temperature before heating chamber is 20°C. While the traditional houses in summer and before heating in winter the average indoor temperature is 27°C and 15°C, the envelope thermal performance of prefabricated houses has greatly improved, which provide people with a warm, comfortable, harmonious, green living environment.

Statistics shows that the indoor temperature is lowered 1°C, energy consumption can be reduced by 5%–10%, while in the cooling conditions, the indoor temperature is increased by 1°C, energy consumption can be reduced by 8%–10% (GB50189-2015). By contrasting prefabricated housing and traditional residential electricity consumption of household appliances, make a conclusion that in using and maintenance phase of prefabricated housing has significant energy efficiency.

17# building the average household annual heating costs and gas costs for the 3350 RMB

and 361.8 RMB. After consulting the relevant departments, 1,000 square meters of heating area consumes about 110 t of coal per month. Based on projections 17# monthly heating needs coal 1038.46 t. According to the Shenyang public piped natural gas price of 3.30 RMB/m³ and natural gas consumption per household per month for 9.14 m³, by the conversion to give 17# the annual consumption of natural gas is about 13161.6 m³. The same data can be measured in traditional houses.

Vanke Spring River residential building area of approximately 423000 m², while 17# building area of 9375 m². Greenbelt by the size of trees, shrubs, flowers and close planting, covering an area of approximately 114210 m². After the conversion of green carbon sinks as -3480.5 t, contrast of traditional residential as -3041.7 t.

About 335 households in 17# building. According to the average water consumption of residents in Liaoning Province with the 0.18 t per person per day, the 17# annual water consumption is about 22009.5 t, corresponding traditional houses' for 24302.7 t.

4.3 Analysis of demolition and abandonment phase

Construction waste is generally divided into five categories, including scrap, waste concrete sand, waste brick, combustible waste and waste glass (Teng 2011), the proportion as shown in Figure 2. Currently, the highway diesel vehicles are the main form of transport construction waste in Shenyang.

4.4 Comparison of carbon emissions between prefabricated housing and traditional housing

In summary, the carbon emissions comparison results between Vanke Spring River 17# and traditional residence as shown in Table 6.

It can be drawn from Table 1 that compared to traditional housing based on various stages in whole life cycle Vanke Spring River 17# make total carbon emissions reduction about 38985.2 t, equivalent to 4.16 t/m², of which the using and

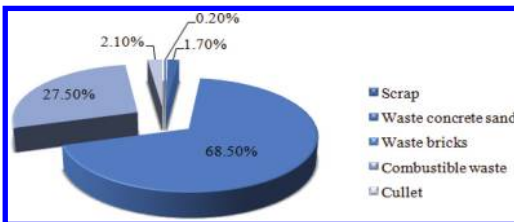


Figure 2. The proportion of construction waste.

Table 6. The carbon emissions comparison results of Vanke Spring River 17# and traditional house.

Phase	Prefabricated housing	Traditional housing
1 Production and construction	1818.8	1909.9
1.1 Labor	40.1	57.1
1.2 Production	1373.1	1414.4
1.3 Transport	76.4	96.4
1.4 Construction machinery	329.2	341.9
2 Using and maintenance	32904.9	71790.5
2.1 Electric energy	19145.3	37155.3
2.2 Coal and gas	16837.1	37147.1
2.3 Green carbon	-3480.5	-3041.7
2.4 Water	309.1	341.3
2.5 Maintenance	93.9	188.5
3 Demolition and abandonment	1744.2	1752.7
3.1 Demolition	1600.9	1667.4
3.2 Abandonment	143.3	85.3
4 Total	36467.9	75453.1

maintenance phase of carbon emission reductions is the most significant.

5 CONCLUSIONS

Based on the life-cycle theory, the building life is divided into four stages. Combined with the characteristics of each stage, carbon emission calculation models are established, which include production and transportation of building materials, construction machinery, electric power, coal, gas, etc. These models provide technical and theoretical support for the assessment of traditional housing and prefabricated housing in aspect of resource and energy conservation, reducing pollution emissions and ecological and social benefits, etc. The conclusion proves that compared with traditional housing, prefabricated housing make a more significant energy savings, in which the using and maintenance phase is the most. Models are convenient to the evaluation of building eco-efficiency and promote the construction of low-carbon energy. However, the carbon emission calculation models still have many areas could be improved, which need researchers in multidisciplinary field to make more exploration and innovation.

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Analysis of the prefabricated building construction cost

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ABSTRACT: The implementation of prefabricated building is beneficial to promote the healthy development of the modern construction industry and sustainable development of the industry. As sample to XX Community in Shenyang, this paper analyzed the existing state of the cost of the development of prefabricated buildings, and emphatically discussed the cost structure of technical-economic indicators between prefabricated buildings and cast-in-place buildings, it is concluded that the high cost is the “bottleneck” in the development of prefabricated buildings, and it found that construction costs is the main reason for the high cost of prefabricated building. Aiming at this problem, this paper puts forward some measures to solve this problem, such as reforming of the production enterprises of PC components tax, improving the qualification administration of PC components enterprise, using Rat-Trap Bond, various integration and utilization and mixing frame structure system of seismic prefabricated building.

1 INTRODUCTION

Prefabricated building is the special construction of technical system that using proprietary connection technology all kinds of general precast concrete promote to factory production and mechanized assembly in the spot (Tianhua et al. 2012), the production, logistics and on-site of the components which fully reflects the characteristics of industrialization of modern architecture. Shenyang was approved becoming the nation’s first modern architecture industrialization demonstration city in April 2014. Related to the opportunity that Shenyang created demonstration city by competent department of construction, the work promoting the modern construction industry encouraged by the government transform into encouraged by the market spontaneous, and the project invested priority by the government extend to a more extensive commercial housing construction transformation. And the realization of the goal that the market spontaneous guide depends largely on certain advantages of prefabricated building engineering cost. At present there is no unified norms and standards of the construction project cost and how to do cost accounting. Researchers need a lot of instances to perfect the cost accounting method and system, in order to improve policies becoming more scientific and effective.

Using a public 3 # housing project in Shenyang as an example in the paper, it is concluded that the high cost is development “bottleneck” of

prefabricated building engineering, and high cost of civil engineering is the main reason. Research results will guide the attention of related construction industry management department from prefabricated building engineering overall cost to the civil engineering cost. Civil engineering is an important part of construction projects, reducing civil engineering cost is advantageous to the promotion of prefabricated building.

2 PREFABRICATED BUILDING COST COMPARED WITH THE COST OF CAST-IN-SITE TYPE BUILDING: TAKING 3 # AS AN EXAMPLE

According to project bidding documents and drawings of a public 3 # housing project in Shenyang, calculate the cost data of prefabricated building in the project, and calculate the civil engineering cost in accordance with the relevant drawings of 3 # cast-in-site type building, comparative data as shown in Table 1 (Shenyang architecture university team, 2012). According to Table 1, the cost of every square meter of prefabricated building is 738.78 yuan higher than cast-in-situ type building, and the high engineering cost of prefabricated building hinder the market promotion of prefabricated building engineering (Li 2013).

Including the proportion of the civil engineering part of the total project cost is increased from 81.08% to 88.01%, and the civil engineering cost

Table 1. Cost data accounted for separately of a public 3 # housing project building floor area: 4714 m².

Project name	Prefabricated building		Cast-in-site type building	
	Square meter cost (yuan/m ²)	Proportion	Square meter cost (yuan/m ²)	Proportion
Civil engineering	2,128.72	88.01%	1,362.13	81.0%
Decoration engineering	149.05	6.16%	156.33	9.31%
Electric engineering	81.77	3.38%	73.13	4.35%
Heating engineering	28.57	1.18%	42.08	2.50%
Water supply and drainage engineering	30.6	1.27%	46.32	2.76%
Total	2,418.71	1	1,679.99	1

of prefabricated building is 766.59 yuan higher than the cast-in-site type building. Prefabricated building can reduce the waste in economy and resource. Therefore research in exploring how to reduce civil engineering cost of prefabricated building is a problem to be solved.

3 ANALYSIS OF PREFABRICATED BUILDING CIVIL ENGINEERING COST

There are certain differences in civil engineering cost between prefabricated building and cast-in-site type building, and both indicators of civil engineering cost are shown in [Table 2](#).

The subject of prefabricated building mainly is PC components, including sandwich wall, prefabricated beam plate column and composite floor slab. The price of PC components is high because the professional installation increases the cost of the installation. The cost of earth and rock engineering, pile and foundation engineering, laying project, cast-in-situ concrete (including template engineering), roofing and waterproofing project, heat preservation and heat insulation engineering, ground floor engineering, plastering engineering, measure program, super-high building artificial, mechanical effect and pump pressure affecting and tower crane foundation and others are reduced.

Comb structure and change of civil engineering cost, we can found that:

1. Earth and rock engineering: Including main construction process of field leveling, foundation pit and groove excavation, transportation and filling and compaction, also including foundation pit precipitation, water drainage, soil wall and other auxiliary projects. As to 3#, as a result of the construction scheme of prefabricated building is improved than the construction scheme of prefabricated building, thus leading to slightly reduce the square meter cost.
2. Pile and foundation engineering: Prefabricated building and cast-in-site type building all adopt the method of pile and foundation engineering. Due to the overall performance of prefabricated building is better than that of cast-in-site type building, when in the pile and the design of pile and foundation engineering, it can also meet the design requirements when being appropriate to reduce the usage of concrete. So its square meter cost reduced 28.41 yuan compared with cast-in-site type building.
3. Laying project: Prefabricated building project has been build with small hollow block wall, brick masonry and gutter piecemeal, leading to its quantities less than the quantities of cast-in-site type building project. Masonry work, under the condition of the comprehensive unit price without much difference, the square meter price of prefabricated building construction per was 5.11 yuan lower than the cast-in-place style buildings.
4. Roofing and waterproofing project: The component in the Roofing and waterproofing project of Prefabricated building which uses the composite floor slab prefabrication has a good integrity. Waterproof performance of the project can also be improved. By using this prefabrication, the amount of roofing and waterproofing can be reduced, and make its cost reduce to 6.65 yuan per square meter.
5. Heat preservation and heat insulation engineering: Cast-in-site type building uses the exterior insulation that the heat preservation material in the outer wall of the lateral; Prefabricated building exterior wall thermal insulation engineering USES a sandwich insulation board (also known as a sandwich wall plate), which is set to bearing, palisade, insulation, waterproof, fire prevention, and other functions as one of the important prefabricated concrete casting. The cladding, extruded board and inner panel are connected by the chain structure

Table 2. Comparing indicators of civil engineering cost.

Serial number	Civil engineering	Prefabricated building	Cast-in-site type building
		Square meter cost (yuan/m ²)	Square meter cost (yuan/m ²)
1	Earth and rock engineering	8.73	16.09
2	Pile and foundation engineering	152.12	180.53
3	Laying project	0.97	6.08
4	Cast-in-situ concrete (including form work engineering)	299.35	757.67
	PC components and installation	1,435.02	
5	Roofing and waterproofing project	20.65	27.3
6	Heat preservation and heat insulation engineering	12.54	78.58
7	Ground floor engineering	13.25	44.77
8	Plastering engineering	71.1	79.79
9	Measure program	65.27	84.33
10	Super-high building artificial, mechanical effect and pump pressure affecting	10.83	29.14
11	Tower crane foundation and others	38.88	57.85

prefabrication, and effectively assembled through local cast-in-situ and steel sleeve connection. Compared to cast-in-place type, this kind of architectural design can not only enhance the service life of the thermal insulation material, but also strengthen the heat preservation effect, make the whole heat preservation function get obvious promotion. The cost of Sandwich panels accounted to PC component, makes the cost of heat preservation, heat insulation engineering of prefabricated one-piece construction 66.04 yuan per square meter lower than cast-in-situ type building.

6. Ground floor engineering: 3 # assembled monolithic construction uses the composite floor slab. The floor consists of prefabricated sections and cast-in-situ sections. Prefabricated parts play the role of the template in the construction, and form as a whole with cast-in-place parts after the completion as a part of the fabricated structure system. The cost of prefabricated composite floor belongs to the PC components and installation costs. The use of precast floor slab makes the usage of cast-in-situ concrete is about 40% less. The prefabricated building floors engineering make cost down 31.52 yuan per square meter.
7. Plastering engineering: PC components of prefabricated building keep the surface smooth, flatness is better than that of cast-in-place style buildings in the process of production in the factory, at the same time, the components that has defects will be repaired after production. And installed components are only need some simple repair in the process of construction.

- General plastering engineering should be layered construction that consists of the bottom, middle, and the surface. The leveling jobs of prefabricated building mainly include the metope inside 10 mm thick anti-crack mortar leveling and the metope with cement mortar. 3 # precast can reach the rate of 65%. The cost of plastering engineering per square meter is about 35% of the cast-in-situ type construction in theory, which is 27.93 yuan per square meter. From the table, the cost of prefabricated building's plastering engineering is 71.1 yuan per square meter, higher than the theoretical value of 43.17 yuan. The main reason for the higher cost is the domestic production of PC components are not standardized process and construction management system is not sound.
8. Measure program: Measures project cost mainly includes the cost of cast-in-situ concrete beam, plate, stair and wall panel with template and support. After using the PC components, prefabricated building projects reduce the quantity of template and support, templates, dismantling and scaffolding greatly in the process of construction site (Li 2013). Due to prefabricated rate of prefabricated building is low, prefabricated building still need to compute the cost of template and scaffolding that are saved. It makes the project per square meter cost of measures program won't have too big difference.
 9. Super-high building artificial, mechanical effect and pump pressure affecting: Prefabricated building reduce the site construction personnel in construction process and the number of hoisting machinery and other machinery, the

decrease cost of labor and machinery reduces the cost of artificial drop cost-effectiveness. Finally it makes per square meter cost reduce 18.31 yuan compared with cast-in-place style buildings.

4 ANALYSIS REASON OF HIGH CIVIL ENGINEERING COST AND COUNTERMEASURE

4.1 Analysis reason of the high civil engineering cost

The composition of civil engineering cost of pre-fabricated building project was shown in Table 3.

The Table 3 shows that PC components and installation costs and cost of cast-in-place reinforced concrete are the main part of the construction cost as to the prefabricated building. PC components and installation cost is 1435.02 yuan/m², and this

Table 3. Comparing civil engineering cost indicators of prefabricated building.

Engineering name	Square meter cost (yuan/m ²)
Earth and rock engineering	8.73
Pile and foundation engineering	152.12
Laying project	0.97
Cast-in-situ concrete (including form work engineering)	299.35
PC components and installation	1435.02
Roofing and waterproofing project	20.65
Heat preservation and heat insulation engineering	12.54
Ground floor engineering	13.25
Plastering engineering	71.1
Measure program	65.27
Super-high building artificial, mechanical effect and pump pressure affecting	10.83
Tower crane foundation and others	38.88

part of the cost includes PC component prices, supporting related cost, hoisting cost, transportation cost of components, etc.

PC components (excluding installation) summary is shown in Table 4, and the project # 3 PC component price is 6242499.4 yuan, and the per square meter is about 6242499.4/4714 = 1324.24 yuan/m², and installation of the PC components cost is 1324.24/1435.02 = 92.28%, and other cost is 1435.02–1324.24 = 110.78 yuan/m².

Through the analysis and calculation can be concluded that the proportion of PC components and installation cost is larger, its proportion reaches 92.28%, and the price is 1324.24 yuan/m². It has been close to 1362.13 yuan/m² which is the civil engineering cost of cast-in-site type building. The high cost of PC component and installation is the main reason for the civil engineering cost of prefabricated building on the high side.

In addition, as to plastering engineering, due to the domestic production of PC components are not standardized process and construction management system is not sound, its cost is on the high side. By some measures to improve, the project cost has a reduction space of 71.1–27.93 = 43.17 yuan/m²; As to measure project, the cost still has a reduction space of 65.27–53.21 = 12.06 yuan/m², because of lower prefabricated rate and construction management mode.

From what has been discussed above, the main reason of high civil engineering cost includes both management and technical aspects:

- 1. Management aspect: The government promotion of PC component and its related business management are not strong enough; Government tax the pre-casting PC component enterprises carrying out in accordance with the industrial enterprises; For enterprise qualification administration of PC components, the PC member enterprise contracting scope in the production of PC components, which were not high cost with steel structure on the installation of the PC components and the

Table 4. The summary of PC components (excluding installation).

Serial number	Location name	Measuring unit	Concrete	Comprehensive unit price	Full cost price
1	320 external wall	m ³	776.04	3596.47	2791002.02
2	200 interior wall	m ³	584.56	3280.67	1917759.61
3	100 interior wall	m ³	218.4	1859.78	406171.49
4	Composite plate	m ³	237.17	4302.36	1020387.07
5	Stairs	m ³	19.43	3024.05	58742.17
6	Special-shaped beam	m ³	5.37	4329.94	23246.37
7	Balcony slab	m ³	7.39	3407.83	25190.68
Total	—	—	1848.36	—	6242499.4

construction curtain wall project. All of these restrict the development of some PC components enterprise (Jiang 2011).

2. Technology aspect: The backward technology, production technology, and inappropriate cost calculation rules are reason of high cost; Construction area wastes because of the lack of improvement for the traditional construction technology.

4.2 *The recommendations about reducing the civil works costs of assembly buildings*

1. Management Measures

From the aspect of management, the government's main administration plays an important role to reduce the civil engineering costs of prefabricated buildings.

- ①. The government coordinates the implementation of the work in prefabricated building industry, gives some related policies to the development, plans for the operation of fabricated market, and is responsible for construction investment and responsible for the demonstration project. According to the relevant provisions, the State Assembly building can also apply for special support fund to remote energy saving ideas, and you can build prefabricated building affordable housing. These management practices of government provides system guarantee to reduce the cost of a prefabricated building. It can also reduce costs by some other management measures.
- ②. The Government Reform on PC component manufacturers taxation taxes, and cancel or reduce of PC components in the transfer part of the tax. PC member companies currently operate specific construction by construction enterprises, but the state of prefabricated enterprises shall be carried out in accordance with the industrial enterprise of VAT tax. This makes the current industry rate be about 6% to 8%, twice than the fixed rate of 3.4% in construction enterprises. From the perspective of industry management. It is also the objective requirements of construction management at the present that the position of member companies is shifted from the industrial enterprises into a professional contracting construction enterprise (Shenyang architecture university team, 2012).
- ③. Improve the management of PC component business qualification. In the current "construction enterprise qualification regulations", PC member enterprises belong to professional contracting business qualification. It can only contract the projects of the extent precast manufacturing production and it is blanking

installation member. It makes the installation part discontinuous, resulting in a series of adverse effects. It is the key reason for the overall image of the industry and ability getting worse, and also seriously hampered the development of large enterprises at the same time.

- ④. It also play a role in reducing costs that introducing, digesting and absorbing foreign advanced technology systems and management models, and promoting the design of prefabricated building construction general contracting model.

2. Technical measures

- ①. For the assembly building, you can use various types of integration technology, and then complete sets (Feng 2009). The so-called integrated technology that two or more single technology are recombined to become a unified whole function of new technologies in accordance with certain technical principles or functional purposes. Complete residential technology, such as engineering, construction typically encountered in water, exhaust duct, lightweight partition, building envelope insulation and other specialized construction techniques. It not only can improve quality, increase productivity, and effectively reduce the construction costs.
- ②. For the construction of the wall, It can use fewer bricks, reduce costs, while increase the area of usage that using cavity wall puzzle together (Rat-Trap Bond). In addition, the new hybrid seismic whole assembly frame structure will also have a place in China's construction industry.

In addition, increasing the promotion rate of prefabricated buildings also can reduce civil cost. Construction companies can also implement appropriate cost control strategies to control costs. Imposing a quota on the design during the design phase of the project can avoid unnecessary expenses. In the construction phase, implementing dynamic management control of the cost, comparing the goal of the project cost and dynamic cost and dynamic analysis, adjusting for the difference between the two modified construction programs, can effectively control costs (Li 2013).

5 CONCLUSION

In this paper, it drawn by empirical analysis that prefabricated building promotion and development have huge cost "bottleneck" when civil costs is high. Specific measures to reduce the cost of civil engineering from the technical and management aspects are proposed by comparing building engineering data and analysis of the causes of variation.

The study of prefabricated buildings cost can provide reference for decision-making of the relevant departments and corporate, and help actively promote the development of prefabricated buildings featuring modern architecture industry and sustainable development of the industry.

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Comprehensive benefits evaluation of sandwich exterior wall panel

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ABSTRACT: Prefabricated building is in line with the inevitable trend of China's construction industry development which is in a direction of energy saving, carbon reduction, and environmental protection. Prefabricated sandwich exterior wall paneling is the core technique of prefabricated concrete construction, which can effectively improve construction efficiency, shorten the construction period, save resources and protect the environment. Based on an analysis of the relevant literature, here we compare the data related with prefabricated sandwich exterior wall panels, which is taken from the Shenyang Metro Huimin Town project, with that from traditional cast-in-situ construction. Then, we analyze the comprehensive benefits of using sandwich exterior wall panels during the residential construction from the aspects of economic, ecological and social benefit.

1 INTRODUCTION

Prefabricated buildings are the buildings with using function, completed by some or all of the components made in the component precast factory, which are transported to the construction site by the appropriate means, assembled by reliable installation method and installation machine (Li 2014). Prefabricated buildings have the advantages of energy saving, low carbon, environmental protection, etc, and come up with the requirements of the transformation and upgrading of China's construction industry (Zhang 2015). Prefabricated sandwich exterior wall paneling is the core technique of prefabricated concrete construction, fully presenting the characteristics and advantages of prefabricated buildings, having more comprehensive benefits than using traditional construction methods.

2 DEFINITION OF PREFABRICATED SANDWICH EXTERIOR WALL PANEL AND ITS COMPREHENSIVE BENEFITS

2.1 *Introduction of prefabricated sandwich exterior wall panel*

Precast concrete (PC component for short) refers to the concrete product which is manufactured and finished in factory, with a standardized design and a construction mechanization, etc. It is a major component of prefabricated buildings. Prefabricated sandwich exterior wall panel is one of precast concrete wall panels, which can make structure layer, insulating layer, and finish layer connected as

a whole in the factory. So it is not necessary to re-create the insulating layer or finish the layer at the construction site. Thus it can greatly shorten the construction period, improve construction speed and reduce labor costs.

2.2 *The technological process of making prefabricated sandwich exterior wall panel*

The technological process of making prefabricated sandwich exterior wall panel is shown as Figure 2.

2.3 *Definition of comprehensive benefits of prefabricated sandwich exterior wall panel*

Prefabricated sandwich exterior wall panel is the core technique of prefabricated concrete construction,

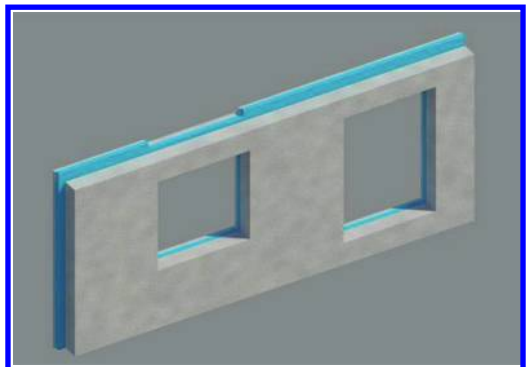


Figure 1. Prefabricated sandwich exterior wall panel.

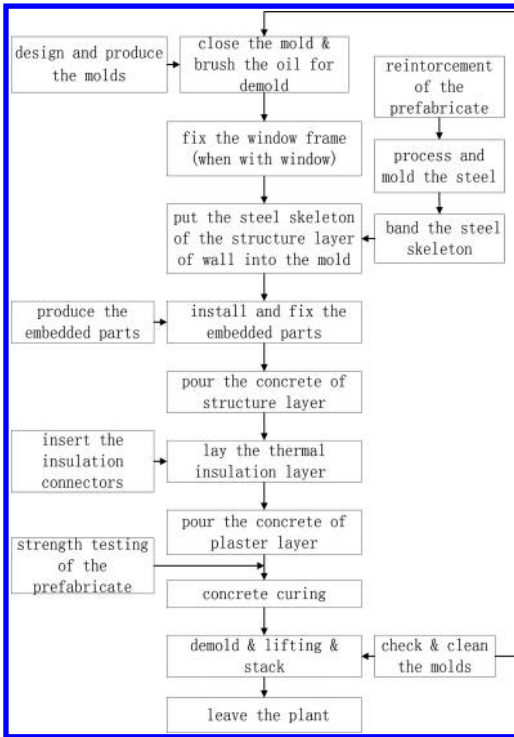


Figure 2. The technological process of making prefabricated sandwich exterior wall panel.

fully demonstrating the characteristics of prefabricated buildings. Currently, many scholars have studied and analyzed the benefits of prefabricated sandwich exterior wall panel. For example, Zhang (2013) decomposed the construction technology of PC exterior wall panels, analyzed the economic benefits of PC exterior wall panel with actual case. Tang (2013) compared the production and lasting effects of prefabricated exterior wall panels with traditional wall and made a conclusion that prefabricated wall can improve construction efficiency, conserve resources, reduce energy consumption, reduce environmental pollution and so on with qualitative analysis. Yu et al. (2013) analyzed the energy consumption of prefabricated concrete exterior wall panels and aerated concrete panels, made a conclusion that prefabricated sandwich exterior wall panel is more energy-efficient, and has better development prospects. Building's comprehensive benefits refer to all the effects and benefits which acquired after the completion of a building (or structure), it is an important index for building integrated situations. Therefore, we believe that all of these benefits' studies belong to the scope of the comprehensive benefits of prefabricated sandwich exterior wall panel.

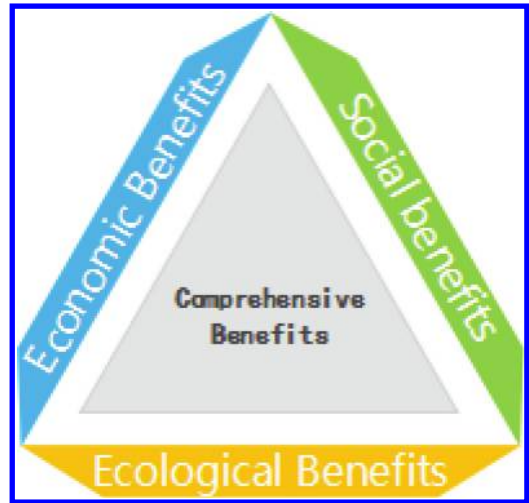


Figure 3. Diagram of comprehensive benefits.

In order to fully and accurately analyze the comprehensive benefits of prefabricated sandwich exterior wall panel, we read a lot of literature, and redefine the comprehensive benefits of prefabricated sandwich exterior wall panel, decompose it into three aspects which are economic, ecological and social benefits, card and integrate views of previous studies and select the appropriate evaluation. In addition, previous studies are mainly based on qualitative analysis. Now from the integration of qualitative and quantitative methods, we account the related data and compare the analysis of using prefabricated sandwich exterior wall panels and of traditional construction methods in a different way, and complete the analysis of comprehensive benefits of prefabricated sandwich exterior wall panel by the methods on the basis of the real data of Shenyang Metro Huimin Town project.

3 PRESENTATION OF SHENYANG METRO HUIMIN TOWN PROJECT

Shenyang Metro Huimin Town public rental project (referred Huimin Town project) is located in a junction of Daoyi Four village and Gong Jiangtun village in Puhe Town, Shenyang. The total construction area is about 110,000 square meters, with 22 12-storey high-rise. The project can accommodate 2,000 households in the project design. The project is developed by Shenyang Metro Real Estate Development Co., Ltd. It is using prefabricated buildings construction method with the prefabricated concrete elements

provided by Liaoning Yuhui New Building Materials Co., Ltd.

Huimin Town project uses total prefabricated construction method, including prefabricated sandwich exterior wall panels, prefabricated interior wall panels, prefabricated laminated floors, stairs and other prefabricated, the precast rate is about 65%. According to the project's bidding documents, tender documents and drawings, we calculate and collate the cost, energy consumption, water consumption and other data about building 3 # (Construction area: 4713.56 m²), and calculate the relevant data which is in traditional cast-in-situ construction way with the drawings. The precast rate is about 30% when only use the prefabricated sandwich exterior wall panels (Li 2015), we can obtain the data of comprehensive benefits of prefabricated sandwich exterior wall panel by calculating the ratio of precast rate.



Figure 4. The aerial view of Shenyang Metro Huimin Town projects.



Figure 5. The plan of Shenyang Metro Huimin Town projects.

4 ANALYSIS AND ACCOUNTING OF THE ECONOMIC BENEFITS OF SHENYANG METRO HUIMIN TOWN PROJECT

Economic benefit is the savings of social labor from the exchange of commodity and labor. It means to minimize the cost of labor to maximize the business results, or get more business results in the same labor-consuming condition. We calculate the cost of Huimin Town project, getting the result that, the construction way of using prefabricated sandwich exterior wall panel costs 200 yuan/m² more than traditional construction way due to the high cost of PC components (Li 2014). However, in view of the whole life cycle, the construction way of using prefabricated sandwich exterior wall panel will just be slightly higher than the cost of traditional cast-in-situ construction way, thus the former has great economic benefits (Research Group of Shenyang Jianzhu University 2015).

4.1 Reduction of construction cost

Prefabricated sandwich exterior wall panel is made in one-time molding inside the factory. It is not necessary to re-create the insulating layer or finish the layer at the construction site. Prefabricated sandwich exterior wall panel can greatly shorten the construction period. By accounting the relevant data of Huimin project, using Prefabricated insulated sandwich wall panel can cut about 15% of construction period. The economic benefit of construction period of the project is about 25 yuan/m². At the same time, due to the cut of the template support, pouring concrete and other construction processes at the construction site, the template and labor costs are reduced, so that the construction cost will be reduced.

4.2 Reduction of maintenance cost

According to the required useful life requirements of 25 years of the insulation, the insulation of ordinary residential exterior wall made in traditional cast-in-situ construction way needs to be redone after 25 years. While the insulation of prefabricated sandwich exterior wall pane will not. The insulation can be used in the whole life cycle. Thus it saves the cost about 70 yuan/m² which is for the second insulation system. And other maintenance cost will also be reduced about 5 yuan/m². In summary, using prefabricated sandwich exterior wall panel can increase the service life and reduce maintenance costs later.

4.3 Improvement of the construction quality

Compared with traditional cast-in-situ construction way, using prefabricated sandwich exterior

wall panel can effectively reduce the accuracy of structure deviation, basically eliminate the common quality issues, such as leakage, cracking, empty drum, room size deviation, and improve comfort level of the interior space significantly. Through calculation, the use of prefabricated sandwich exterior wall panel can save construction cost about 12 yuan/m² in avoiding quality problems and improving the quality of residential construction.

5 ANALYSIS AND ACCOUNTING OF THE ECOLOGICAL BENEFITS OF SHENYANG METRO HUIMIN TOWN PROJECT

Ecological benefit refers that people foster the nature biological systems be more beneficial to human's production, living conditions and environmental conditions according to the ecological balance, it is related to fundamental interests and long-term interests of humanity's survival and development. Ramesh et al. (2012) made life cycle energy analysis of different types of residential buildings (one storey, two storey, and duplex and multi storey) in Indian context. Buildings demand energy in their life cycle right from its construction to demolition. Ramesh et al. (2010) made a conclusion that building's life cycle energy demand can be reduced by reducing its operating energy significantly through use of passive and active technologies even if it leads to a slight increase in embodied energy. With the development of rapid economic, higher income levels, urbanization and other socio-economic drivers, people's lifestyles in China have changed remarkably over the last 50 years. Feng et al. (2009) used the IPAT model to analyze how these main drivers contributed to the growth of CO₂ emissions over this time period. The study emphasized the need to guide people's lifestyles towards more sustainable ways of living. These studies have shown that energy-saving and environmental protection is increasingly important for social development. Prefabricated buildings can save energy efficiently. Using prefabricated sandwich exterior wall panel particularly reflects good benefits of energy saving from every phase of its life cycle. For example, manufacturing stage, construction stage and the usage phase. Ecological benefit of prefabricated sandwich exterior wall panel mainly includes the "A green four sections" in five areas, which include energy saving, land saving, water saving, material saving and environment protection.

5.1 Energy saving

Using prefabricated sandwich exterior wall panels can save energy, which is mainly reflected in

two aspects. Firstly, to use prefabricated sandwich exterior wall panels can significantly reduce the template support, concrete pouring and other processes in construction site, as well as reduce the use of vertical transportation and concrete vibrators and other energy-consuming equipment. The main mechanical equipment of the external wall construction of the Huimin project and the use of electric power are shown in Table 1. Secondly, to use prefabricated sandwich exterior wall panels can reduce the number of workers on-site, reduce power and energy consumption. Calculation result shows that using prefabricated sandwich exterior wall panels in Huimin Town project saved electricity 2371.28 kW, saved a total of 2252.72 yuan according to the current electricity charges of 0.95 yuan/kwh in Shenyang.

5.2 Land saving

Since the prefabricated sandwich exterior wall panel is produced in factory, which can reduce the amount of steel, concrete, insulation board and other materials in construction site, and size down the warehouse, materials yard, plants and other temporary construction site. So that the on-site construction space can be cut down and the layout of construction site will be more scientific and rational. In addition, prefabricated sandwich exterior wall panel will not be piled in a large scale and take up too much place since it can be transported from off-site as the construction schedule.

5.3 Water saving

Saving water can also be analyzed by construction water and domestic water. Using prefabricated sandwich exterior wall panel can reduce the quantity of wet operations in construction site, reduce the quantity of concrete mixing water and

Table 1. Electricity consumption of construction machinery and equipment (Unit: kW).

No.	Name of machinery or equipment	Specification	Power
1	Elevator	SCD200/200AJ	2 × 10.5
2	Vibrator	ZN42	1.2
3	Bar cutter	GQ40F	3
4	Steel bar bender	GW40D	3
5	Steel-bar straightener	LGT6/14	15
6	Steel-bar butt welder	UN1-75	75.0 KVA
7	Welder	BX10-500	40.5 kVA
8	Mortar mixer	UJZ325	3
9	Concrete mixer	JZ350	5.5

conservation water and other water in the scene. It can also reduce the number of workers in field operations, therefore living water is saved. As it is shown in Table 2, when using traditional cast-in-situ construction way, Huimin Town project needs construction water 77.9 kg/m³, domestic water 1300 kg/m³, while using prefabricated sandwich exterior wall panel needs construction water 68.5 kg/m³ and domestic water 910 kg/m³. According to the current price of construction water in Shenyang is 3.85 yuan/m³, domestic water price is 2.4 yuan/m³. After accounting, using prefabricated sandwich exterior wall panel can make the project reduce the water of construction 9.4 kg/m³, reduce the domestic water 0.39 kg/m³, using prefabricated sandwich exterior wall panel will save the cost about 0.97 yuan/m² in reducing water consumption in Huimin Town project.

5.4 Materials saving

Using prefabricated sandwich exterior wall panel can reduce the amount of pouring concrete, templates and other materials on-site. At the same time, due to the prefabricated sandwich exterior wall panel is completed in the factory, the flatness of the surface of the wall panels is apparently better than that made in traditional cast-in-situ construction way, and it reduces the amount of plastering mortar and other materials. The calculation shows that, using prefabricated sandwich exterior wall panel can save template 2993.47 m² in Huimin project. It equals to 53882.46 yuan according to the template is 18 yuan/m² in market prices.

5.5 Environmental protection

The use of prefabricated sandwich exterior wall panel can effectively protect the environment, mainly reflected in the reduction of dust emissions, construction noise, construction waste and other aspects.

Table 2. The comparison of water consumption in traditional cast-in-situ construction way and prefabricated construction way of Huimin project (Unit: kg/m³).

Item	Water for construction	Water for life
Traditional cast-in-situ construction way	77.9	1300
Total prefabricated construction way	57.6	455
Prefabricated sandwich exterior wall panel construction way	68.53	910

Table 3. The calculation of construction waste reduction (Unit: t/m²).

Item	Construction waste of the main construction	Construction waste of decoration
Traditional cast-in-situ construction way	0.005	0.0314
Total prefabricated construction way	0.03	0.1
Reductions	0.025	0.0686
Reduction rate	83.33%	68.6%

Coal, transportation and dust are the three main factors of air pollution in our country, the construction dust accounts for about 14% of the total dust emissions (Research Group of Shenyang Jianzhu University 2014). The prefabricated rate can reach 30% when using prefabricated sandwich exterior wall panel. According to the proportion, using prefabricated sandwich exterior wall panel can reduce about 30% of total construction dust emissions.

Building construction is one of the major sources of life noise. The exterior wall construction will generate noise too, which comes mainly from the steel processing, template support, removal, knocking, concrete mixing, sound of vibration and other machinery. The intensity of these noises is mostly in 80~90 dB. The use of prefabricated sandwich exterior wall panel will effectively avoid the noise.

Using prefabricated sandwich exterior wall panel can avoid the waste of materials, which is caused by the inaccurate calculation of the amount of steel, concrete, insulation materials and other materials as well as the remaining reject material after cast construction. Huimin Town project saves about 2 yuan/m² for reducing the construction waste after calculation. In addition, prefabricated sandwich exterior wall panel can be recycled effectively when building demolished, so that reduced a large amount of construction rubbish.

6 ANALYSIS AND ACCOUNTING OF THE SOCIAL BENEFITS OF SHENYANG METRO HUIMIN TOWN PROJECT

Architecture's social efficiency refers to the contributions to the society after the project implementation. The construction method of using prefabricated sandwich exterior wall panel can improve the efficiency of construction workers, and improve the working environment, promote the economic development and provide convenience

for many of the construction project stakeholders. It has significant social benefits.

6.1 Establishment of social benefit evaluation system of prefabricated sandwich exterior wall panel

In this paper, according to the characteristics of prefabricated sandwich exterior wall panel, we select the main factors of the influence on the society of the prefabricated buildings, establish the social benefit evaluation system of prefabricated sandwich exterior wall panel, and develop the evaluation index of all the influencing factors, as shown in Table 4.

However, due to the relevant indicators of social benefit evaluation is difficult to measure actually, in this paper, on basis of this social benefit evaluation system, according to the actual situation of Shenyang Huimin Town project, we analyze the social benefits of prefabricated sandwich exterior wall panel mainly by the way of qualitative analysis.

6.2 Promoting the development of social economic

The social economic influence of using prefabricated sandwich exterior wall panel is reflected in macrocosm and microcosm two aspects.

Table 4. The evaluation system of prefabricated sandwich exterior wall panel's social benefits.

Item	Factors	Evaluation
Social economic	Impact on the national economy	GDP Prefabricated buildings value of the total value Per capita industrial value of prefabricated buildings
	Impact on employment	The total number of employees in the construction industry Employment rate The proportion of employment in the three industries
	Impact on income distribution	The proportion of income distribution among countries, regions, enterprises and individuals
	Impact on the economic life of the residents	The proportion of urban residents' living expenses in total consumption expenditure The proportion of rural residents' per capita net income from the construction The residents income ratio between city and country
	Impact on the relevant interest groups	Manufacturing enterprise Design enterprise Construction enterprise Residential users Construction worker
	Influence of cultural education	Illiteracy rate The per capita level of education The proportion of professional and technical personnel
	Influence of technological progress	Input level of construction industry R & D Contribution rate of technological advancement The number of proprietary technology and patent The number of technical staff in construction Labor productivity in prefabricated buildings
		The degree of conform to the national policies The degree of cultural and technology adapt to project
Social interaction adaptability	Adaptability of social environmental conditions	Support level of the project
	Attitude of interest groups	The proportion of direct interest groups The proportion of indirect interest groups
	Participation of related interest groups	
Social risk	Risk of relevant interest groups	Qualitative description
	Technical risk	Qualitative description
	Policy risk	Qualitative description
	Market risk	Qualitative description
	Management risk	Qualitative description
Social time	Natural and intangible loss	Qualitative description
	Fixed costs	Qualitative description
	Scale of construction	Qualitative description

Macro social economic impact mainly refers to the economic benefits of using prefabricated sandwich exterior wall panel, which can promote the development of the national economy. At the same time, prefabricated construction way is an important foothold in transformation and upgrading of China's construction industry, which can promote industrial restructuring.

Micro social and economic influence mainly includes the following aspects. (1) Using prefabricated sandwich exterior wall panel can expand employment. The development of prefabricated construction industry chain is still inadequate, the construction projects as well as the related facilities provide a lot of jobs for the community, such as Yuhui company needs to hire production and skilled workers to produce prefabricated sandwich exterior wall panel. (2) Using prefabricated sandwich exterior wall panel can improve the working environment. Mechanization replaces the manual operation with the transformation of the construction methods, so that the construction field workers change into skilled workers, production workers in factory. In Huimin project, there are only 15 workers in a working group who is responsible for the construction of a building. (3) Using prefabricated sandwich exterior wall panel can improve the quality of the workers. Workers are asked to improve the knowledge, skills and qualities because of the new technologies and new techniques. Prefabricated Building is bound to promote the construction industry to increase the education and training of workers, promote the overall improvement of the quality of workers. (4) Using prefabricated sandwich exterior wall panel can Promote the progress of science and technology. Prefabricated Buildings Change the production methods of construction industry, using Precast Concrete instead of traditional cast-in-situ construction way, and promote increasing investment in science and technology in the construction industry.

6.3 *Improvement of social interaction adaptability*

Compared to traditional construction methods, the use of prefabricated sandwich exterior wall panel has good economic and ecological benefits, more easily accepted by the stakeholders, and its social interaction adaptability has been improved significantly. Social interaction adaptability mainly embodied two aspects, adaptation to social environment condition and adaptation to interest groups.

Prefabricated Building is in the ascendant and in line with the development direction of China's construction industry. The government introduces policies to support and guide the development

of the prefabricated buildings. And the available technology and cultural conditions can support the development of prefabricated buildings. So, prefabricated buildings can be well adapted to the current social environmental conditions.

Prefabricated Building is beneficial to the development of various interest groups. For prefabricated production enterprise, the production of prefabricated can improve production efficiency and increase profits. For the design corporation, using prefabricated sandwich exterior wall panel can reduce the number of design classification, reduce the duplication of designers, speed up the design progress. For construction enterprise, using prefabricated sandwich exterior wall panel can improve the working efficiency, promote the technology more professional at the construction site, ensure the safety civilized construction and reduce the rate of accident. For construction workers, it can improve the working environment, reduce the operating strength, improve work efficiency, improve the quality of life. For people living around the project, it can reduce noise during construction, reduce environmental pollution, be more easily accepted by the residents around the project.

6.4 *Reduction of the project risk*

In the process of project construction, there are a lot of risks, for example natural risk, social risk, technology risk, economic risk, legal risk, political risk, market risk and management risk. The construction enterprise usually try to shorten the construction period to avoid the occurrence of these risks. Using prefabricated sandwich exterior wall panel can effectively shorten the construction period by about 15%, which can also effectively avoid the risk of extended construction.

6.5 *Saving social time*

Compared to traditional construction methods, using prefabricated sandwich exterior wall panel can greatly improve construction efficiency, shorten the construction period, put the project into production use ahead of schedule, effectively save social time. According to China's basic construction scale in production currently, if the construction period of the project is shortened one year time, the national economy can increase income of 5 billions yuan, while also can absorb more labor employment. Huimin Town project is an affordable housing project, it is urgent and timed. The tenants will be able to live in earlier when shorten the construction period. So that, using prefabricated sandwich exterior wall panels can save the community time effectively.

7 CONCLUSION

This article analyzes the comprehensive benefits of prefabricated sandwich exterior wall panel from aspects of economic, ecological and social benefits three aspects, and concludes that the prefabricated sandwich exterior wall panel which has cut the construction and maintenance costs, improved construction quality and other economic benefits. It conserves land, water, energy and materials, protect environment and has other ecological benefits. It also has the social benefits of promoting social and economic development, improving social interaction adaptability, reducing project risk, saving social time and other social benefits. Through the combination of qualitative and quantitative research, we draw a conclusion that the use of prefabricated sandwich exterior wall panel has significant comprehensive benefits. It is in line with the requirements of transformation and upgrading of China's construction industry, and it will help promote the development of prefabricated buildings. But because this article is only based on Shenyang Metro Huimin Town project, part of the data is not universal, relevant conclusions are still limited. In the future, with the improvement of prefabricated building construction technology and expanding the scope of application, comprehensive benefits of prefabricated insulated exterior wall panel will also increase, and benefiting the larger scope.

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Current state of information exchange between the two most popular BIM software: Revit and Tekla

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ABSTRACT: Information exchange between domain-specific software products is essential for capturing the life cycle information of a building from its inception to operation. To collect all specialized information within a single model requires a platform that is acceptable to all different software tools. Many researchers have suggested that this objective can be approached by using vendor-neutral and open building information models such as the Industry Foundation Classes (IFC) to capture and exchange data. To analyze the effectiveness of the IFC model, this paper presents a data exchange analysis between two of the most popular BIM software to date: Revit and Tekla. Models are exchanged in the two software using IFC and a comparison is made in terms of data loss and misrepresentation. Revit seems to process IFC data easily however minor changes are noticed during the exchange. Tekla needs conversion procedures before IFC data can be processed. The architectural features of the model are found to be seriously misrepresented in Tekla.

1 INTRODUCTION

The building industry today has complex and collaborative environment that requires repeated, iterative data exchanges and communication among different domains and applications in a high frequency. The domain-specific experts usually employ software products that are highly specialized and often provide only very limited means of exchanging data with other software products. To automate information processing, standardized and qualified data is necessary for efficient working processes. BIM (Building Information Modeling) technology has been receiving an increasing attention in the AEC (Architecture, Engineering and Construction) industry (Eastman et al. 2011). Compared with the traditional CAD technology, BIM is capable of restoring both geometric and rich semantic information of building models, as well as their relationships, to support lifecycle data sharing. A growing trend is found in the usability of the data produced by BIM that is produced by many different tools, but the data format is mostly proprietary due its commercial nature (Vanlande et al. 2008). However, some models contain non-propriety building data such as Industry Foundation Class (IFC). IFC was developed by International Alliance for Interoperability (IAI) and is currently known as BuildingSmart.

This papers aims to analyze the current state of data exchange between the two most popular BIM software: Autodesk Revit and Tekla Structures

using the IFC model. The structure of the paper is such that a brief description of BIM and IFC is provided followed by outline of the software tools used for this research. Then the methodology of the research is presented along with the case study followed by results and conclusion.

2 LITERATURE REVIEW

2.1 BIM

BIM is the abbreviation of Building Information Modeling and the initial concept was proposed by Professor Chuck Eastman of Georgia Institute of Technology 35 years ago. However, BIM technology exactly drew attention in 2002; the Autodesk Inc. released BIM white papers, which officially proposed the solution for construction information technology is BIM (Eastman et al. 2008). The National Building Information Modeling Standard (NBIMS 2007) vision for BIM is “an improved planning, design, construction, operation, and maintenance process using a standardized machine readable information model for each facility, new or old, which contains all appropriate information, created or gathered about that facility in a format useable by all throughout its lifecycle.” (National Institute of Building Sciences 2007).

Many researchers have evaluated the effectiveness of Building Information Modeling (BIM) applications within different educational or

industrial settings, BIM is accepted as a process and corresponding technology to improve the efficiency and effectiveness of delivering a project from inception to operation/maintenance. A number of case studies have been published that show useful BIM implementations on actual construction projects including use of proposed use of 3D/4D models for design review from the perspective of constructability, design of a BIM-based game for fire evacuation simulations, 4D visualization technology for safety (Zhou et. al. 2013).

The key concept of BIM is a complete object-oriented digital model of the building that can be reused by different stakeholders at different stages of the building life cycle. Besides the geometric information, other building information that is not available in traditional Computer-Aided Design (CAD) solutions is also stored in the model, including building materials and costs, project specification, and contract information. The stored information can be easily retrieved and reused once inputted into the model, thus eliminating the need to re-input or even recollect information that is valuable in later stages of the building's life cycle. (Zhang et al. 2015).

Building information modeling solutions create and operate on digital databases for collaboration, manage change throughout those databases so that a change to any part of the database is coordinated in all other parts, and capture and preserve information for reuse by additional industry-specific applications. Data integration of the Construction is the basic of BIM, although BIM technology solves how to build information models, a unified standard is also needed to connect the work of information exchange and sharing etc. Therefore, developing general data models or interoperability standards is become to one of the important tasks to realize information integration. (Qing et al. 2014). As a comprehensive international standard for BIM interoperability, IFC is a set of definitions describing the consistent data representation of building components (Liebich 2010).

2.2 IFC

The Industry Foundation Classes is a standard data model that supports a full range of data exchanges among heterogeneous applications. Its schema is developed in the EXPRESS modeling language (ISO 10303-11 1997). The IFC data model allows the building geometry and materials property information to be exported from a BIM authoring tool to a standard format such as the IFC compliant STEP (Standard for Exchange of Product Model Data) physical data file (ISO 10303-21).

The structure of this STEP physical file is such that its extension is “*.ifc” and thereby it is a structured ASCII text file. It has a header section and a data section; the header section contains

information about the IFC version used, the application that exported the file, the date and time when the export was done, the name, company and authorizing person of the file etc. The data section contains all instances for the entities of the IFC specification. These instances have a unique (within the scope of a file) STEP Id, the entity type name and a list of explicit attribute.

The architecture of IFC is such that it has four layers, IFC 2x consists of the (i) Resource Layer, (ii) Core Layer with Kernel & Extensions, (iii) Interoperability Layer and (iv) Domain Layer. A detail is given of these layers in the following paragraph which is adapted from BuildingSmart Chapters (Ch. 9, BuildingSmart, Dhillon et.al. 2014):

- I. Resource Layer: Fundamental concepts expressed as entity types such as geometry (point, line and curve) topology (vertex, edge, face and shell), geometric model (CSG, B-Rep, Geometric Set).
- II. Core Layer: Provides the basic structure of the IFC object model and defines most general concepts that will be specialized by higher layers of the IFC object model. *The Kernel*: Provides all the basic concepts required for IFC models within the scope of the current IFC Release. It also determines the model structure and decomposition. The Kernel can be seen as a template model that defines the form in which all other schema within the model is developed. Its constructs are very general and are not AEC/FM specific. *Core Extensions*: Core Extensions, provide extension or specialization of concepts defined in the Kernel. More specifically, they extend those constructs for use within the AEC/FM industry. Each Core Extension is a specialization of classes defined in the Kernel and develops further specialization of classes rooted in the IfcKernel. Additionally, primary relationships and roles are also defined within the Core Extensions.
- III. Interoperability Layer: This layer defines basic concepts for interoperability between different domain extensions. Shared building elements like beam, door, roof, window or ramp are defined in this layer.
- IV. Domain Layer: Domain Models provide further model detail within the scope requirements for an AEC/FM domain process or a type of application. Examples of Domain Models are Architecture, HVAC, FM, Structural Engineering etc. The primary IFC element hierarchy is based on the accessing structure, Project > Sites > Buildings > Stories > Spaces > Elements.

Various systems can implement converters between the IFC schema and their native data

models to export and import IFC instances to exchange information with each other. This approach has contributed to addressing the problem of data exchange and integration. In addition to the shared IFC data model itself however, the interoperability also depends on two other factors: a) The quality of implementations of IFC export-import converters; b) The quality of the instance modeling of buildings. The respective work is carried out by application implementers and domain end-users alike. (Zhang et al. 2015).

2.3 Software tools

The software tools used to undertake this research are basically of three types. The first type include the BIM authoring tools which are used to produce the design and influence on the instance modeling of the buildings such Revit and Tekla. The second type includes the export-import converters such as TEKLA BIM sight and the third type includes the analyzers such as Solibri Model Checker (SMC) and IFC Analyzer which analyzes the data exchange between the BIM authoring tools. The Solibri Model Checker (SMC) is one of the most widely used checking applications (Eastman et al. 2009). SMC is a powerful checking platform that can also be used to perform e.g. clash detection and code compliance checking. It has already pre-converted some standardized model views and BIM manuals to rule-sets, such as the BIM Coordination View. Other, customized rule sets such as the Dutch Rgd BIM Norm are being implemented upon request. It can also be customized further by some rule configuration tools e.g. its built-in Rule Manager.

3 PROBLEM STATEMENT

Research indicate specific technical issues in IFC-based data exchange, such as geometric misrepresentation, loss of object information, confusion in interdisciplinary revision, application-specific IFC input/output, and large IFC file sizes (Kam & Fischer 2004). (Jeong et al. 2009, Lim et al. 2008, Pazlar & Turk 2008) took a more structured approach to evaluating the technical problems of IFC-based data exchange. These studies conducted visual checks, compared file sizes, and observed changes in a few selective attributes [e.g., a Globally Unique Identifier (GUID)]. If an IFC file is repeatedly imported and exported among different systems, its file size may rapidly increase even if no modification is made to the original model. On the other hand, a wide range of detailed and specific domain information are not explicitly modeled and covered on a schema level. Moreover, many strict data modeling mechanisms on the schema level have side effects that do not

meet the flexible requirements in daily information processing practice. An example is the strict attribute inheritance that would mandate information often too detailed to be provided e.g. in early project phase. As well as existing product model servers are unable to interpret the geometric and topological information that is implicitly or explicitly contained. There are as well problems reported in the interpretation of parametric characteristics of 3D Models. Also the IFC files generated from different systems often contain enormous redundant information, which will greatly limit IFC-based data storage and exchange, management, transmission and other applications (Lee 2011).

4 METHODOLOGY

To address this issue, this paper presents a data exchange analysis between two of the most popular BIM software to date: Revit and Tekla. Two model (one story and two story) are modeled on Revit. [Figure 1](#) shows the methodology flow chart of the undertaken research. Since both the software have their own IFC import-export method; the IFC files are imported and exported using the native tools. An alternate method of data exchange is also adopted using the additional feature of Tekla BIM sight which is able to import an IFC file and save as Tekla BIMsight project file (*.TBP) which

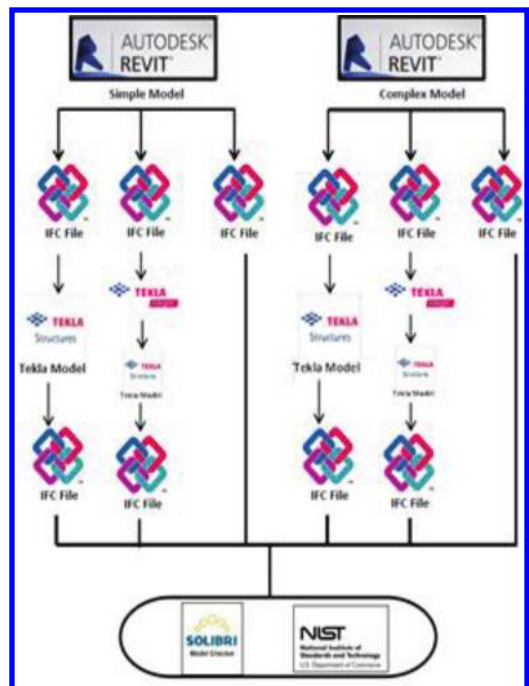


Figure 1. Methodology flow chart.

can be imported into Tekla Structures as reference model.

These files are then viewed and analyzed in the IFC checkers and analyzers like Solibiri Model checker and IFC Analyzer. The IFC files produced for both the buildings are compared in terms of: (1) physical file-size; (2) differing numbers of instances; (3) inconsistent object types; (4) inconsistent attribute values (missing or new values, loss of numerical precision, string length differences, value differences, reference number differences, etc.); and (5) schema inconsistencies.

5 CASE STUDY

Two case studies are conducted for this paper. First a one room structure of 50 sq. feet is modelled in Revit. The structure contains four wall, two slabs, four columns, two windows and door. Figure 2 shows the revit model for the structure.

This revit model is exported in IFC format using the Revit Alternate User Interphase for IFC for IFC 2X3 coordination view 2.0. Table 1 shows the summary information of the IFC file extracted using IFC File Analyzer. Originally 6 building elements were modelled however IFC export added two more objects namely IfcOpeningElement for windows/doors and IfcCovering for ceiling.

This IFC file was imported in TEKLA as a reference model. Figure 3a, 3b and 3c below shows the import procedure and object conversation in TEKLA.

From Tekla the converted objects were exported as IFC file. IFC Log report was generated to identify the converted elements and the status of conversation. Part of the IFC conversion report is shown in Figure 4. The Class column represents the adequacy of the information available to convert into native objects. The class value ranges from 990 to 996. 990 depicting that there is enough

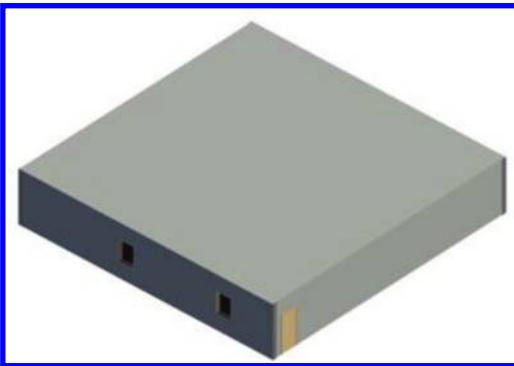


Figure 2. Single story Revit model.

Table 1. IFC export summary information.

Parameter	Value
Revit model size	2.56 MB
Physical IFC file size	37 KB
Number of entities	588
Object types modeled	6
Object types IFC	8
IFC export method used	20140606_1530(x64)— Exporter 15.4.0.0— Alternate UI 15.4.0.0
Schema name	IFC2X3

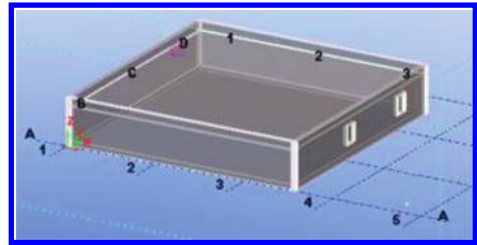


Figure 3a. Revit model imported in Tekla as reference model.

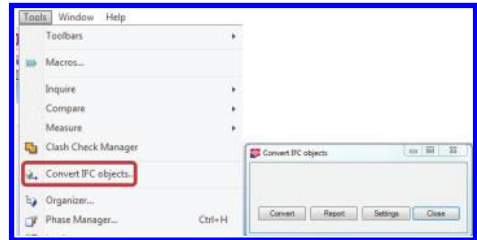


Figure 3b. Tekla native object conversion procedure.

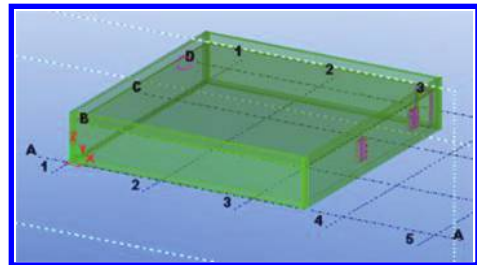


Figure 3c. Model converted in native Tekla elements.

information in IFC model to convert the objects successfully and 996 showing that there is limited information to convert objects and is actually converted from B-rep (Boundary Representation) object type.

Profile	Initial profile	Class
304.8*15037	GENERIC-12"	990
787.4*12.7	36"X48"	990
787.4*12.7	36"X48"	990
914.4*50.8		990
3048*203.2	GENERIC-8"	990
3048*203.2	GENERIC-8"	990
3048*203.2	GENERIC-8"	990
3048*203.2	GENERIC-8"	990
15036.8*15036.8	2'X4'ACTSYSTEM	990
HD260*73	W10X49	990
HD260*73	W10X49	990
HD260*73	W10X49	990
HD260*73	W10X49	990
36" X 48"		992
36" X 48"-1		992
36" X 48"-2		992
36" X 48"-3		992
36" X 48"-4		992
36" X 48"-5		992
36" X 84"		992
36"X84"	36"X84"	992

Figure 4. Part of the Tekla IFC object conversion report.

Table 2. IFC export summary information.

Parameter	Value
Tekla model size	796 KB
Physical IFC file size	70 KB
Number of entities	988
Object types modeled	8
Object types IFC	4
IFC export method used	Tekla Structures Educational 20.1 Service Release 2, IFC Export Version: 4.0.0.0 Jan 14 2015
Schema name	IFC2X3

Table 2 shows the summary information of the IFC file extracted using IFC File Analyzer. The IFC file exported from Revit had 8 object types which now reduced to 4; as the frame/panels for doors/ windows were converted in Beams and the ifcOpening was discarded.

An alternate method was also employed to import the IFC file through Tekla BIMsight but there was no difference found in any of the procedure followed for this case. Finally both the IFC files were loaded in Solibir Model Checker (SMC) for comparison. SMC has a very powerful rule template named Model Comparison (Rule Template # SOL/206) that is able to compare two versions of an IFC model for differences. The final report for the model comparison is summarized in Table 3.

The second case study consisted of a two storey building having gross area of 3200 sq ft. The building has Slabs, Walls, Stairs, Railings, Furniture, Openings and Doors. The revit model of the building and its section is shown in Figure 5a and 5b.

Table 3. Model comparison report.

Location	Component	Type	Δ Count	Δ Length	Δ Area
Floor 0	Beam	36" X 48"		0.15	
		36" X 48"-1		0.46	
		36" X 48"-2		0.21	
		36" X 48"-3		0.15	
		36" X 48"-4		0.46	
		36" X 48"-5		0.21	
		36" X 84"		0.08	
		36"X84"		0.08	
		Column		0.19	
		787.4*12.7		7.17	
		914.4*50.8		7.00	
		HD260*73		40.00	
	Door	36" x 84"	-1		
		Slab			2433.83
		Generic - 12"			-2433.83
		Suspended 2' x 4' ACT System			-2433.83
	Wall	3048*203.2			1959.14
		Generic - 8"			-1959.14
	Window	36" x 48"	-2		
Level 2	Column	W10X49		-40.00	

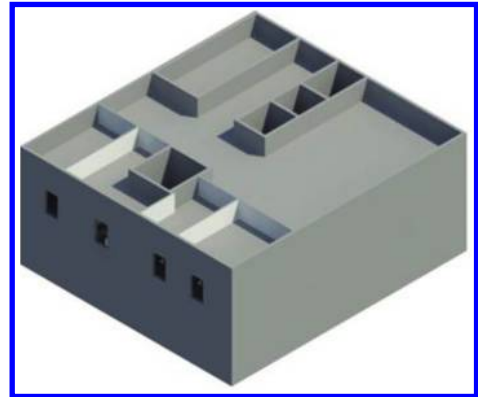


Figure 5a. Two story Revit model.

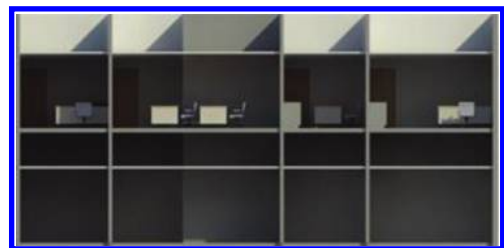


Figure 5b. Section of two story Revit model.

This revit model is exported in IFC format using the Revit Alternate User Interphase for IFC for IFC 2X3 coordination view 2.0. Table 4 shows the summary information of the IFC file extracted using IFC File Analyzer. Originally 7 building

Table 4. IFC export summary information.

Parameter	Value
Revit model size	5.57 MB
Physical IFC file size	734 KB
Number of entities	12539
Object types modeled	7
Object types IFC	9
IFC export method used	20130308_1515—Exporter 2014.0.2013.0308— Alternate UI 2.13.0.1
Schema name	IFC2X3

Profile	Initial profile	Class
12.5*1000		990
12.5*1000		990
12.5*1000		990
12.5*1000		990
12.5*1000		990
50*571.76	???-50MM??	990
150*2180	??-150MM	990
750*51		990
750*51		990
750*51		990
750*51		990
1000*250		990
1000*250		990
1000*250		990
3850*124	50MM*??13MM??	990
4000*124	??-???100	990
4000*124	??-???100	990
4000*124	??-???100	990
4000*124	??-???100	990
4000*124	??-???100	990
8000*124	??-???100	990
???-50MM??-1	???-50MM??	992
???-50MM??-2	???-50MM??	992
???-50MM??-3	???-50MM??	992
???-50MM??-4	???-50MM??	992
IFC BREP	???	996
IFC BREP-1	1200X2100MM	996
IFC BREP-2	???	996

Figure 6. Part of the Tekla IFC object conversion report.

elements were modelled however IFC export added two more objects namely IfcCovering for ceiling and IfcMember of stair case stringer.

This IFC file was imported in TEKLA as a reference model and then the reference model was converted into TEKLA native objects. IFC Log report was generated to identify the converted elements and the status of conversation. The IFC Log file is shown in Figure 6.

As can be seen in the Figure 6, three types of classes are displayed; 990 type which shows these objects have sufficient information, 992 type which shows that parametric information may be missing from the elements and the last 996 type showing there is minimum information about the object and it is converted from Boundary Representation type object (BREP). Both these models were

Table 5. Model comparison report.

Location	Component	Type	Δ Count	Δ Length	Δ Area
Floor 0	Beam	750 X 2000 MM		0.33	
		750X2000MM		0.33	
		???-50MM??-1		0.16	
		???-50MM??-2		0.16	
		???-50MM??-3		0.16	
		???-50MM??-4		0.16	
	Column	1000*250		3.23	
		12.5*1000		13.01	
		50*571.76		1.31	
		750*51		26.25	
		???-50MM??		1.31	
	Railing	900mm round pipe	-2		
	Slab	150*16538			1701.24
		150*16562			2979.77
		150*2180			77.39
		150*5888			856.91
		50*1020			28.95
		Normal - 150mm			-2979.77
		Stair Case Landing			-28.95
	Stair	190mm X 250mm	-1		
		50mm Tread 13mm Riser	-2		
	Suspended Ceiling	Plain			-2578.82
	Wall	3850*124			312.37
		4000*124			860.57
		8000*124			1145.50
		8000*200			4058.32
		8150*200			5393.47
		8150*300			728.07
		Internal - Partition walls 100			-1703.28
		Normal - 200mm			-9451.79
		Normal - 300mm			-728.07
Floor 2	Door	1200 x 2100mm	-1		
		750 x 2000 mm	-4		
	Furniture	1830 x 915 mm	-7		
		Glass Cabinet	-5		
		Rotating Chairs 9	-7		
	Slab	Normal - 150mm			-2635.54
	Suspended Ceiling	Plain			-2928.32
	Wall	Internal - Partition wall 100			-615.16

loaded into Solibiri Model Checker and after using model comparison ruleset the results summarized in Table 5 were obtained.

An alternate method was also employed to import the IFC file through Tekla BIMsight and then storing in *.tbm format. The model was imported in Tekla as Tekla BimSight project and converted into native tekla objects. IFC Log report was generated to identify the converted elements and the status of conversation. The IFC Log file showed improvement as class 996 was eliminated and previously 28 objects were in class 992 and 996 and now only 17 objects are in class 992. The models were compared in the Solibiri Model Checker and the analysis showed similar results but without the BREP objects as these were understood by the tekla in this case.

6 CONCLUSION

Reusability of the information has always been the prime concern for all information modeling systems; however different software solutions have a unique approach towards data processing and management. BIM from its very initiation has shown a tendency to collect all the information on a single platform that can be used, reused and improved along the lifecycle of the project. An information exchange analysis of the two most popular BIM software: Revit and Tekla is conducted in this paper. Both software have their own different import-export procedures that are based on IFC. Revit is able to process IFC data without conversion into native objects but Tekla needs object conversion which tends to more data loss and irregularity. There was an instant small change noticed when the Revit file was converted into IFC and a significant change noticed when the same file was imported to and exported from Tekla. Pure architectural features such as doors, windows, railings and opening caused drastic changes in the ifc object configuration. Tekla BIMsight showed the tendency to regulate these drastic changes due to architectural features however to make the information flow in conformation to the design intent needs much more improvement.

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Green and low carbon building

Sustainable design strategies for sports facility in China

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ABSTRACT: The construction of sports facilities in China had thrived for six decades, but most of the sports facilities had the disadvantages of big energy consumption for operation, unsound indoor physical environments, poor evacuation conditions and low utility rates after sporting events. This paper puts forward a design strategy and method to improve the natural lighting and ventilation of sports facilities, proposes specific optimization measures of design for exit and longitudinal channels of stands to avoid crowd trampling accidents in the course of evacuations, and advocates to adopt the measures of proper selection of site, arrangement of retractable seating and increased versatility of auxiliary rooms to enhance the flexibility for space, to provide favorable conditions for post-game operation.

1 INTRODUCTION

As the public places for urban cultural entertainment and commercial leisure, sports facilities have become an integral part of urban construction. Stadium and sports hall are important public buildings in the city, and even become the symbols of a city. They are important symbols of politics, economy, technology and culture of a region and even a country. The concept of physical education in urbanization has been uniform in terms of ideology with the sustainable social development strategy, which regards people's comprehensive development as the center, and has been included in the strategic objectives of Chinese economic and social development. Twenty years ago, China promulgated Outline of National Fitness Program (State Council, 1995), which vigorously promoted the rapid development of mass sports in China, and the construction of sports facilities also boomed. In recent years, China's urban residents almost all entered a moderately prosperous life. The new concept of consumption of "spending money for better health" has been universally recognized, and now people are willing to spend more time and money on sports consumption in spare time. Especially ever since the Olympic Games in Beijing in 2008, the construction of sports facility in China experienced a new round of upsurge and the quantity and quality of sports facilities were greatly enhanced (L.L. Li et al. 2011). The new design concepts and design methods of sports architecture emerged endlessly.

Many domestic large-scale sports facilities often become the "concomitants" of sports activities. Most of them were built just in order to meet the demands for comprehensive or special sports

games, and the functions are comparatively very single and limited, so they cannot meet the needs for mass sports, recreation and leisure. They are partially abandoned after games. There were many "transient" sports facilities that were prosperous during sports games but abandoned after games in many cities, including the first tier cities like Beijing, Shanghai and Guangzhou, as well as the second and third tiers cities. For example, Beijing "Bird's Nest" Olympic Stadium now only holds no more than 30 sports activities every year after the Olympic Game, but the cost for facility maintenance is considerable, and its economic benefit is quite limited. It even relies on governmental financial subsidies for its maintenance. Most of the sports facilities are under bleak business state in many cities of China, and the degree of comfort is not satisfactory. They have low degree of opening to mass sports. As they have very limited and single function, they cannot be used to make acceptable revenues. In other words, they have very limited social benefits and economic benefits. Usually, the investment on sports facilities exceeds one hundred million Yuan, while the utility efficiency is quite low, so it is actually a great waste of public resources and national financial resources and material resources. Therefore, the sustainable design and study for sports facilities is an extremely practical and strategic research topic.

2 NATURAL LIGHTING AND NATURAL VENTILATION

As a type of large public building, the lighting and air conditioning of sports venues are always the major consumers of the operating cost.

The perfect natural ventilation and natural lighting design for sports building can significantly reduce the operating cost for electrical lighting and air conditioning. If we can optimize the indoor lighting and thermal physical environment of sports buildings, we can effectively save energy, reduce consumption and improve indoor air quality. Comfortable indoor environment can provide higher quality conditions to better use, and ensure the health and comfort of athletes, spectators and other Personnel.

2.1 Natural lighting

In the stadium without completely closed roof or with half open type roof, the problem of lighting of game space in sports facilities can be solved easily; however, for the stadium with retractable roof (when closed) or indoor gymnasium, the lighting problem becomes quite difficult. A majority of gymnasiums in China adopt skylight on roof and high window on side to meet the needs of indoor lighting. In the early period after the founding of P.R. China, there were some gymnasiums with titular skylights, which often encountered the problem of water leakage due to problems of the materials and technology, and eventually the skylights were sealed and blocked for poor or even no effects (Sun, Y.M. 2007). For example, Beijing Workers' Gymnasium was built in 1960s, and its skylight on the roof had no function of natural lighting, it needs artificial lighting even in the daytime. From 1960 to the 1980's, there were numerous "dark box" sports halls without natural lighting everywhere. In the recent past ten years, the importance of natural lighting for energy conservation has been deeply recognized by the architects, together with the progress of waterproof technology, so all the newly built sports buildings had adopted the design concept of natural lighting. Four successful cases are illustrated in Figure 1: Beijing Olympic Wrestling Gymnasium adopted scattered roof steel frame to form some

open type side windows on high places; Beijing Olympic Badminton Gymnasium had set up a ring type skylights composed with 6 arch lighting masks on the center of roof; Guangzhou Asian Games Wushu Gymnasium had set up the wave shaped roof with the meaning of "marine culture" above the playing hall; Guangzhou Asian Games Gymnasium had set up a dozen circular lighting apertures in the central area of the roof.

Along with the development of new materials and high technologies, represented by membrane structure, optical pipe and retractable roof, further promoted the design practice of natural lighting. The polymer membrane materials with perfect light transmission were experimentally used in National Aquatics Center (Beijing Water Cube Natatorium) and Guangzhou New Gymnasium, and the results showed that the architectural form is light and airy with obviously good lighting effect and sufficient daylight during daytime and without turning on electrical lighting (Fig. 2). The working principium of optical pipe lighting is to install light collector system on the roof to change the light transmission direction and transmit the light into indoor space where there is no daylight opening. Beijing Forestry University of Science and Technology installed 148 optical pipes on the roof of its gymnasium, so it's not necessary to turn on electric lamps during daytime and the collected light of the



Figure 1. Natural light in gymnasiums.

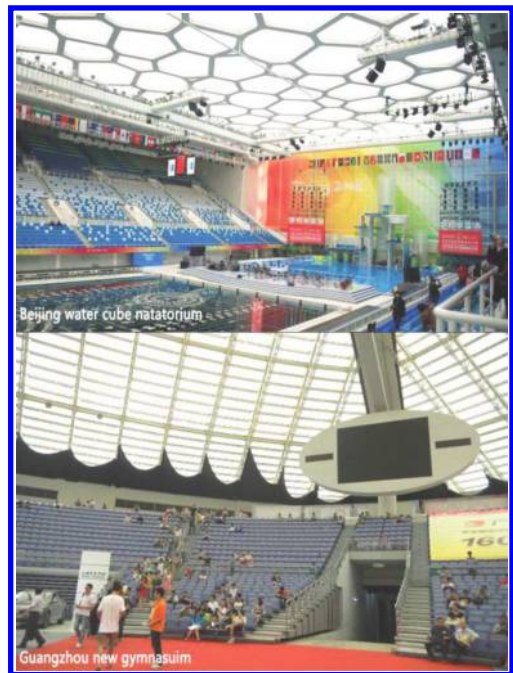


Figure 2. Lighting by membranes in gymnasiums.

optical pipes can basically satisfy the requirement of illumination for students' physical practice (Fig. 3). The retractable roof was originally designed to cope with climate change (Sheard, R.D. 2001). It can be unfolded during sunny weather conditions to satisfy the requirement of illumination for sports events and let the audience feel the natural light fully. However, as the retractable roof involves complicated technologies and high cost, it was not generalized in China. There are some few practical works in economically developed areas, such as: Shanghai Qizhong Tennis Center. When the steel structure roof formed by 8 pieces of petals is fully opened, bright natural light seems as if the spectators were staying in the outside.

2.2 Natural ventilation

The natural ventilation of the building can let the indoor temperature and humidity very close to the natural state, to ensure the comfort of human body and healthy air quality. Good ventilation design is conducive to create a comfortable indoor environment, effectively reduce the cross infection of air borne infectious diseases. As Figure 4 showed, the sports buildings main use wind power



Figure 3. Interior scene in Beijing Forestry University gymnasium with roof optical pipes.

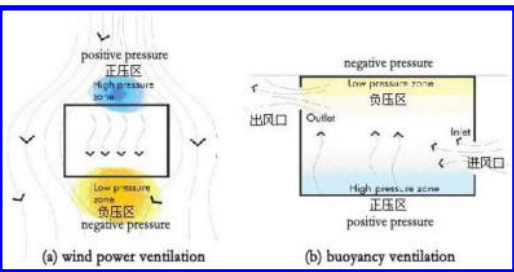


Figure 4. Natural ventilation principle.

for ventilation and buoyancy for ventilation to realize the indoor natural ventilation. The principle wind power is derived from the wind pressure; the wind pressure difference horizontally will cause air flow and cause ventilation. When the wind blows to the building, the positive pressure will be generated on the windward side and negative pressure will be generated in the leeward side, and the pressure difference of air will drive the air to flow from positive pressure area to the negative pressure area. The principle of buoyancy ventilation is based on hot pressure, and it uses the vertical temperature pressure difference of indoor air to drive the air to flow, so the high density air of lower temperature will be stacked on the bottom to form positive pressure, while the hot air will gather on the top to form a negative pressure, and the pressure difference will drive air to move from bottom to top. Our experimental study showed that in the situation of no wind or low wind speed, the ventilation driven by wind power has no remarkable effect for large space sports hall and the people on the stand will still feel obvious stuffiness. The buildings adopting wind-driven ventilation should adopt shallow depth plane along the wind direction, "generally it's better to be less than 14 m" (Wang, P. & Tang, G. 2000), and large span sports buildings often exceed the upper limit and are not applicable. Under normal circumstances, the positions of high windows are much higher than the personnel activity level, and the wind power ventilation in the bottom space below the windows is not satisfactory. Therefore, the design should take more consideration on how to realize the hot pressure ventilation in sports buildings, and this needs to be deliberated according to building characteristics, structure form and details. South China University of Technology gymnasium built in the Guangzhou University City was used as the Asian Game Judo Wrestling Gymnasium, whose roof holistic form is integrated with the ventilated style. The section of competition hall explained this aptly design showed as Figure 5, which uses the hot pressure principle and realized the natural ventilation in the playing hall through the air outlet on the roof and the large number of air intakes on high side windows and side walls.

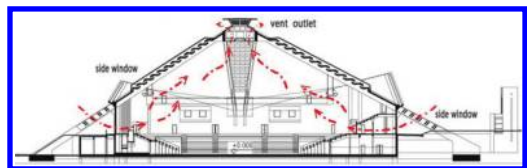


Figure 5. Natural ventilation in South China University of Technology gymnasium.

3 SAFE EVACUATION

Safety is one of the themes for sustainable development. Sports facilities are not only main space for sports matches, but also important places for large-scale theatrical performances, conferences, exhibitions and political rallies. They have the characteristics of highly dense population, hard rapid evacuation and big potential risks. The most terrible accidents are crowd trampling accident that causes mass dead or injury (Zhang, Q.S. 2007). Chinese scholar, Mr. Mei Jikui, spearheaded in design and research of sports building, and he put forward the four basic requirements for safe evacuation of sports facilities: “timeliness, safety, convenience and benefit” (Mei, J.K. 1999), which was commonly acknowledged by the peers in this filed. According to many years of observation and research of the author, during the collective evacuation of spectators, the two places where crowd trampling accident is most likely to occur in sports venues are the exit and longitudinal channels of stands.

3.1 Design of exit

When spectators evacuate from the playing hall after games, the crowd converges from the wide space to the narrow exit, and the density of population instantly increase by over 90%, the movement speed slows down near the exit and it's quite likely for people to be stranded and crowded; when the population entering the waiting area exceeds that passing the exit, the phenomenon of “bottleneck” will occur (Fig. 6). When the crowd congests over a certain period of time, the crowd trampling accident will happen. The bottleneck congestion problems can be resolved by widening the exit, designing some buffer space. Generally, most fireproof doors of the exit are 1.2–1.4 m wide in China, and only allow at most two queues of people to pass through at the same time. At present, most architects in China would rather design fewer seats to broaden the evacuation space approach, to design wider safety doors that allow more queues of people to pass through and some appropriate buffer space to greatly increase the passing rate of crowd to some extent. It is

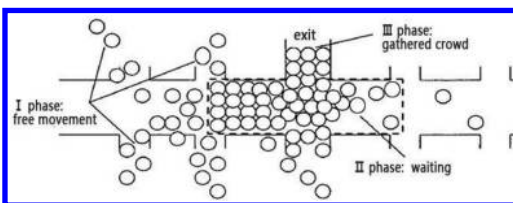


Figure 6. Bottleneck phenomenon explanation.

the extreme practice not to design doors for exit. This practice connects the playing hall with the rest of the audience rest hall into a whole space, so the audience can pass through the evacuation channel to reach the lobby directly. Guangzhou New Gymnasium adopted this design concept, so the seating area and the lobby area are separated only by a thick and low partition (the height is about 1.2 m), and the space seems much broad without blocking the vision to provide favorable conditions for quick and convenient evacuation for spectators.

3.2 Design of longitudinal channel of stand

Generally speaking, the more audience capacity on the stands of playing hall, the more rows of seating there would be, and the steeper the slope of stairs for upper grandstand would be. When the scale is large enough, the height of steps would even exceed 60 cm. What's worse is that the height of each step of the longitudinal stairs is not uniform, and the upper place the slope of the stairs is steeper and usually there is no handrail on both sides. This is why the falling of people often happens. The design strategy to prevent falling phenomenon to happen in the longitudinal channel is to adopt structural measures to improve: adopt wearing and slip-resistant material or coating for the step ground; add slip preventive strips on the steps; add safety handrails on both sides of the channel for the channels with comparatively steeper step slope. According to national Design Code (Design Code for Sports Building, 2003), the width of channel is 1.1 m usually which allow two queues of people walk side by side. But this width is a bit narrow which cause slow and uneasy evacuation. Now most designers adopt 1.2 m width which is the minimum width of British Design Code to consistent with international standard (Guide to Safety at Sports Grounds, 2008).

The side type ramp is a good way to solve this problem. This way has special shape and spectators can evacuate from lower stand to higher stand through the longitudinal channel. When they arrive at lateral ramp, they can easily evacuate downward along the ramp. The stadium of Harbin Sports & Exhibition Center had successfully applied the side type ramp evacuation way (Fig. 7). The ramps are naturally connected with each row of seating to realize barrier-free walking, and it can directly guide the spectators to the outdoor platform on the second floor during evacuation. At the same time, this way has formed a ring road that runs through the playing hall on the outside of the stands and provides additional type of appropriate path for the spectators to enter into stand.



Figure 7. Side ramp in the stadium of Harbin Convention & Sports Center.

3.3 Performance design

Ever since the year of 2000, the foreign sports facilities professional design agencies directly participated in the bidding for domestic large sports facilities. Represented by the Olympic architecture, China set off an upsurge of a new round of construction for sports building. With the application of steel structure and membrane materials that have poor refractory performance and the new big-span technologies in sports facilities, the problem of lagging and incompleteness of some existing standards and specifications in China had become more severe (Sun, Y.M. & Ji, H. 2013). At present, many large-scale sports facilities projects in China use the performance optimization design method for safety assessment, in order to make up for the shortcomings of the conventional design code, so the two types of design methods coexist in China. Some large-scale sports facilities in big cities had adopted the performance optimization design method to evaluate the evacuation safety, such as Beijing “Bird’s Nest” stadium and Water Cube natatorium and Wukesong gymnasium, Guangzhou Asian Games gymnasium, Guangzhou Olympic natatorium and Tianjin Olympic Aquatic Center.

4 FLEXIBLE SPACE

Modern sports facilities are no longer just pure places for sports events and sports training, but are integrated with leisure, entertainment, fitness, catering, shopping and other functions. In case of urban disasters, sports facilities can be used as the public shelters for citizens (Zhang, Y. et al. 2008). Mianyang Jiuzhou Gymnasium in Sichuan province had been used to accommodate about 0.1 million refugees during Wenchuan Earthquake in May 2008. Only with multi-purpose development, making full use of resources and improving the

utility efficiency, can sports buildings solve the problem for their own survival and development. To discuss the design method at the architecture level to improve the utility rate of sports venues after games, and to help the sports venues of low comprehensive efficiency out of plight is an important topic that has huge economic benefit and social benefit.

4.1 Improve utilization rate of site

The formal competition has very strict requirements on the size of site. All professional architects advocate select larger sites rather than small ones. Because larger sites can not only meet the requirements of more games, they have reserved feasibility for variability of space. If the site selection is determined only for ball type games: $(34-36) \text{ m} \times (44-46) \text{ m}$, the site cannot meet the requirements of international stage gymnastics and ice hockey: $(40-48) \text{ m} \times 70 \text{ m}$. Chinese sports buildings design expert, Professor Sun Yimin, advocates the selected site for games should be $40 \text{ m} \times 70 \text{ m}$. As illustrated as Figure 8, the site can be properly modified to become the site of basketball, badminton, volleyball, handball and table tennis through the setting retractable seating; when all the retractable seats are all removed, the site can satisfy the requirements of international gymnastics competition. Here, the retractable seating plays a significant role for increase of space utility rate. When the retractable seating are all folded and piled at the edge of the site, the width of its volume accounts for only about 1.0 m (Fig. 9). According to the design verification of the author, when the selected site is $40 \text{ m} \times 70 \text{ m}$, the site of standard basketball game is set in the middle and the outer ring of the site can be used to place nearly 2000 retractable seats, and this greatly increases the flexibility of the inner space. In the

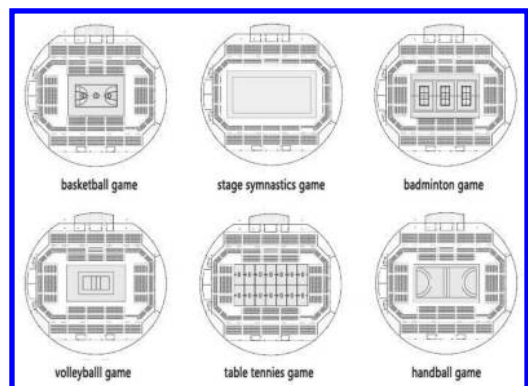


Figure 8. $40 \text{ m} \times 70 \text{ m}$ play site conversion.



Figure 9. Retractable seating in the site.

playing hall of the stadium in Dalian University of Technology, where there are 3000–4000 seats for spectators, a lot of retractable seats were adopted and the available space reaches 65% of the total area of the playing hall, to provide sufficient conditions for diversified fitness and entertainment activities after games (Mei, J.K. & Luo, P. 2004).

4.2 Enhance versatility of auxiliary rooms

As we all know, the auxiliary rooms for news media, events management, VIP spectator and athletes in the sports buildings are highly vacant after games. To improve the flexibility of these auxiliary rooms can realize the secondary chance for higher utility rate after games. Currently, architects mainly deliberate the design of plane column net and elevation storey height and the adding of movable partition to shape highly universal large space (also called “universal space”), to create favorable conditions to convert the functions of different types of rooms after games. The universal design method should not be limited to rigid and fixed mode, but should fully embody the core design concept of “changing to satisfy the variable demands”. We can rely on the design principle of minimum modification, use the similarity of spatial layout to modify VIP box on the playing hall stands into guest rooms after games to make revenues through operation. Figure 10 shows the changes in the plane from VIP box to guest room clearly. The unique experience of facing the game site will let lodgers enjoy the fresh and interesting feeling. We can also lay out the washing rooms of referees and athletes on one concentrated place; on one hand, we can save the equipment and pipe materials for better economy of pipeline layout; on the other hand, we can also merge them as public bath and massage center to open to the nearby residents to make some economic benefit.

Modern sports facilities have gone far beyond the single category of sports places, but have turned into a multi-purpose composite integrated business center. Shanghai eighty thousand people Stadium and Guangzhou Tianhe Sports Center are

the two most successful cases in terms of business operation after games in China (Fig. 11). Shanghai Stadium had set up a four-star hotel “East Asia Hotel”, Century Lianhua Supermarket, a featured catering street, some sports goods store and drug stores, while Tianhe Sports Center was mainly integrated with the commercial catering, leisure and entertainment functions. The mixed business style not only brings considerable economic income to

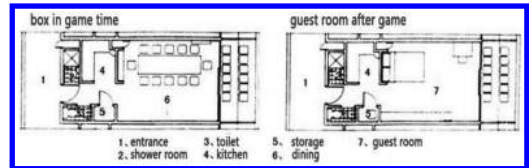


Figure 10. Box transform to guest room.



Figure 11. Successful cases for multi-purpose use.



Figure 12. The lobby of Beijing Industry University gymnasium in linden's wedding ceremony.

them, but the sports facilities are perfectly melted into the life of urban citizens, so the sports facilities become the core facilities that are prosperous and attractive to people. Many operators of the sports facilities in China are changing their concepts and trying to promote the commercial operation of their sports facilities, such as: renting the site, undertaking vocal concert of singing stars and holding conferences, exhibitions, commercial promotions and marriage celebrations etc. In August 2012, the world badminton champion Lin Dan had held a grand wedding ceremony in the gymnasium of Beijing University of Technology to witness him getting married to the female badminton player Xie Xinfang (Fig. 12).

5 CONCLUSION

Throughout over 60 years of development history of sports buildings in China, the quantity and quality of sports facilities had developed rapidly in the past decade, the new design concepts and methods emerged endlessly, and the design has reached international advanced level. This Architectural design industry advocated “sustainable design” for twenty years, so what kind of sports facilities could be called really sustainable building? The answer is Uncertain, but what we can affirm is: “sustainable” sports buildings should be “people-oriented” and at least have such basic characteristics as: green and energy-saving, safety and comfort, high flexibility and versatility.

Sports facilities are large public buildings with huge energy consumption, so the natural ventilation and lighting are the inevitable approach for energy saving design. As the places with high density of population, the safe evacuation of sports facilities has always been the focus of design, and this is the fundamental requirement to avoid crowded trample accident and to maintain public safety. It is the basic idea to strengthen flexibility of sports facilities, so they can be fully used after games to realize their social and economic benefits.

ACKNOWLEDGEMENTS

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Introduction of Decentralized Ventilation systems in buildings

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ABSTRACT: In this study, the decentralized ventilation performance of a small air ventilator to replace natural ventilation for use in urban areas is analyzed and compared to conventional centralized ventilation systems in Central European climates. This unit is equipped with a fan, air filter and air damper, as well as a heat exchanger, which is to be operated when the supply air also needs to provide heating or cooling. Compared with a conventional centralized economizer, this system has shorter air transport distances and therefore entails lower pressure losses. In a decentralized system, fan speed and airflow rate are adjusted simply and effectively depending on indoor thermal conditions.

1 INTRODUCTION

Hybrid Ventilation (HV) combines traditional centralized mechanical ventilation and natural ventilation to reduce HVAC energy demand based on outdoor climate conditions. During intermediate seasons, when outdoor conditions are mild, these systems use natural ventilation modes to save energy, whereas during hot or cold seasons centralized mechanical ventilation modes are used. HV systems can implement a broad range of ventilation strategies to adapt to local environmental conditions. HV is also categorized into three types (Heiselberg 2002): natural and mechanical ventilation, fan-assisted natural ventilation, and stack- and wind-assisted mechanical ventilation.

Decentralized Ventilation (DV) is a type of mechanical ventilation that operates in similar way to fan-assisted natural ventilation. However, outdoor air is distributed into the room by passing through a compact decentralized air-handling unit rather than being utilized directly. Numerous publications have presented and discussed these systems' potential as a viable replacement for centralized ventilation systems (Manz et al. 2001, Manz et al. 2000, Baldini et al. 2014, BINE Informationsdienst, Baldini, 2009).

Figure 1 shows schematics of centralized, semi-decentralized, and decentralized ventilation systems. Centralized Ventilation (CV) systems are most commonly used, but have long supply and exhaust air distribution passages. Semi-decentralized ventilation captures outdoor air at the façade and exhausts air to a centralized exhausting system. Decentralized ventilation has the shortest air travel distance as individual compact air handling units control each room's ambient conditions.

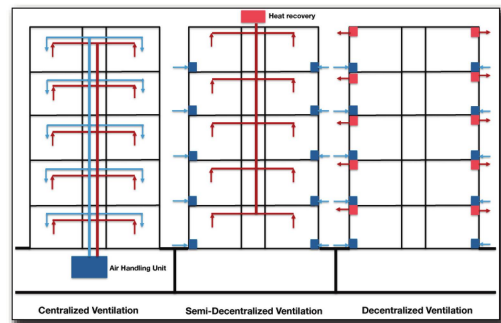


Figure 1. Comparison of centralized, semi-decentralized, and decentralized ventilation systems, based on the novel research (Mahler and Himmler 2008).

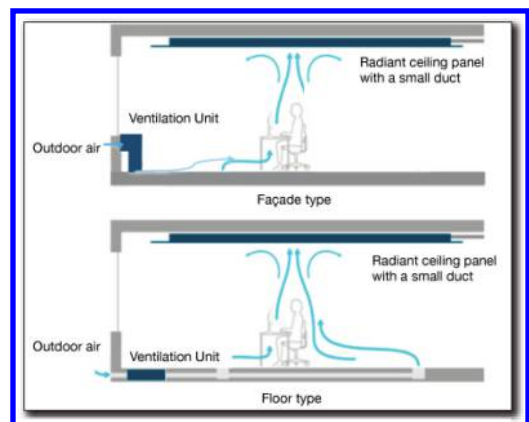


Figure 2. Schematic of decentralized ventilation systems: façade type (top) and floor type (bottom).

DV systems are further categorized into façade and merged floor or ceiling type. Figure 2 shows schematics of decentralized ventilation systems. The main advantage of using façade type ventilation systems is their simple installation, but they cannot distribute fresh air deeply into the space. Floor type ventilation, on the other hand, does not take up indoor space and can supply fresh air more deeply, but its system has higher construction costs than those of façade type.

2 METHOD

Selected European weather conditions were used to determine acceptable conditions for the operation of fan-assisted ventilation systems and to analyze the DV system's cooling and heating loads. The case study buildings are located in Paris, Berlin, Zurich, Rome and Copenhagen. Outdoor air temperature is one of the main parameters that determine whether a fan-assisted NV system can be operated. Considering time-dependent criteria, a method to determine the possibility of using outdoor air depending on occupied and unoccupied hours is presented (Menassa et al. 2013). This method calculates the minimum and maximum indoor air temperature (T_{IA}), humidity ratio based on the adapted model by Brager & Dear (2000) and ASHRAE standard 55 (2010).

3 ANALYSIS

Figure 3 presents the availability of fan-assisted NV in each city throughout the year. With the exception of Rome, the operation of the NV system generally occurs between May and September because the outdoor weather conditions of Central European countries during this period are neither too hot nor too cold. The total availability of the fan-assisted NV system is around 22–32% per year in the selected European countries, which means that the full HVAC demand for 22–32% of the year corresponds to the fan energy of the system because no additional cooling and heating output is needed during that season. Thus, saving fan energy is a very important strategy to minimize building energy consumption in these climate conditions.

Feustel & Stetiu (1995) described that Radiant Ceiling Panel with Centralized Ventilation systems (RPCV) save 45.3% of peak energy consumption compared to all-air systems, as shown in Figure 4. This study also found that Radiant Panel with Decentralized Ventilation systems (RPDV) save 5.5% of fan energy but had a 1% higher pump energy demand due to pressure losses in the hydronic water supply system for the decentralized

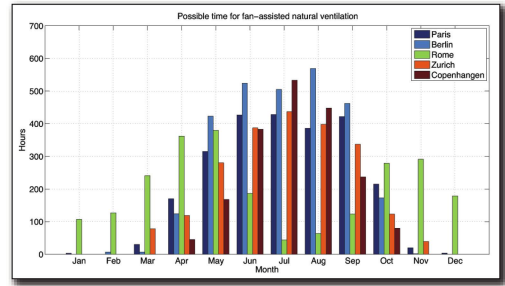


Figure 3. Availability of fan-assisted natural ventilation in each city throughout the year.

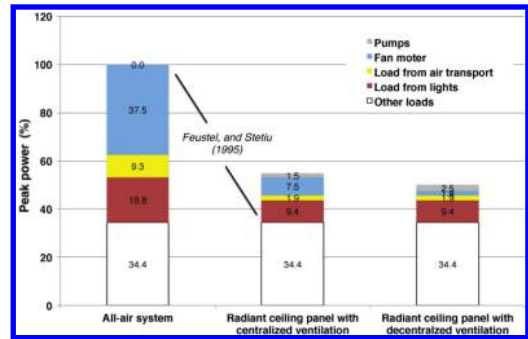


Figure 4. Comparison of electric peak power load for all-air, radiant panel with centralized ventilation, and radiant panel with decentralized ventilation systems.

air convector. Therefore, RPDV saved an additional 5% of total energy consumption.

4 CONCLUSIONS

We analyzed the energy consumption of three HVAC systems in European climatic conditions: all-air, Radiant Panel with Centralized Ventilation (RPCV), and Radiant Panel with Decentralized Ventilation (RPDV) systems. The RPDV system had the lowest energy demand because it not only minimizes supply and exhaust air pressure losses, but also allows the system to be operated as a fan-assisted natural ventilation system in periods in which it is possible to use outdoor air without thermal loads.

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A new reflection on the window to wall area ratio from an energy conservation view

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ABSTRACT: The exterior window serves as an essential part of a building, both in function and configuration. Due to its poor thermal performance, compared to other building parts, it has been a key factor in affecting a building's energy consumption. "Window to wall area ratio" is an important parameter indicating a residential building envelope's thermal performance. However, with the rapid development of design concepts, as well as building materials, a variety of window materials' physical parameters and thermal performance have changed. Thus, making it difficult to use present "window to wall area ratio" to measure different windows' energy efficiency effects. Therefore, it is necessary to rectify the notion according to new conditions. This paper will, from an energy conservation view, clarify what effects different exterior windows would have on energy conservation and puts forth a new concept "equivalent window-wall area ratio", in order to improve the traditional definition. The new concept will be favorable to building energy conservation and calculation.

1 INTRODUCTION

Window to wall area ratio is a key indicator and is frequently used in construction and thermal energy conservation design. According to *Civil Building Thermal Design Code* GB 50176-93, window to wall area ratio is the rate of the window's projected area to elevation unit area, which can be refined as the ratio of the external window to the wall area ratio. *Civil Building Thermal Design Code* GB 50176-93 stipulates that of the residential buildings in all directions, the window to wall area ratio of buildings having a northern exposure cannot exceed 0.25, the ratio of buildings facing east or west cannot exceed 0.3, and the rate of those having a southern exposure cannot exceed 0.35. However, in fact, architectural design should take the energy consumption into account for practical application. Given that the external window rates poorest in thermal performance in a building's envelope structure and can affect a structure's air ventilation and material consumption significantly, we should set up a more reasonable window to wall area ratio to help conserve energy.

In recent years, new types of external windows, such as bay windows and concave windows have been widely used in construction. Their physical property parameters such as thermal bridges, solar heat gain, and superficial areas are quite different from that of traditional windows, however, the present definition of window to wall area

ratio fails to notice the changes in the lighting and energy consumption calculation, and as a result, it is unable to reflect the actual energy-saving effect. Therefore, modifying the concept according to specific climates, geographical conditions, designs, and materials becomes imperative.

This paper discusses the window to wall area ratio mainly in views of window style design and climate distinction. Taking traditional windows as reference standards, it tries to compare the energy consumption and loads of both casement windows and square bay windows during the winter of three Chinese cities, Harbin, Beijing and Shanghai. The comparison is made under the quantitative analysis of those windows' energy-saving effect. According to analysis results, this paper puts forward a correction method of the window to wall area ratio, and the new correction coefficients as well as its value ranges.

2 BUILD COMPARABLE MODELS

This paper analyses the window to wall area ratio from the perspectives of energy conservation and heating. It chooses Harbin, Beijing and Shanghai as the experimental plots, and develops 3 multi-story building models, each of which comprises 8 floors with 4 units per floor. They are the same in terms of configuration, function, structure, material, and orientation, as well as window to wall area ratio. For the buildings with the same

floors, same sides, same plane, same heights and same projected areas of windows, we design 3 different types of windows for each: one is a casement window (Fig. 1: in the top and left, type A), one is a square bay window protruding 600 mm (Fig. 1: top and right, type B), and one is a semi-circle window protruding 1200 mm (Fig. 1: below, type C).

Building envelope system's performance parameters are set up as follows: according to *Design Standard JGJ26-2010 for Energy Efficiency of Residential Buildings in cold and severe cold area*, we chose cold zone A's (Beijing) building envelope thermal performance parameters. For comparative convenience, we assume the envelope system's parameters of Harbin and Shanghai are the same as Beijing's. Under identical conditions (All windows are 2.1 meters high), a bay window has two more pieces of glass with 1.26 m² per piece, as well as a top board and a bottom board with 1.44 m² per board, than a casement window. Given that the top and bottom boards have a much better thermal insulation than that of the glass, it will be more convenient for calculation if it was left for discussion until later. In addition to that, all parameters including thermal insulation, solar heat gain performance and airtight performance have been major factors affecting energy consumption, particularly airtight performance.

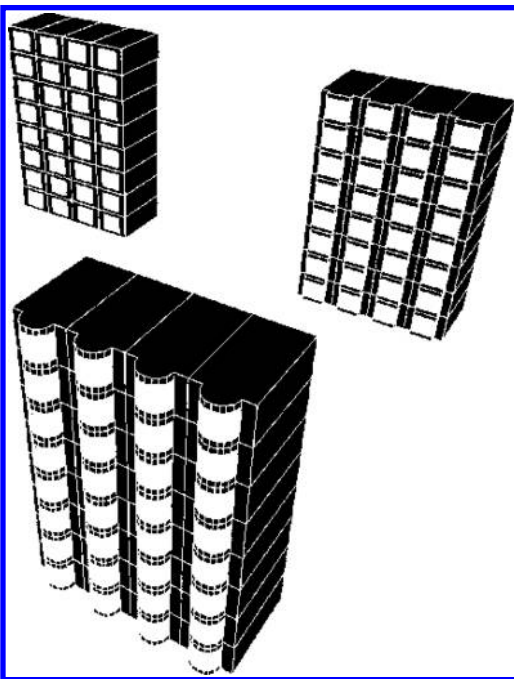


Figure 1. Comparative models.

Through simplification, the window's energy saving effect is mainly related with the window orientation, configuration and the solar heat gain performance. The stimulation tool adopted in this paper is CDAJ's integrated building stimulation software-IES <VE>.

3 ENERGY CONSUMPTION COMPARISON

3.1 Comparisons of climates

The major factors affecting energy consumption are the same. For calculation convenience, we assume that the parameters of the building envelope in Harbin, Shanghai and Beijing are the same, but there still lies large differences between their energy consumption due to their different climates. Of all the factors, dry-bulb temperature and solar radiation affect the building energy consumption in winter significantly.

Figure 2 shows that during the typical winter week, from January 1st to January 7th, Harbin's dry-bulb temperature is the lowest with a temperature between -15°C to -20°C , while Shanghai's temperature is the highest between 5°C to 10°C , and Beijing is in the middle with -5°C to 0°C . Within these chosen days, the three locations' solar radiation values all have changes and the maximum value is about 400 W/m^2 . We can see that the main factor affecting their energy consumption is the dry-bulb temperature.

3.2 Loads comparison

1. The loads of same buildings in different locations

Through comparing buildings with casement windows, Figure 3 shows that during the chosen week, the indoor loads of building in Harbin is the highest while that in Shanghai is the lowest, and Beijing is in between. This result is as expected.

2. The loads of buildings with different bay windows in one location

Figure 4 shows the thermal loads distinction between Beijing buildings with the same orientation and dwelling types, respectively. By comparing their heating equipment's loads, dry-bulb temperature and global solar radiation, we can find the result of thermal loads comparison is: semi-circle bay window > square bay window > casement window. The order is also applicable for the other cities. Obviously, bay window's energy consumption is much higher than that of the casement window's.

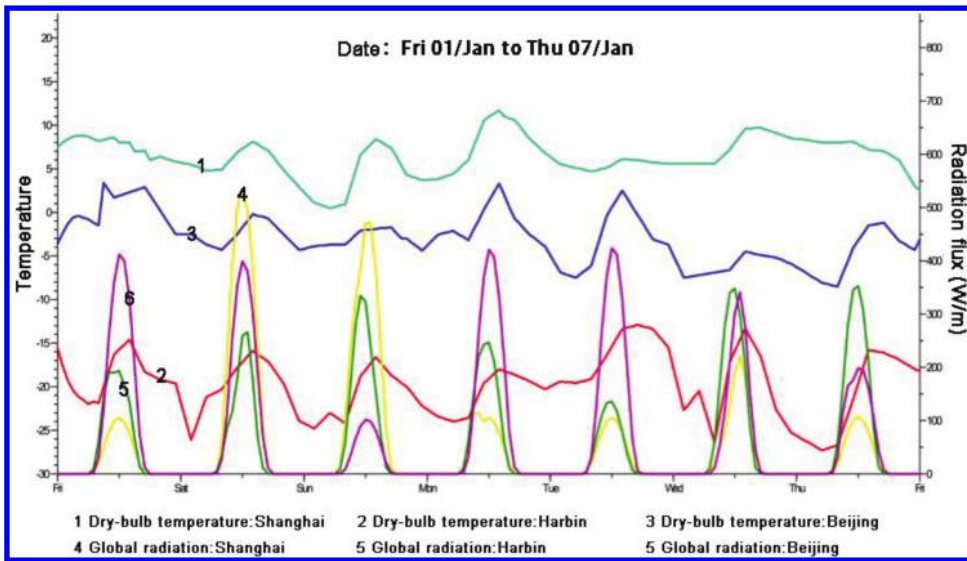


Figure 2. Climates.

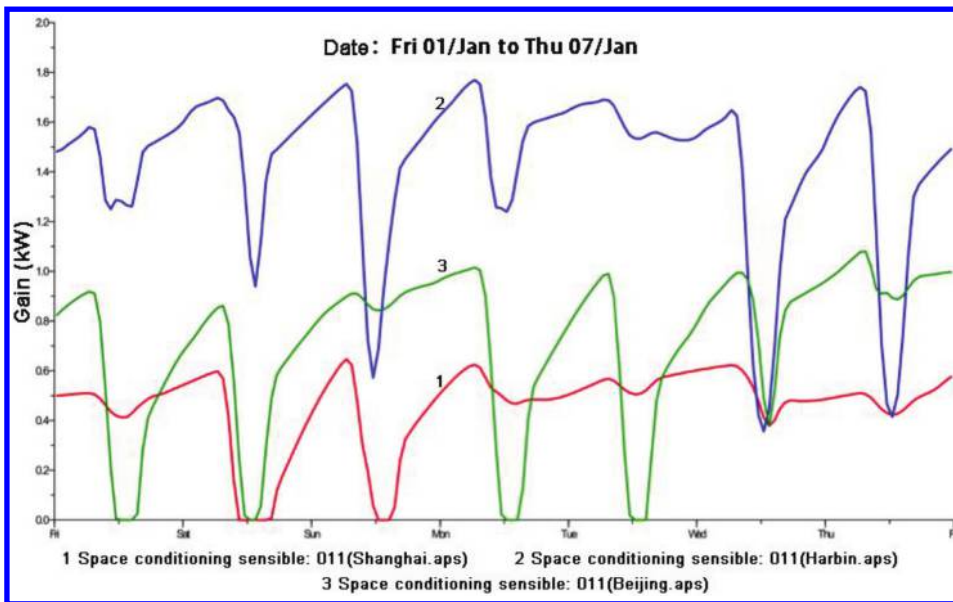


Figure 3. Loads comparison of general buildings (casement windows).

3. Gross thermal loads comparison

Diagram-5 and 6 are the entire building's per unit area thermal loads comparison during the heating period, from November 15th to March 15th, which shows the order is consistent with that of a single unit's thermal load comparison: curved bay window > square bay window > casement window.

3.3 Loads comparison conclusion

In terms of the building's thermal loads, different windows, even with equal window to wall ratios, have different loads. With bigger windows, having a larger superficial area, more energy is lost. So it is necessary to rectify the traditional "window to wall area ratio".

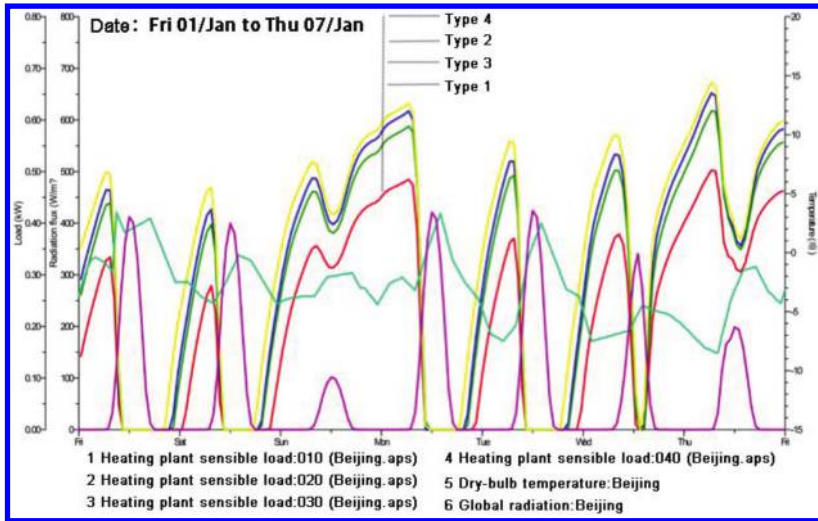


Figure 4. Loads comparison of Beijing's different bay windows.

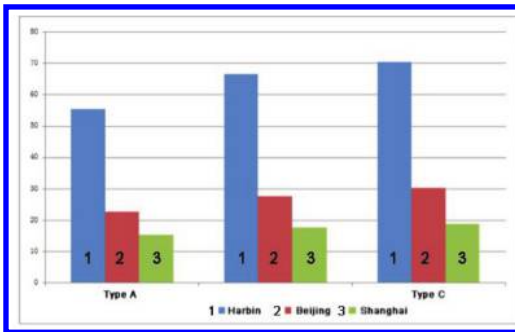


Figure 5. Gross thermal loads comparison.

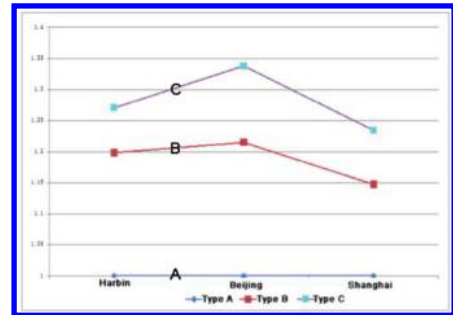


Figure 6. Energy-consuming comparison of different windows.

4 WINDOW TO WALL AREA RATIO RECTIFICATION

4.1 Brief introduction of rectification

From the above descriptions we can conclude that it is unreasonable to define the “window to wall area ratio” only from the perspective of energy conservation, as the thermal performance should also be taken into account to keep up with the diverse architectural designs and modern technology, as well as the material’s development. Therefore we try to rectify the original definition by putting forward “equivalent window to wall area ratio” for relevant departments and design organizations to reference.

“Equivalent window to wall area ratio” is the ratio of a window’s equivalent area to elevation unit area for residential units on the same floor, same side and same dwelling type but with windows that have different materials and types. The calculation of

“equivalent window area” is to multiply a window’s projected area by the area coefficient. The calculation of area coefficient should consider such factors as a window’s facade area, material, form, each year’s energy consumption, and baseline consumption.

The formula is as follows:

$$\alpha_{\text{gross}} = \alpha_{\text{area}} * (\beta_{\text{air-tightness}} * \beta_{\text{solar heat gain}} * \beta_{\text{boards}} * \beta_{\text{thermal bridge}})$$

α_{gross} is the correction coefficient of the original window to wall area ratio, which is equal to the product of various modified coefficients in it. α_{area} is the increased surface area after adopting bay window. β is a series of thermal coefficients based on α_{gross} and these coefficients include: coefficients of air tightness, solar heat gain (taking orientations into account), top and bottom boards, and thermal bridge. The following passage will introduce one by one.

4.2 Thermal coefficients revision

This part will study the calculation of thermal correction coefficients and its database can be received through massive data analysis in future practice. In theory, loads are composed of external window's heat-transfer loss, infiltration loads (window's air-tightness) and solar heat gain. The revision of all heat transfer factors $W/(m^2 \cdot k)$ can make the calculation less affected by the changing temperatures and areas to make sure the correction coefficients are more reliable.

1. Area revision: α_{area}

We assume the "area coefficient" of casement with the same materials and same types as 1. In a residential building, there are 2 units with the same latitude and height, whose southern external wall area is 28 m^2 . One uses square bay windows while the other uses casement windows. Here are their shape parameters:

Casement window: width*height: 2400×2100

Bay windows: width*height*depth:

$2400 \times 2100 \times 600$

Window projection area: 5.04 m^2 .

Under the same conditions, a bay window has two more pieces of glass, with an area of 1.26 m^2 per piece, than a casement window. A bay window also has top and bottom boards with an area of 1.44 m^2 per board. Given that good thermal insulating effect of boards, we will discuss the top and bottom boards later.

The equivalent area coefficient:

$$\alpha_{\text{area}} = (5.04 + 1.26 \times 2) / 5.04 = 1.5$$

2. Thermal correction coefficient: $\beta_{\text{air-tightness}}$

Except for equivalent area coefficient, we should also correct other thermal coefficients. For example, bay windows have stretched the length of the osmosis seam, whose infiltration airflow can be figured out according to the length of seams and the air-tightness levels.

When calculating, we can refer to the data of *Graduations and test methods of air permeability, water tightness, and wind load resistance performance for building external windows and doors* GB7106-2008. Then we can put the calculated values into corresponding virtual reality software to work out the energy consumption. The $\beta_{\text{air-tightness}}$ is often between 1.1–1.3.

As shown in the Table 1, we take 4 level length of seams as a basis of calculation. There is a large amount of thermal loss as well as other loads caused by infiltration; we will take type B as an example, and its correction coefficient, 1.13.

3. Thermal correction coefficient: $\beta_{\text{solar heat gain}}$

	Type A	Type B	Type C
Solar heat gain	1	1.28	1.1
$\beta_{\text{solar heat gain}}$	1	0.78	0.91

After adopting bay windows, despite the envelope structure causing great conductive heat loss, the solar heat entering the building increases more. The energy loss can also be found by stimulation software. If we assume the casement window's solar heat gain capacity (type A) is 1, then the bay window's capacity (type B) is 1.2804 because compared with the casement window, the bay window can absorb more solar heat in more directions during the daytime (Azimuth). According to the formula $Q = KF\Delta t$, the area value of solar heat gain is $1/1.28$, namely, $\beta_{\text{solar heat gain}} = 0.78$.

4. Thermal correction coefficient: β_{boards}

As mentioned above, the boards' thermal insulation effects are much better than glass'. For easy calculation, we did not take the bay window's top and bottom boards' revision into account when doing thermal calculation. Actually, top and bottom boards can bring the windows 10–20 percent thermal loads. If the top and bottom boards' external insulation construction do

Table 1. Air-tightness levels.

Level	1	2	3	4	5	
Unit length $q_1/[m^3/(m \cdot h)]$	$4.0 \geq q_1 \geq 3.5$	$3.5 \geq q_1 \geq 3.0$	$3.0 \geq q_1 \geq 2.5$	$2.5 \geq q_1 \geq 2.0$	$2.0 \geq q_1 \geq 1.5$	
Unit area $q_2/[m^3/(m^2 \cdot h)]$	$12 \geq q_2 \geq 10.5$	$10.5 \geq q_2 \geq 9.0$	$9.0 \geq q_2 \geq 7.5$	$7.5 \geq q_2 \geq 6.0$	$6.0 \geq q_2 \geq 4.5$	
	Length (m)	Osmolality (m ³ /h)	Ventilation (Ach)	Loads (W/m ²)	Other loads	
Type A	9	20.25	0.201	4.455	38	1.000
Type B	20.4	45.9	0.456	10.098	38	1.133
Type C	19.57	44.0325	0.438	9.68715	38	1.123

not take any thermal/cold bridges measurement, the β_{boards} is greater.

We can use the steady heat-transfer method to calculate:

$$\begin{aligned}\beta_{\text{boards}} &= \frac{Q_{\text{loads of boards}}}{Q_{\text{loads of glasses}}} = \frac{k_1 * F_1 + k_2 * F_2 + k_3 * F_3}{k_1 * F_1} \\ &= 1 + \frac{k_2 * F_2 + k_3 * F_3}{k_1 * F_1}\end{aligned}$$

k_1, F_1 are the heat-transfer coefficients and area of transparent parts. k_2, F_2 and k_3, F_3 are the heat-transfer coefficients and areas of top and bottom boards.

For casement window, we can assume its β_{boards} as 1. Taking type B in Beijing as example, we can conclude:

$$k_1 = 2.0 \text{ W}/(\text{m}^2 \cdot \text{k}), k_2 = k_3 = 0.6 \text{ W}/(\text{m}^2 \cdot \text{k}), \\ F_1 = 5.04 \text{ m}^2, F_2 = F_3 = 1.44 \text{ m}^2.$$

$$\text{So, } \beta_{\text{boards}} = 1.17$$

5. Thermal correction coefficient: $\beta_{\text{thermal bridge}}$
Like $\beta_{\text{air-tightness}}$, $\beta_{\text{thermal bridge}}$ can be calculated by stimulation software, which analyzes the heat-transfer quantity that the bay window brings to the thermal bridge and compares it with the window's entire transfer-heat quantity. The coefficient has much to do with the window type, since this paper does not do further research on it, we assume it is between 1.1–1.3 temporarily.

4.3 Integrated correction

We assume that: the $\alpha_{\text{area}} = 1.5$, $\beta_{\text{air-tightness}} = 1.13$, $\beta_{\text{solar heat gain}} = 0.78$, $\beta_{\text{boards}} = 1.17$, $\beta_{\text{thermal bridge}} = 1.1$, the latitude correction coefficient is not into consideration:

$$\begin{aligned}\alpha_{\text{gross}} &= \alpha_{\text{area}} * (\beta_{\text{air-tightness}} * \beta_{\text{solar heat gain}} \\ &\quad * \beta_{\text{boards}} * \beta_{\text{thermal bridge}}) \\ &= 1.5 * (1.13 * 0.78 * 1.17 * 1.1) = 1.70\end{aligned}$$

The results show that bay window's heat-transfer capacity is 1.7 times of casement windows. With this, we can infer that under the same conditions, bay window's heat-transfer capacity is much stronger than that of an ordinary window. While according to the traditional "window to wall area ratio" calculation method, the heat-transfer values of bay window and ordinary window are equal. Therefore, the traditional "window to wall area ratio" doesn't correspond to reality and needs to be rectified.

Meanwhile, we find the equivalent area of windows is the decisive factor in the process of revision.

5 CONCLUSION

The traditional definition of the "window to wall area ratio" is concise and practical, but not scientific under new conditions. This paper, from a heating and energy consumption perspective, emphasizes on what effects different external window forms would have on energy conservation and puts forward a new concept "revision of equivalent window to wall area ratio", aiming to improve the traditional residential building energy conservation design standard. The revision can help designers count and analyze the thermal performance of present windows, and then make up a clear diagram for reference without complex calculation. This can allow them to quickly calculate the energy conservation effect of windows having certain exposures in a city and take conservation measurements, which not only saves time, but also increases efficiency. Once applied, this revision also helps make the traditional notion of "window to wall area ratio" more concise and operable, and better reflects what effects different windows would have on the energy consumption.

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Energy saving design method study on university existing buildings in cold climate zone

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ABSTRACT: Based on numerous universities of various types it is identified that they have become a major energy consumer. This study takes existing public buildings of universities in cold climate zones as the object and starts with the investigation into the current situation and characteristics of energy consumption by public buildings of one typical university in Xi'an to systematically analyze the characteristics of existing buildings, test the thermal environments in summer and winter, and obtain first-hand data. Besides, the principles and design methods suitable for the existing public buildings' energy-saving, the transformation of universities are discussed with a view to find out its key problems and establish a corresponding energy-saving technology system. With respect to the application technologies, flexible technology combination and application is achieved, aiming at different characteristics of existing building energy consumption and the input of transformation funds, and energy-saving technology clusters at different levels are formed to practically implement the transformation.

1 INTRODUCTION

Since the outburst of the first energy crisis, the contradiction between supply and demand of energy has drawn more and more attention of people. It is well-known that the proportion of building energy consumption in the total energy consumption stays at a high level, and the efficient energy utilization and building energy conservation has become a hot topic at all levels day by day (Tsinghua University Building Energy Research Center, 2013). Accommodating 24 public and private universities in total, Xi'an city is the fourth city, following Beijing, Shanghai, and Nanjing, where universities are most concentrated in China and the city where there are the most universities in the northwest region. The flourishing higher education undertakings make the universities in Xi'an city the major energy consumer of public buildings of the city. The current situation of building energy consumption of universities in Xi'an city is mainly characterized by:

1. High power consumption

As shown by data, the total power consumption of universities is rising at an average rate of nearly 20% every year, and the immediate cause of this high-speed growth is the increase of students. The average annual electricity expenses of universities, as a highly energy-consuming unit, are all above RMB ten million, and not a few universities are found wasting electric energy to different extents.

2. High energy consumption of heating system

There are relatively few energy-saving transformation projects for universities in Xi'an city, which are basically at the phase of campus system transformation, especially for the heating system in winter. The universities in Xi'an city began to develop in succession after the founding of New China, and the first batch of teaching buildings were constructed from the 1950s to the 1960s, almost without energy-saving measures taken. Such buildings were featured by relatively low building comfort, low electric energy consumption, and high heating energy consumption, thus having a good potential for energy-saving transformation. However, many buildings of that period have been rated as the unit under cultural relics protection, so special caution should be given to their energy-saving transformation. After the reform and opening up, universities of China expanded their enrollment, and the 1980s consequently became another peak period of teaching building construction. Buildings constructed in that period took energy-saving methods at a relatively low standard and had a little change in electric energy consumption and comparatively high heating energy consumption. At the beginning of the 21st century, enrollment expansion was implemented again, and most universities started a new round of campus construction. Although the measure of 50% energy conservation had been taken in Xi'an city at that time and the majority of buildings adopted corresponding energy-saving approaches, the electric energy consumption of teaching buildings

still increased substantially at that phase on account of factors including expanded volume and scale of buildings, electronization of teaching equipment, and increasing application of air-conditioning system; nevertheless, the heating energy consumption decreased than before, and the potential for energy-saving transformation was great (Liu et al. 2013).

2 ENERGY CONSUMPTION INVESTIGATION AND TEST

The heating energy consumption of buildings in cold region occupies a very large proportion (MOHURD. 2012). Meanwhile, buildings of universities have their own characteristics of energy use, principally embodied by power consumption of office and teaching buildings; water and heat consumption of bathrooms; power consumption of dormitories; cooking energy consumption of canteen; energy consumption of air-conditioning equipment necessary for continuous running of special laboratories; outdoor street lamps; energy consumption of newly-established central air-conditioning system, etc. As a result, analysis must be conducted case by case to seek energy-saving

transformation approaches by starting with energy consumption investigation. This study conducts energy consumption investigation and test of office buildings in large quantity and scale so as to identify the main problems of existing buildings in the aspect of energy consumption at present and thereby find out the scientific, rational, and integrated solution.

2.1 Overview of energy consumption of existing office buildings in Xi'an Jiaotong University

Xi'an Jiaotong University is the most renowned university in the northwest region, and a large number of teaching buildings in the main campus were constructed during the 1950s–the 1960s; new buildings that are constructed recently have been improved significantly in terms of the thermal insulation of building envelope, and the indoor comfort has also been improved correspondingly.

Hence, Xi'an Jiaotong University can be regarded by the study as a representative of the overall features of universities in Xi'an city region. Through investigation and interview, the study summarizes the current energy consumption situation of principal office buildings in the main campus of Xi'an Jiaotong University, as indicated in [Table 1](#).

Table 1. Information of office buildings in Xi'an Jiaotong University (Source: Self-drawn).

Building description	Building area (m ²)	Construction time	Function	Annual power consumption (kWh)	Power consumption per unit area (kWh/m ²)
Science and Technology Building at North Gate	1280		Office	110352	86.2
Software Building	1813		Office	157920	87.1
Computer Center, former Infrastructure Department	4467	1980s	Teaching and office	274836	61.5
Former Administrative Building	5696	1950s	Office	181430	31.8
Material Building and attached bungalow	6351	1980s	Experiment and office	633024	99.7
East Building 3 and attached bungalow	15923	1950s	Experiment and office	1991784	125.1
Multiphase Flow Laboratory	3348	1970s	Experiment and office	397200	118.6
Energy Source Museum	5310	1990s	Experiment and office	296400	55.8
High-pressure and Pneumatic Building and Westward Movement Memorial Hall	4150	1950s	Office	207680	50.04
Science Museum	10700	1988	Office and conference	223200	20.9
Sports Department	3820	1950s	Office	190934	49.98
Main Administrative and Teaching Building	52944	2004	Office	5564945	105.11
Building of the School of Management	19508	2003	Office	1460694	74.88

It can be observed from Table 1 that the power consumption per unit area of office building for experiment use is very high due to special power supply and experiment needs, therefore such building cannot be taken as the object of study on the power consumption of general office building. Among the buildings only for the purpose of office, the Main Administrative and Teaching Building consumes the most power per unit area, and the growth rate of building energy consumption, compared to old buildings constructed during the 1950s–the 1960s, is much larger than that of comfort. Against current general background of stress on energy conservation and high efficiency, the Main Administrative and Teaching Building of Xi'an Jiaotong University, which has been put into use for about 10 years, is facing various problems.

2.2 Energy consumption test of the main administrative and teaching building of Xi'an Jiaotong University

The Main Administrative and Teaching Building of Xi'an Jiaotong University were changed from its predecessor—the Teaching and Scientific Research Building—as a result of the need for administration space. The middle high-rise building is surrounded by 4 attached teaching buildings. The 24-floor administrative office space is the commanding height in the central axis of the university and one of the landmarks of the university, as shown in Figure 1.

The focus of the study—Block E of the main building—is in 24-floor frame structure, with lobby set on the second floor, and connected with the Four Great Inventions Square on the north via the platform. The main space starts above the second floor, the functions are made up of various scientific research and administrative departments, and an internal full-height atrium is provided every 3 floors to organize the space, as shown in Figure 2.



Figure 1. Main administrative and teaching building external view (Source: self-take).

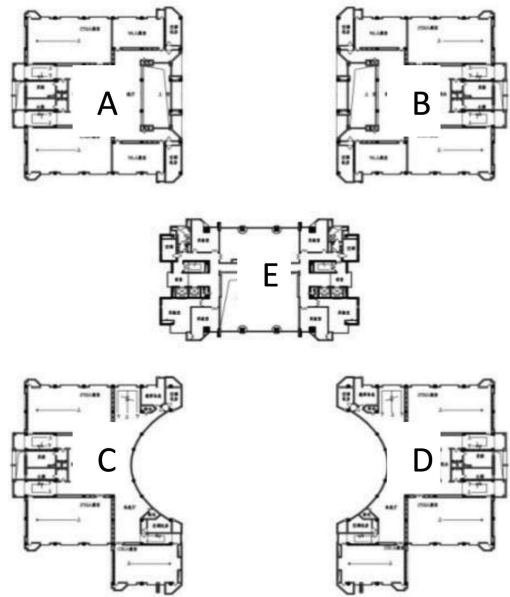


Figure 2. The fourth floor of main administrative and teaching building: A/B/C/D as Teaching Building, E as Administration Building (Source: self-drawn).

The <Design Standard for Energy Efficiency of Public Buildings> had not been issued at the outset of main building design, so only 30% energy-saving standard was adopted. Though the exterior wall of building was transformed to a certain degree in the later construction, the thermal insulation performance of building envelope still failed to meet the requirement for 50% energy conservation. The Main Administrative and Teaching Building was put into formal operation in 2006, but according to the office staff and students, the ventilation in the main building is relatively poor, and both heating and cooling there are realized completely by air-conditioning equipment. Therefore, the study selects it as the representative of new buildings in campus to test its energy consumption and thermal environment.

1. Location of test points

The test points were majorly located in the central part of each main space on the target floors to obtain relatively accurate indoor data as atrium and north and south office. Since Block E of the main building is basically set with an atrium every 3 floors, the test points were distributed by taking 3 floors as one test gradient, so that the vertical variations of temperature and humidity from low to high could be gotten through data analysis.

Block E of the main teaching building of Xi'an Jiaotong University was tested twice, i.e.,

in summer and winter, respectively for continuous 36 sunny hours in the hottest month and continuous 48 sunny hours in the coldest month.

2. Data of summer test

Because Block E is a high-rise building whose wall temperature at too high position cannot be obtained with a point temperature gun, merely automatic temperature/humidity recorder was used in summer test to collect the changing data of indoor temperature and humidity. Block E of the main building was attended during vacation in summer and the air-conditioning unit was running in the daytime, the changing data of indoor temperature without air conditioning were not obtained. And the air conditioner of Xuanhuai Hall on the 23rd floor wasn't turned on during the test due to low utilization rate of the hall.

It can be seen from Figure 3 and 4 that in the time period with air conditioning in summer, the indoor temperature and humidity are both within the value ranges of human comfort, but the indoor temperature and humidity of Xuanhuai Hall (exhibition hall) on the top floor obviously exceed the said ranges. The temperature is basically between 24°C and 28°C as revealed by the data of the first day; however, it sharply drops from 30.5°C to nearly 26.5°C since 21:00 on the second day, indicating the air-conditioning unit increases cooling load outside office hours at night and inappropriate management gives rise to high energy consumption.

3. Data of winter test

Winter data were collected in the process of building use, that is, in the heating state.

It can be found from the statistical table of test data that:

- The temperature change of southern and northern rooms is inversely proportional to the humidity change of both, the temperature on the south side is a little higher than that on the north side, and the temperature change of southern room is slightly more violent than that of northern room.
- The highest temperature occurs at about 15:00–16:00, while the lowest temperature appears at around 24:00.
- The indoor humidity values of both rooms are apparently low.
- The indoor temperature of atrium on a higher floor is higher than that on a lower floor, which may probably be related with the application function of room (floors above 16F are occupied by leader offices).

Sectional statistical method was employed for the building energy consumption of the main campus of Jiaotong University, making it impossible to obtain the energy consumption data of single

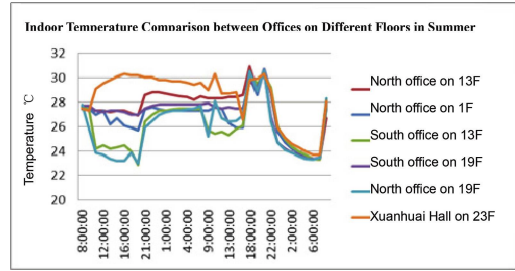


Figure 3. Indoor temperature comparisons between offices on different floors of block E in summer (Source: Self-drawn).

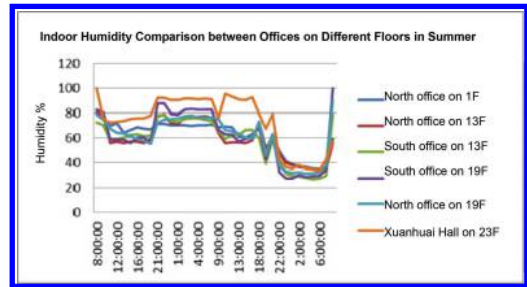


Figure 4. Indoor humidity comparison between offices on different floors of block E in summer (Source: Self-drawn).

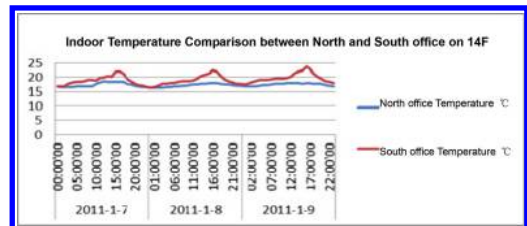


Figure 5. Indoor temperature comparison between south and north offices on 14F of block E (during heating period) (Source: Self-drawn).

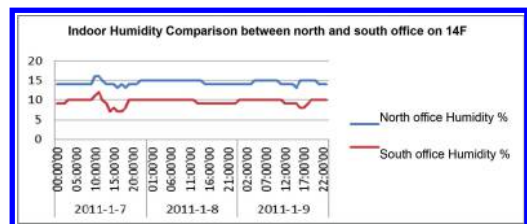


Figure 6. Indoor humidity comparison between south and north offices on 14F of block E (during heating period) (Source: Self-drawn).

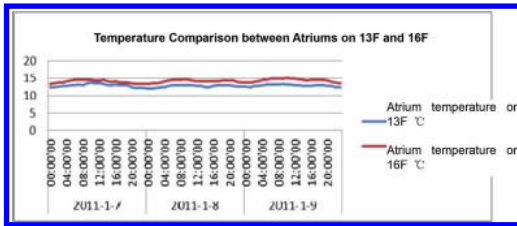


Figure 7. Temperature comparison between atriums on 13F and 16F of block E (during heating period) (Source: Self-drawn).

Block E of the main building. Nevertheless, it was learnt about that the annual power consumption of the main teaching building was 5564945 kWh, and the power consumption per unit area was 105.11 kWh/m². The heating of main building is primarily dependent upon coal-fired boiler of the university, but due to the absence of heat metering system, it's impossible to get the specific value of annual heat consumption. The average coal-burning energy consumption of 71.68 kWh (m²/a) of Jiaotong University was used as the reference. The main building was provided with heat source in winter by the boiler room near the playground on the southeast, and the heating circulating pump transformed hot water into hot wind which was then sent by fan coil into the rooms for heating, so not all the air-conditioning equipment was running, and consequently the power consumption of air-conditioning equipment in winter is less than that in summer by 1/2.

3 PRINCIPLES AND TECHNICAL SYSTEM STUDY OF ENERGY-SAVING TRANSFORMATION

3.1 *Energy-saving transformation principles for existing buildings in cold climate zone*

Based on the energy consumption characteristics of existing buildings in universities and the transformation requirements for buildings of different times, the study proposes the principles for the transformation of existing buildings in universities, serving as the guidance for the implementation of transformation design.

1. Principle of integrity

The local climatic conditions, surroundings, buildings, indoor conditions, and structure should be taken into account as a whole to achieve an integrated transformation.

2. Principle of appropriateness

Appropriate energy-saving technologies suitable for local climatic conditions, economic development

level, resource conditions, and transformation object's basic conditions should be selected (Hong 2009);

In the process of energy-saving transformation, the ecological environment of surroundings should be valued and the impact of such factors as wind environment, sunlight, and so on taken into consideration. Positively utilize renewable resources, and reduce dependence on non-renewable ones. Select ecological building materials as many as possible and make sufficient use of old and local materials;

The energy-saving transformation should be aimed at elevating the human comfort level. The standards for indoor thermal comfort should be raised and the indoor thermal environment improved;

The indoor thermal comfort of existing building should comply with the standards for human thermal comfort. The indoor thermal environment of building should be improved through the energy-saving transformation to bring it to the range of human thermal comfort under local climatic conditions.

3. Principle of economic efficiency

Appropriate technologies of relatively high cost performance should be selected to shorten the investment payback period as much as possible (Liu et al. 2014).

4. Principle of authenticity

The energy-saving transformation of building's facade, space, and equipment should be realized with new building materials and technologies on the premise of respecting history and current situation and not changing the existing of building (Wu et al. 2007). The load on roof and wall should be lightened as much as possible, and the original application function of building should not be affected. For any transformation object with historical value, the historical style and features of original building should be reserved.

5. Principle of safety

The safety of construction should be guaranteed during building transformation; the safety of building structure and fire evacuation should be ensured during use in the future; the energy-saving technologies selected should minimize building load and ensure the safety of building structure. Materials of good fireproof performance should be selected (Wang 2007).

3.2 *Technical system for energy-saving transformation of existing buildings of universities in cold climate zone*

Once the transformation principles are established, the study on energy-saving transformation technologies can be started. The energy-saving

Table 2. Energy-saving technical system table for existing public buildings of universities in cold climate zone (Source: Self-drawn).

Recommended energy-saving technologies for building envelope	Recommended energy-saving technologies for renewable energy sources	Recommended energy-saving technologies for HVAC system	Recommended energy-saving technologies for architectural environment control system
• External thermal insulation, internal thermal insulation, sandwich thermal insulation • Respirable curtain wall, • Double-layer glass curtain wall • Double-layer hollow LOW-E glass • Double-layer vacuum LOW-E glass • Multi-layer vacuum LOW-E glass • Ceiling insulation • XPS-insulated roof, • Inverted roof, • Green roof, • Cold roof, evaporative cooling roof • Roof ventilation system • External sunshade (louver, grating, and roller blind), internal sunshade (louver and roller blind), glass self-sunshade, vertical greening sunshade, push-pull plate sunshade, automatically adjustable sunshade, PV panel sunshade • Adding anteroom • Increasing air tightness of door and window • Different window-wall ratios for different facade designs	• PV panel • Solar house • Solar chimney (general cap) • Heat recovery cap • Tunnel wind (pool, soil cooling) • Solar wall • Solar hot water • Independent VRV cooling/heating heat pump-type system • Ground-source heat pump + Primary air system • Water-source heat pump + Primary air system	• Improving the efficiency of boiler and pipe network: insulating the heating and cooling pipelines; transforming the heating system from single-pipe type to double-pipe type; high-efficient radiator • Optimizing cooling and heating systems; realizing modularization transformation (with air conditioner) • Recovery of heat emitted by air conditioner • Primary air system • Radiant cooling ceiling • Under-floor air supply • Capillary heating and cooling • System of combined supply of cooling, heating, and power • Air conditioner of independent temperature-humidity control • Variable-Air-Volume (VAV) air-conditioning system • Adding thermal meter	• Manual window-openable atrium + Sunshade • Intelligent window-openable atrium + Sunshade • Prism glass window • Reflector • Draft staircase • Skylight • Enlarging the area of daylight opening • Increasing energy-saving lamps • Light pipe • Optical fiber • Intelligent lighting control system

Table 3. Transformation schemes for Block E of the main building of Xi'an Jiaotong University (Source: Self-drawn).

Problem factor classification		Scheme 1: Energy-saving technologies adopted by integrated scheme	Scheme 2: Energy-saving technologies adopted by high-cost-performance scheme	Scheme 3: Energy-saving technologies adopted by economical scheme
Lighting	Extremely poor lighting in the indoor atrium	Prism light-guiding system Adding PV panels on roof and providing lighting electricity Enlarging the glass area of interior wall of atrium	Prism light-guiding system Adding PV panels on roof and providing lighting electricity Enlarging the glass area of interior wall of atrium	Reflector Enlarging the glass area of interior wall of atrium
Ventilation	Almost no ventilation in the atrium Poor indoor ventilation in office space	Connecting all indoor atriums and setting vertical draft well Transforming the top of atriums between 16F and 19F into openable type Transforming the south-facing glass curtain wall into exterior respirable curtain wall	Connecting all indoor atriums and setting vertical draft well Transforming the top of atriums between 16F and 19F into openable type	Connecting all indoor atriums and setting vertical draft well Transforming the top of atriums between 16F and 19F into openable type
Thermal comfort	Very low temperature in Xuanhuai Hall on the top floor in winter Very low temperature in the entrance hall on 1F and the lobby on 2F and serious permeation of cold wind in winter	Transforming the single-glazed curtain wall into exterior respirable curtain wall Enhancing the thermal insulation performance on the floor Setting anteroom	Transforming the single-glazed curtain wall into double-layer type Enhancing the thermal insulation performance on the floor Setting anteroom	Enhancing the thermal insulation performance on the floor Setting anteroom
System	Inability of 1 centrifugal pump to satisfy normal cooling load. Modular design unavailable and high power consumption Excessive dependence of heating in winter on natural gas boiler and no guarantee of indoor comfort Relatively poor fan quality, affecting indoor ventilation	Optimizing cooling and heating systems and realizing modularization transformation Providing cold/heat source by using underground ground-source heat pump Radiant cooling ceiling Under-floor ventilation system Recycling system for exhaust heat of air conditioner	Optimizing cooling and heating systems and realizing modularization transformation Raising the efficiency of boiler, air-conditioning unit, and pipe network Replacing or repairing fan coil Recycling system for exhaust heat of air conditioner	Optimizing cooling and heating systems and realizing modularization transformation Raising the efficiency of boiler, air-conditioning unit, and pipe network Replacing or repairing fan coil

technologies can be classified into four categories, respectively for building envelope (wall, window, roof, and sunshade), HVAC equipment system (heating and air-conditioning system), architectural environment control system (natural lighting, building ventilation, and building lighting), and renewable energy sources (passive, active, and biomass).

Table 2 shows the technical system formed by incorporating the technology clusters established based on the climatic characteristics of cold climate zone with those of energy-saving transformation of existing buildings.

The selection of energy-saving technologies above takes into account the characteristics of buildings of different types and times on the one hand. For example, the internal thermal insulation of exterior wall is applicable to buildings whose exterior facade needs protection, i.e., cultural relic buildings of the 1950s and the 1960s (Li 2010); the same thermal insulation is also suitable for teaching and office buildings, because they are mainly used in the daytime, when the indoor temperature rises quickly during heating and heat is accumulated and obtained within a short time; thus external thermal insulation, which radiates heat into the room at night, is not needed (Zhou & Zhao 2013).

On the other hand, the selection also takes into consideration of the diversity of technologies to meet the demands of buildings of different types and under different situations. For instance, different sun-shading modes at different levels are given, from elementary external sun-shading louver to hi-tech automatically adjustable sunshade and PV panel sunshade, to satisfy different needs and investment scales.

Take the transformation of Block E as example. Different technologies are combined into three available transformation schemes, namely, integrated one, high-cost-performance one, and economical one. The integrated scheme mostly adopts new or innovative energy-saving technologies and represents the advancement of energy-saving transformation; the high-cost-performance scheme consists of popular energy-saving technologies for the most part and acts as a energy-saving transformation strategy of moderate construction cost and fine effect; the economical scheme largely employs basic or common energy-saving technologies, as indicated by Table 3.

4 CONCLUSION

It can be seen from the study above that the existing buildings of universities in cold climate zone, as a major energy consumer, have great potential

in the aspect of energy-saving transformation. In addition, buildings of different times take on diverse energy consumption characteristics, so in-depth survey, test, and simulation are necessitated to formulate appropriate energy-saving transformation schemes. The established energy-saving transformation technology clusters for existing buildings of universities in cold climate zone can meet the needs of different buildings and transformation investment funds.

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Research on architecture and structure integrated design of temporary venues aimed at low carbon construction

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ABSTRACT: Based on the background of sustainable development, in order to reduce carbon emissions, more and more temporary venues are being built for large-scale sports events, expo activities. These venues boast characteristics such as light-duty, reusable, convenient in assembling and disassembling, economy applicable, and so on. As the research object with the architecture and structure is the core problem, the integrated design, aiming at achieving the goal of construction of low carbon venues is put forward in this paper. First, the connotations of temporary venues and integrated design are probed. Second, the connotation of integrated design is summarized, through the analysis of a large number of design examples and the compare with permanent buildings. Then, the process of integrated design of architecture and structure for temporary venues is put forward. Last, some key points and design strategies are given.

1 INTRODUCTION

The development of society and the improvement of standard of living promote the evolution of large-scale sports events and expo activities. This led to construction of a great number of venues (see Fig. 1). However, the resource consumption during the construction of the venues is contrary to the low carbon construction thought. Temporary venues are short-term built and recyclable. They correspond to the low carbon construction thoughts and circular economy. The construction of temporary venues becomes the key method to solve the problem above-mentioned. It has wide development prospect. The gradually matured construction technology release temporary venues from

single structure and high-cost. The conditions are provided for the venue construction aimed at low carbon construction.

Buildings and structures are interrelated and inseparable as a whole (Weinong 2009). This is especially intensive in temporary venues. On one hand, buildings rely on constructions to set up space, build architectural image and achieve function. On the other hand, temporary venues need to be constructed and dismantled in the short term. This leads to a high complexity of technology problem. Both buildings and structures in the design phase need to be considered overall. Through the research on relevant theory and design countermeasure of integrated design of architecture and structure on temporary venues, this article aimed at optimizing venue properties and realizing low carbon building and using target.

2 RELEVANT CONCEPTS

2.1 Temporary venues

According to durable years, buildings can be divided into permanent buildings and temporary buildings. Temporary buildings are constructed casually with simple structures and can be dismantled within the allotted time. Venues include sport venues, exhibition venues, performance venues and so on. They serve for social public activities. Temporary venues are in the service of temporary public events. They share the characteristics of temporary buildings and venues (see Fig. 2). According to the concepts of temporary

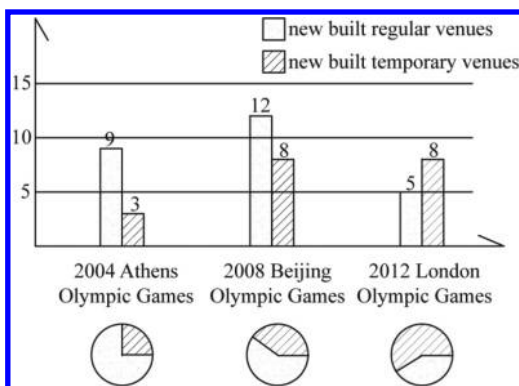


Figure 1. The proportion of the temporary venues in the near three Olympic Games.

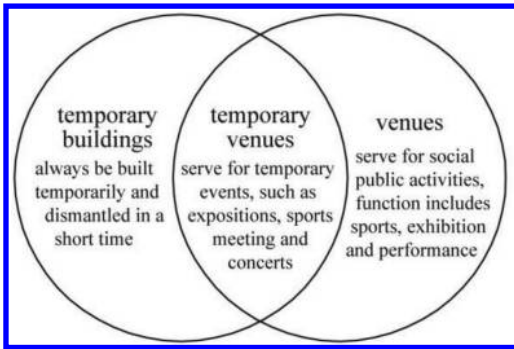


Figure 2. The concept of temporary venues.

buildings and venues, there is a conclusion that temporary buildings serve for temporary social public activities such as expositions, sports meetings and concerts and they could be constructed and dismantled within a short period of time. The space of some permanent venues could be divided convertibly. These venues have certain characteristics of temporary venues. But they do not belong to the research range of this article. Typical venues of this article are the venues with temporary main structures such as Basketball Arena and Shooting Range of London Olympic Games, temporary venues of Chichester Drama Festival, the temporary theater of Cannes Film Festival and other temporary venues in Shanghai expo. They serve for temporary public events such as Olympic Games, drama festival, the filmfest and the World Expo. The existing time of them are in the range of one month to ten mouths.

2.2 Integrated design

Integrated design idea first appeared in Japan in the 1970s. The early research is limited to electro-mechanical integration design. With the emergence of other all kinds of advanced design and manufacturing technology, the research field of integrated design was extended (Jimin 2006).

In the field of architecture, integrated design has two layers of connotation. The connotation in the narrow sense mainly refers to the building, structure and equipment integrated design. That is to consider the building shape, space, function, structure, equipment layout and piping arrangement and other factors together in the process of design. The connotation in the broad sense refers to the whole life cycle, the multidisciplinary collaborative, multi-factor comprehensive, all-round, three-dimensional integrated design (see Fig. 3). The integrated design in this article spans across architecture and structure two subjects. The whole

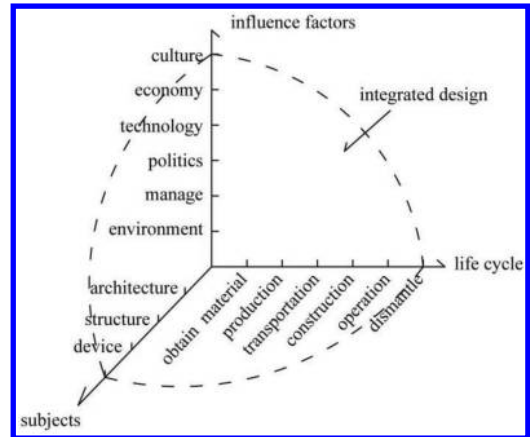


Figure 3. Concept of integrated design.

life cycle of buildings and the factors on all aspects are considered in it.

3 ANALYSIS OF TEMPORARY VENUES CHARACTERISTICS

Grasping the features of architecture and structure is the primary condition of architectural and structural integrated design of the temporary venues. This part analyzes the characteristics of temporary venues by comparing them with those of permanent venues. The permanent venues and temporary venues have some basic characteristics in common, such as practical, economic, security and artistic. In addition, the permanent venues generally are used for many years and they tend to have iconic appearance. The differences between the period during the events and after the events have to be considered into the function of them. The structures of permanent venues are solid and the lightening and convenience of construction process to a certain degree is demanded. With the goals of quick construction and serving for public activities in a short period of time, the temporary venues have characteristics of cyclicity and timeliness on service time aspect. Mobility according to requirements is sometimes required. Also, entire disassembly, convenience and rapid disassembling process are demanded on structural aspects. Furthermore, the temporary venues have high requirements for structure lightening and short-term recycling. The characteristics of venues for Shanghai expo and London Olympic Games are summarized in Table 1. The definite interpretation of the characteristics of permanent venues and temporary venues is in Table 2.

Table 1. Characteristics of venues for Shanghai expo and London Olympic Games.

	Architectural characteristics			Structural characteristics				
	Event-ness	Timeliness	Mobility	Durability	Recycling	Light-weight	Convenience	Disassemblability
<i>Pavilions of Shanghai expo</i>								
Switzerland	•	•	•	•	•	•	•	•
France	•	•		•	•	•	•	•
German	•	•		•	•	•	•	•
Japan	•	•		•	•	•	•	•
Norway	•	•	•	•	•	•	•	•
Britain	•	•		•	•	•	•	•
Korea	•	•		•	•	•	•	•
New Zealand	•	•		•	•	•	•	•
Poland	•	•		•	•	•	•	•
Canada	•	•		•	•	•	•	•
Nepal	•	•		•	•	•	•	•
United Arab Emirates	•	•		•	•	•	•	•
Holland	•	•		•	•	•	•	•
Luxembourg	•	•	•	•	•	•	•	•
Finland	•	•		•	•	•	•	•
America	•	•		•	•	•	•	•
<i>Venues of London Olympic Games</i>								
Stadium	•	•		•	•	•	•	•
Basketball hall	•	•	•	•	•	•	•	•
Shooting hall	•	•	•	•	•	•	•	•
Water polo arena	•	•	•	•	•	•	•	•
Beach volleyball court	•	•		•	•	•	•	•
Hockey field	•	•	•	•	•	•	•	•

4 INTEGRATED DESIGN METHODS

4.1 Design philosophy

Based on the analysis above, collaborative design and whole life cycle design are two important aspects of integrated design. Collaborative design is for two or more design agents to complete design task respective and the overall design by information exchange and management mechanism. The core of collaborative design is information exchange and management. The problem to solve is cooperation of multiple professional in the field of architecture. Whole life cycle is the process from building up to tearing down. It is usually divided into five phases which are plan, design, construction, utilization and dismantling. It is used in research about building energy efficiency and low carbon. Temporary venues integrated design of architecture and structure involve architects and structural engineers two professionals, and possess complicated life cycle of frame. The philosophy combined collaborative design and whole life cycle design should be used.

4.2 Design process

Through combing the philosophy of collaborative design and whole life cycle design, and taking multiple influence elements into consideration, temporary venues integrated design of architecture and structure can be summarized in to a composite process. The process includes a number of subjects and whole life cycle under some influence of multiple elements (see Fig. 4). First, analysis of whole life cycle should be made to have a macro control. Second, design content can be carried out with a comprehensive consideration of the influences to all phases. Last, through the coordination of all phases the final design can be built. The three points which are the plan of the whole life cycle, the consideration of multiple elements and the combination of architecture and structure are the key factors of the design.

5 INTEGRATED DESIGN STRATEGIES

Through combining the relevant concepts with the analysis of the characteristics of temporary venues,

Table 2. Characteristics of permanent venues and temporary venues.

	Permanent venues	Temporary venues
Practical	• Contain function	
Economic	○ Construction, operation cost	• Construction, operation, dismantle cost
Security	• During and after events	
Artistic	• Building image	
Eventness	• Image and function	
Timeliness	◆ Differences between period during and after events	• Cyclicity and short-term
Mobility	Special conditions	○ Part of the venues
Durability	• Whole	○ Part
Recycling	◆ Long-term	• Short-term
Light-weight	○ Low demand	• High demand
Convenience	○ Long-term, aimed at security	• Short-term, aimed at quick construction
Disassembility	○ Part	• Whole

- the type of venues have this characteristic.
- ◆ the type of venues have this characteristic, but in different form from the other.
- the type of venues partially have this characteristic.
- the type of venues does not have this characteristic.

the conclusion can be made that the core elements of low carbon construction of temporary venues are material saving, efficient building and cyclic utilization. Design strategies should focus on the key points on structure selection strategies, material selection strategies and node design.

5.1 Structure selection strategies

Light-duty design, constructing facilitation and component standardization are the main points of structure selection. Also space use, image expression and economy should be considered together, through realizing the efficient and recycled utilization of structure to achieve low carbon targets. The light-duty design is to use lightweight structure, lattice work to reduce weight of structures and the burden to build. Constructing facilitation is to use factory-made component, whole assembly way to enhance the efficiency of construction. Component standardization is to use components with the same modulus, uniform standard to build. The aim is the convenience to be put into other projects or other sites. In addition, the structure such as cable dome is novel and light. But due to the limitation of expensive construction cost, it is not appropriate for the temporary structure at present.

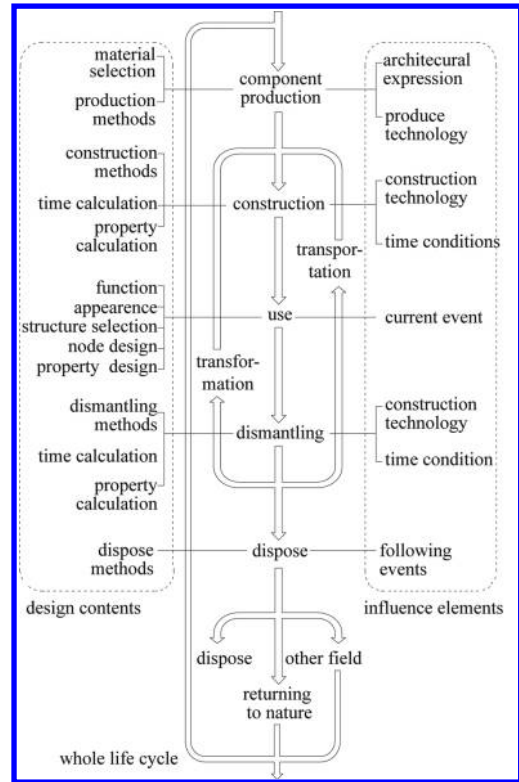


Figure 4. Design process.

China pavilion of Milan expo used wood as main structure. This is in line with the light-duty structure selection strategy. The theme is the fields of hope, the source of life and the venue itself is part of the show. The appearance of the venue is like wheat waves, components are transformed to respond to the space (see Fig. 5). Fuji pavilion of Osaka expo was constituted by pneumatic membrane structure which is of light weight as a whole. Air-inflated arch formed space and the dismantle process was convenient and monolithic (see Fig. 6). The shooting range of the London Olympic Games consists a set of standard light steel truss to constitute its modular structure (see Fig. 7). The structure is widely used, could be rent from the temporary engineering company. After the Olympic Games, the main structure of building was returned to the leasing company, and the steel pile foundations were pulled out from ground, leaving no waste or traces (Burke 2013).

5.2 Material selection strategies

Lightweight, ecology and recyclable are the main points of material selection of temporary venues.



Figure 5. China pavilion of Milan expo.



Figure 6. Fuji pavilion of Osaka expo.



Figure 7. Shooting range of London Olympic Games.

Also the material performance, function adaptability and expression of events ought to be considered as a whole. The corresponding relations between the various conditions of material and demands of venues should be attached importance to, such as material lightening and venues building, material durability and recycle of venues, etc.

Three indoor venues of London Olympics all adopted steel and membrane as main material. The advantages of the characteristic of lightweight and strong durability were taken. The adoption is consistent with the demand to be moved and rebuilt after the games. Japan pavilion of Hanover expo used paper to be the main material. Paper is of low density and could meet the requirements of structural safety performance after processing. Also paper is of weak durability and strong regeneration at the same time, this is consistent with the demand of short time use. In addition, the ecology Building

image of paper is a response to the unique demand of exhibition buildings.

5.3 Node structure strategies

Flexibility and simplicity are the points of temporary venues node design. Low carbon targets could be implemented through improving the efficiency of disassembly and installation. Also, the construction process should be considered. Bolting nodes can realize disassembly, moving nodes can realize convenience and simple nodes can reduce the construction cost through public participation.

Japan pavilion of Hanover expo used gridshell structures as structure. The structure has a special building process (see Fig. 8). The nodes could rotate around and they were tied up with steel wire and rope. This provided support for the structure to transfer from two-dimension to three-dimension (see Fig. 9). The ring dome which was built for the 25th anniversary of New York art and architecture exhibition hall consisted many plastic exercise hoops connected by thousands of bolted node. The nodes are constructed simply, students, artists and some other volunteers could participate in the assembly (see Fig. 10). This brought a party atmosphere in the process of building the venue. The ring



Figure 8. Construction process of gridshell structures (Tong 2013).



Figure 9. Construction process and node of Japan pavilion of Hanover expo (Tong 2013).



Figure 10. Appearance and construction process of ring dome (Linfei 2010).

dome was rebuilt in Milan and Yokohama afterwards (Linfei 2010).

6 CONCLUSIONS

Low carbon building is a long-term goal for the development of architecture. Based on this goal, the construction with integrated design of architecture and structure is necessary in the current high-speed development period of temporary venues. Facing the opportunities and challenges, the contemporary architects and structural engineers should closely cooperate and seek breakthrough in the theory and practice of integrated architectural and structural design about temporary venues. The aim is to construct comprehensive, efficient and

promising temporary venues and provide support for the construction of low carbon.

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Building energy management based on occupant location

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ABSTRACT: Reducing building energy consumption has received growing attention in the recent decades. The primary energy consumption in buildings is related to heating, ventilation, cooling, and lighting systems. A well-design control system may result in a significant reduction of energy usage. This paper presents and implements three strategies to control and analyze energy consumption in buildings. The schedule, real-time monitoring and a multi-agent system are compared with the baseline energy consumption in a university office. Devices and human agents are introduced to investigate the real-time trend in energy usage and to optimize the consumption of energy, based on the location of occupants. Initial results demonstrate potential energy saving of 16% in the office.

1 INTRODUCTION

Reducing building energy consumption receives growing attention in the current decades. About 40% of global energy usage is linking with buildings (Lausten 2008). In China around 25% of national energy consumption belongs to buildings (Lam, Tsang and Yang 2006). In addition, about 36%, 18% and 10% of energy in buildings are consumed by HVAC systems, lighting systems and electrical appliances respectively (Department of Energy 2009). Nowadays, people spend most of their time in buildings and human productivity, learning, health and happiness, are influenced by buildings (Sensharma, Woods and Goodwin 1998). Therefore, building energy and occupant locations are two significant parameters by which the performance of a building can be analyzed and improved. Traditional Building Management Systems (BMS) do not support real-time input or dynamic parameters such as occupancy area, occupant thermal or lighting preference and user behavior and actions. The behavior, location, decision and action of occupants impact the energy consumption in buildings. Consequently, for optimization of energy in buildings, it is essential to consider and collect these parameters. This paper presents and implements a Multi-Agent Energy System (MAES) to compare alternative managements and different control of building systems. The aim of the MAES is to manage input from human and indoor and outdoor environment and show the potential of reducing building. MAES relies on the real data including indoor and outdoor temperature and humidity collected by Arduino, occupant preferences, occupant schedules and position of occupants recorded by wireless sensor network.

A public office with 9 occupants in a university is selected as the test bed. The operation of this room is evaluated based on four scenarios: Baseline, Schedule, Real-time and the combination of real-time and schedule. Finally, the energy consumption of four scenarios are assessed.

2 OCCUPANT-DRIVEN CONTROL

The operation of current BMS is mostly dictated according to the period of the day based on the fixed pre-defined schedule. The setting point of HVAC systems and lighting are adjusted by the assumption of maximum occupancy in each area without considering the partially occupied situations. However, there is a great capacity to reduce energy usage in buildings by taking into account the actual occupancy loads.

There are a wide range of occupant detection technologies such as motion sensors (Jackson, Callahan and Wang 2006), Radio Frequency Identification (RFID) (Li, Calis and Becerik-Gerber 2012), Wireless Local Area Network (WLAN) and Wireless Sensor Network (WSN) (Pazhoohesh and Zhang 2015). The theoretical simulation of real-time occupancy observes that the potential of 10–20% saving energy in buildings (Agarwal, et al. 2010) (Erickson and Cerrpa 2010). In this research a ZigBee based Wireless sensor network is implemented for the occupants identification.

Adding to the BMS inefficiency, temperature set points are uniform for the whole part of the building. Therefore, parameters such as occupancy preference and sun exposure do not take into account.

Moreover, a wide range of research has shown a difference between code-defined comfort ranges

and what occupants reported as their comfort ranges (Barlow and Fiala 2007) (Corgnati, Filippi and Vaizzo 2007). Allowing a more flexible control for air conditioning systems may result in decreasing this difference which can lead to more saving energy in buildings. The feedback of occupants from their comfort ranges can be a suitable guideline for the future plan of HVAC systems. By the development of smart phones, it is possible to design a mobile application which provides a way for occupants to address their comfort level through direct communication for providing feedback to the BMS.

3 MULTI-AGENT SYSTEMS

Unlike the intelligent systems which the Intelligent Agents (IA) can interact autonomously in an environment by perceiving their environment through sensors and direct the activity toward a specific goal.

In Multi-Agent Systems (MAS), it is possible for agents to communicate and harmonize with each other and with the environment. Multi-agent system are therefore utilized to find a solution for complex problems which cannot be solved by an individual agent. The architecture of MAS is based on dividing the main problem into sub-problems and solves those sub-problems by the representative agents. The combination of solutions for sub-problems may change the current state through agent to agent coordination (Dounis 2010). Figure 1 represents a MAS model in a given environment. Several agents can perceive the environment conditions and make a decision through their reasoning capability.

3.1 A MAS in building energy management

MAS can be utilized for an intelligent control system to learn environment parameters and occupancy

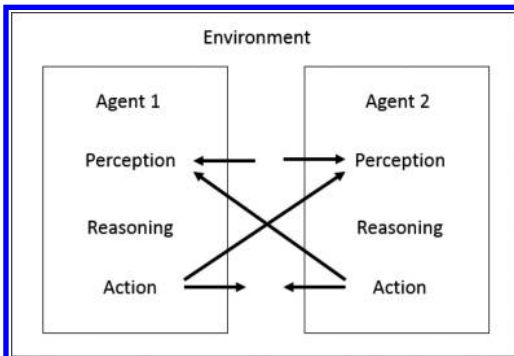


Figure 1. A MAS model served by several agents in a given environment.

trends, and decide which actions (if any) are to be taken. MAS provides a cooperation between air conditioning systems and lighting with the goal of optimizing energy on a smart grid (Ramchum, et al. 2011). They can also be utilized to manage energy cost and renewable energy supply systems (Zhao, Suryanarayanan and Simoes 2010). In smart home and office, MAS has developed to study the distribution of energy demand in buildings and manage and optimize the control of HVAC and lighting systems (Joumaa, et al. 2011). MAS can also provide a dynamic simulation of buildings energy demand. For example, (Hagras, et al. 2008) developed a learning system to respond the internal occupancy loads based on the external climate and maintaining comfort level for occupants to reduce energy consumption in buildings. Liao and Barooah (Liao and Barooah 2010) have developed a MAS prediction system to estimate the occupancy and simulate building occupants for different zones and rooms.

Although these frameworks illustrate a promising results in MAS and energy management in buildings, there are still some points which are needed to be considered. Most of the research are based on simulation results which do not observe the complexities of an actual performance. Most of those systems are only considered the occupancy based on the daily schedule which is not an accurate input for the whole day. Additionally, they only consider the occupant preference as an input parameter to control the HVAC system and ignore the historical behavior of occupants in the previous similar situations.

3.2 Agents definition

The developed MAES include three phases: Simulation module, Input/output module and reasoning and planning module. The input/output module is developed to communicate with agents and collect data and create the world model. According to the world model, the policies are generated in the layer of reasoning and planning module to achieve the coordination between objectives. The generated policies and the world model leads to modeling the physical and behavioral interactions between agents in the simulation module.

The agents in the simulation include device agents and human agents. The device agents consist of lighting agents, appliance agents and air conditioning agents. The lighting agents control the lightings in the room based on the on/off switch and the position of the occupant. The HVAC agents demonstrate the temperature and ventilation in the room based on the implemented sensors in each zone and control of air temperature and flow rates. Appliance agents represent desktop and laptop computers and perform according to “on”,

“off” and “standby” states. To identify occupancy of the room in the office, humane agents include permanent occupants for researcher and temporary occupants such as staff from other rooms. Each human agent include five subsets: wander, perform research, go to the coffee room, go to the meeting and leave the building. Depend on the location of researcher in the building, one subset is selected for human agents. A stochastically temperature preference is determined for each user (normal distribution between 18.5°C and 26.5°C)

4 PHYSICAL TEST BED INFORMATION

The proposed MAES is tested and analyzed in a public office in the Xian-jiaotong-Liverpool University. The office occupied by 9 research staff and the area is divided into two thermal zones based on the number of HVAC system (middle to North-middle to South). Lighting power loads are also divided into two parts. The floor and window areas for each zone are extracted from the building plan as well. Outdoor and indoor temperatures and humidity are recorded at the test bed room over a period of 24 hour in July 2015. The position of occupants also recorded during this date.

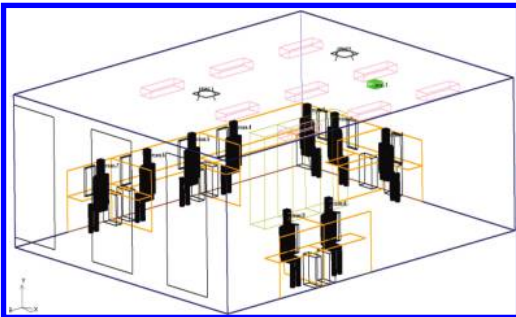


Figure 2. The geometry of office room with nine occupants.

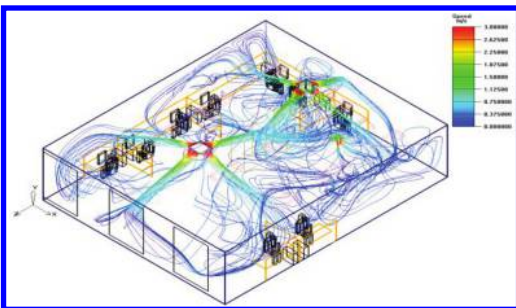


Figure 3. Indoor airflow streamlines from the air conditioning to outlet window.

Figure 2 represents the geometry of office with nine occupants. And Figure 3 illustrates the streamlines from HVAC systems generated by Computational Fluid Dynamics (CFD) software.

5 ROOM SYSTEM CONTROL STRATEGIES

The energy consumption is calculated based on agents and four strategies related to HVAC, appliance, lighting and occupant locations. The first control strategy is called baseline which represents the assumption that two set points are regulated for occupied condition (20°C–24°C) and unoccupied period (18°C–30°C) of the day based on the daily working time and the lighting system is adjusted based on the daily schedule. Appliances are considered as “on” situation during the working hours. The second scenario is called “occupancy schedule” strategy which is based on the historical behavior of the users in the room.

Device agents predict and adjust lighting and temperature based on occupancy schedule and the known occupant’s preferences.

In the third control strategy which is called “real-time” monitoring, when at least one user stays in the related thermal zone, the appliances and lighting assumed to be turned on in that specific zone. The HVAC systems respond to actual occupancy and occupant preferences identified in each thermal area during the day. When all users leave the building, the system turned off the lighting, HVAC system and appliances. According to the data extracted from tracking system, when user go to the coffee room, if the thermal zone is empty, lighting is turned off. If the user go to the meeting room or other classrooms, the airflow and temperature of HVAC system is adjusted to the lower energy consumption conditions. If all users leave the building, the system will turn off all electrical appliances and HVAC system and lighting.

The last strategy is called “Multi-Agent” strategy (MA). In this strategy, the control system utilize both occupants schedule and real-time information to predict energy consumption and occupant’s preference and optimize the energy usage based on the behavior of occupants.

6 ENERGY CONSUMPTION IN THE OFFICE

The energy consumption estimated according to the outcome of each strategy. The heat loads in the office is demonstrated in Table 1.

The energy consumption for HVAC system is estimated based on the total thermal loads in this office. Total thermal loads are calculated as the

Table 1. Heat loads in the office (w).

Agent type	Category	On	Off	Standby	Q
Lighting	Office	120	0	0	120
Appliance	Desktop	150	0	10	150
	Laptop	70	0	7	70
Human	Male	—	—	—	75
	Female	—	—	—	65

sum of heat gains from seven male and two female in the room, their computers, seven lightings, solar gains and transmission through windows in perimeter zones.

Window transmission (q_{trans}) and solar gains (q_{solar}) are calculated according to [equation 1](#) and [2](#).

$$q_{trans} = 0.5A_{zw} \times (T_{OSA} - T_z) / 3.412 \text{ (CFM)} \quad (1)$$

$$q_{solar} = SG \times A_{zw} \text{ (CFM)} \quad (2)$$

where A_{zw} is the area of windows and T_{OSA} in the real-time outside temperature and T_z is the real-time indoor temperature and SG is the maximum solar heat gain in the room. To convert Btu/hr to Watts, the constant 1/3.412 is used.

Based on the predefined zones in the room and thermal heat load in each zone (Q_z), the HVAC energy consumption is calculated. According to ASHRAE 62.1, the minimum outdoor ventilation rates (V_o) is computed based on the real-time occupancy. The supply air flow is computed according to the equation 3.

$$V_s = [(1.1 \times V_z \times \Delta T) + Q_z] / [1.1 \times (T_z - T_s)] \text{ (CFM)} \quad (3)$$

where V_z is the zone volume and T_z is the zone temperature and T_s is the temperature of supply air.

The total HVAC energy consumption (Q_{total}) is the sum of fan distribution energy (QFAN) and Chilled water cooling energy (QCHILL).

$$Q_{FAN} = 1.25 \times V_s \text{ (w)} \quad (4)$$

$$Q_{CHILL} = 0.06 \times 1.1 \times (T_{MA} - T_s) \times V_s \text{ (w)} \quad (5)$$

where T_{MA} is the temperature of mixed air and considered as a linear function of the ratio of temperature of outside and the temperature of the zone.

The compact size of monitoring and control system is shown in [Figure 4](#). This system is adjusted for different strategies. The energy consumption is calculated and saved in a log file on the server. The HVAC systems are controlled by the infrared sensor on the board.



Figure 4. Smart-controller panel.

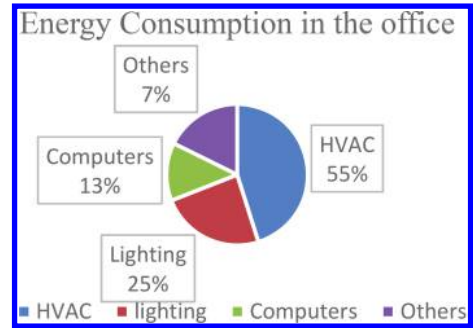


Figure 5. Energy consumption for different appliances.

7 EVALUATION OF RESULTS

To investigate the energy consumption in terms of different applications, a pie chart is depicted in [Figure 5](#). The significant parts of energy consumption in this office allocate to HVAC systems and lighting. Therefore, energy management based on occupant's location may result in saving energy.

Running the proposed strategies over 24 hour period shows reductions in energy usage for three scenarios in comparison to the baseline control system ([Fig. 6](#)). The real-time and MA realized the most significant energy consumption reduction with 14.65% and 16% saving, respectively. The schedule control saw an energy saving of 5.3% as compared to the baseline. The most saving energy were achieved during the combination of real-time and occupants schedule strategy which is known as multi-agent strategy. The results concluded that creating a personal schedule with the current location of the users may lead to more saving energy for control systems. As shown in [Figure 6](#), during

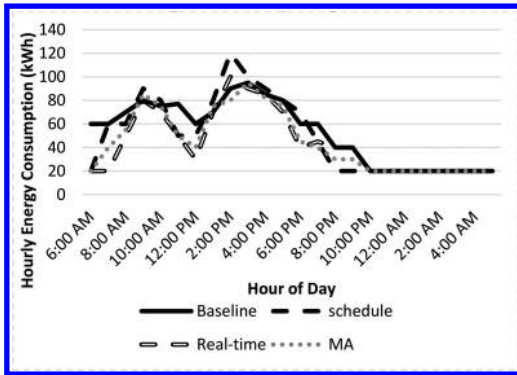


Figure 6. Office energy consumption over 12 hour period with four strategies.

the peak time, 1–3 PM, the schedule strategy consumes more energy than the baseline. It can be explained as different behavior of occupants during the peak time.

8 CONCLUSION AND PERSPECTIVE

The present work is focused on the study of energy management and control systems in buildings. Three schedule, real-time and multi agent strategies are compared with the baseline energy consumption in a university office. The schedule method controls HVAC systems and lighting based on the daily schedule of occupants in the office. The real-time method controls air conditionings and lighting systems and appliances based on the real-time location of occupants in the building. The results provide about 16% saving energy for multi-agent system and 14.65% reducing energy for real-time strategy. The results provide motivation to investigate multi-agent systems for energy management in buildings. The future plan is to extend the multi-agent system for a whole building to investigate the effect of this strategy for large-scale test bed.

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Estimating Embodied Carbon: A dual currency approach

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ABSTRACT: Embodied Carbon (EC) estimating is a growing area of research. EC estimating is evolving since the recent past, thus, undergoing several criticisms. On the other hand, there is a lack of interest among industry practitioners on EC estimating and reporting. However, EC estimating is crucial in a world of swiftly changing climate and visible adverse consequences. Therefore, this paper covers important aspects of EC estimating and explains various sources and methods of estimating, ranging from complex to simple methods of estimating. EC estimating is further explained by case studies of seven office buildings. Further, problems like lack of a standard method of measurement, lack of a robust database/benchmark for EC, lack of EC data for modern products and lack of EC data for sophisticated services are discussed. More importantly, the paper introduces dual currency appraisal of construction projects in which EC estimating plays a crucial role.

1 INTRODUCTION

1.1 *Carbon emissions of buildings*

Emissions from buildings are categorized into mainly two types: operational carbon/emission and embodied carbon/emissions. Operational carbon includes the emissions resulting from fossil fuel usage during the operation of the building (heating, ventilating, air-conditioning and lighting) while embodied carbon accounts for emissions resulting from fossil fuels usage and process which includes raw material extraction, manufacturing of building materials and products, transport, construction on site, repair and maintenance of the building, replacement of any components of the building, and finally demolition and disposal of the building at the end-of-life. Embodied Carbon (EC) covers a larger scope in building life cycle while Operational Carbon (OC) covers nearly three quarter of total emissions. Further, UK-GBC (2014) reported that buildings contribute for nearly half of the UK's emissions of CO₂.

1.2 *Significance of estimating*

Climate change due to the increased concentration of CO₂ in the atmosphere created the rapid need for reducing emissions from all sources. As a result Kyoto Protocol, an international treaty for emission reduction, requires 80% emission reduction from 1990 levels by 2050 and sub targets assigned to signatories (UNFCCC 2014). UK as one the signatories leading the way forward by ambitious targets through Climate Change Act 2008, setting own targets for the country. Accordingly, first

milestone is zero-carbon homes from 2016. However, inclusion of existing stock for the target is remaining undecided. The next target is the 2019 target for zero carbon buildings. Improvements are made to the Part L of the Building Regulations to achieve the above mentioned targets. In that, OC of the building is measured and checked for compliance. Further, the low carbon route map for built environment in the UK sets 34% reduction by 2020 followed by 50% and 80% reductions in 2025 and 2050 respectively. Of which 21% reduction of EC by 2022 and a 39% reduction by 2050 is expected (The Green Construction Board 2013).

This clearly indicates that from 2019 buildings in the UK are expected to be zero carbon building which implies zero OC. Also when attempt is made to reduce OC, most likely EC increases (RICS 2014, Ramesh et al. 2010). Furthermore, there are no any regulatory measures for mandatory measuring and reporting of EC. This highlights the danger of aggravating EC unless it is measured and reported. In fact, to control EC it has to be estimated first. Therefore, estimating EC becomes prominent at this point of time. However, EC estimating is considered to be complex and there is a lack of interest towards it within the industry.

Therefore, this paper intends to break the complexity by explaining fundamentals of EC estimating under the sections of: cost vs. carbon estimating, sources of EC estimating, development of EC estimating, factors to be considered and problems encountered. In addition to that paper also presents case studies of office buildings as an exemplar of EC estimating and also introduces dual currency approach which attests the need of

Table 1. Types of cost estimates prepared during various stages of a project (partially adopted from NRM1).

RIBA 2013 stages	RIBA 2007	Cost plan/estimate	Technique
Preparation and brief	Appraisal	Order of cost estimate	Single rate estimating—unit, superficial area
Concept design	Concept	Formal cost plan 1	Single rate estimating—unit, superficial area, cube
Developed design	Design development	Formal cost plan 2	Elemental estimate
Technical design	Technical design/production information	Formal cost plan 3/ Pre-tender estimate	Approximate quantities
	Tender documentation	Bill of quantities	
	Tender action	Post-tender estimate	Adjusted bill of quantities

EC estimating for construction. Finally, paper also proposes some solutions for the identified problems in EC estimating.

2 COST VS. EMBODIED CARBON ESTIMATING

Cost estimating is one of the core duties of Quantity Surveyor (QS) in the construction industry. The process is well established and governed by industry standards like New Rules of Measurements (NRM2). The detailed estimate is known as ‘Bill of Quantities (BoQ)’ with almost complete details of the project. However, there are various other techniques to estimate cost during different stages of project as prescribed by NRM which is presented in Table 1.

The table above indicates the maturity of cost estimating techniques in the industry. On the other hand, carbon estimating is still a developing branch within the construction industry as it is relatively a new scope of service for QS firms and many regards this as a value added service. As a result, mostly very big firms deliver carbon estimating service to their clients.

Carbon estimating is very similar to cost estimating though it evolved within the past decade. Initially started with Hammond and Jones’ Inventory of Carbon and Energy (ICE) and now it is highly influenced by Franklin & Andrews (2011) UK Building Blackbook. Next section introduces the data sources of carbon estimating available in the UK.

3 SOURCES OF EMBODIED CARBON ESTIMATING

3.1 *Inventory of Carbon and Energy (ICE)*

ICE is an extensive database of carbon and energy data of building materials which was developed by Professor Geoff Hammond (University of Bath)

and Dr Craig Jones (Circular Ecology). The first version was made available in 2006 for free download which then underwent several revisions and version 1.6 was published in 2008. Then, with significant improvement to the previous version the second version was published in 2011 (version 2.0). One of the most important revisions includes, the data had been converted from kgCO₂/kg of material to kgCO₂e/kg of material in the latest version allowing accountability of other GHG emissions (Hammond & Jones 2011).

The database provides embodied carbon and energy data of materials for cradle to gate system boundary. For instance, EC factor of 2.34 means usage of 1 kg of the respective material is accounted for 2.34 kgCO₂e of emissions. Data is derived basically from typical primary energy usage during manufacturing for most of the materials while feed-stock energy is considered in special circumstances. In addition to that, carbon sequestration of materials, if any, is excluded. Also authors state that the data sources include both native and foreign sources which are said to be fairly recent data at the time of database development (Hammond & Jones 2008).

Moreover, the inventory also recognizes the uncertainty in the carbon data due to variant fuel types. Nevertheless, this is the most widely used energy and carbon database for calculations, especially within the UK context, and most EC estimating tools have ICE database as the underlying data source.

3.2 *Hutchins UK building blackbook—small and major works*

UK Building Blackbook is developed and published by Franklin + Andrews based on the ICE database and data from various stakeholders of the construction industry. This is the first book to present both cost and embodied carbon data with respect to building trades in accordance with SMM7 (for major works) and SMM6 (for small works).

Blackbook lists the item description, the resource requirement for unit quantity of the item, and the unit cost and unit carbon of the respective item similar to a price book. The carbon data presented in the book is of cradle-to-gate boundary (see section 5.1). This data source is mainly developed for the use of quantity surveyor who is an expert in construction cost estimating and carbon estimating is no any different to cost estimating.

3.3 Embodied carbon database—WRAP

The Embodied Carbon Database was launched online by WRAP in collaboration with the UK Green Building Council in April 2015. Aim of the database is to capture the embodied carbon data of the buildings within the UK and allow data sharing for effective learning. The database works in a way that industry stakeholders register themselves in the database and upload estimated project carbon data to the database which can be accessed only by the registered users. WRAP and UK-GBC created it as a closed database to prevent misuse of the project information uploaded into the database. There are more than 400 registered users and more than 200 projects stored at present in the database. However, the numbers suggest inactive industry participation.

The database serves as an EC library where projects can be retrieved by specific commands and exported to an excel file for further analysis. Data presented in the database follows the BS EN 15804 standard for system boundary classification (product stage (A1–3), construction process stage (A4–5), use stage (B1–7), end of life stage (C1–4), benefits and loads beyond system boundary (D)). There are very few projects presents data for cradle to grave system boundary at the moment. Further, data are presented in elemental format in which services element is excluded from analysis. However, this is a great asset for EC estimators and researchers to gain insights into EC data. The database can be accessed through <http://ecdb.wrap.org.uk/> (see for more details, WRAP and UK-GBC (2014)).

3.4 End of life dataset of framing materials

PE INTERNATIONAL (an international market leader in sustainability related consultancy and software solutions) developed an end of life (during and after demolition and disposal—C and D modules in BS EN 15804) dataset for common framing materials of buildings like brick, block, concrete and steel. End of life EC is an area which is less researched and lacks sufficient data. This dataset facilitates holistic assessment of framing material selection for building development.

3.5 Department for Environment Food & Rural Affairs (DEFRA) carbon conversion factors

This is an online repository with up to date carbon conversion factors for fuels to calculated carbon footprint of business operations and products. However, this repository is suitable only for UK businesses, researchers and international organizations reporting on the UK operations. This repository allows three options in downloading the factors as an excel file as follows (Department for Environment Food & Rural Affairs 2015):

- Specific data demanded by the user: this option allows users to filter data depending on the scope, fuel or activity type and by the type of data that needs conversion. DEFRA also recommends this option as it eases the process of locating relevant data. However, this option is only available for dataset from 2012.
- DEFRA's frequently used data: this allows users to download pre-filtered factors used by DEFRA frequently for estimating purposes. This includes range of factors which are adequate for average footprint calculations of businesses.
- All available data: this option allows users to download all the factors for a respective year. This option is not recommended by DEFRA for usual carbon accounting while users may be interested in this option for advanced use.

This data becomes useful when estimating EC during construction, use stage and end-of life stage.

3.6 Ecoinvent

Ecoinvent database is developed by the Centre for Life Cycle Inventories. It is an international life cycle inventory database with updated inventory of data from several disciplines, including carbon inventory. The database form as the underlying source of data in many design tools with LCA calculations. The latest version of ecoinvent database is 3.1 with new updates to the inventory and changes to the underlying methodologies. However, database is not freely accessible like other databases discussed above; access is allowed only for registered users (ecoinvent Association 2015).

4 DEVELOPMENT OF EMBODIED CARBON ESTIMATING

Measurement of EC has evolved during the recent past. Initially, Hammond & Jones (2008, 2011) Inventory of Carbon and Energy (ICE) became the fundamental source of reference for EC estimating (cradle to gate) which is composed of dataset of mass CO₂ emissions per mass of set of materials.

Hence, mass of materials constituting the building needs to be quantified to estimate the EC of the building based on a bottom up approach—deconstructing a building element up until the material, labour and plant component and applying ICE embodied carbon factors to arrive at the EC of the building. This is considered to be a tedious task as building constitutes numerous items which needed to be decomposed to follow this method. Furthermore, this process is not very easy to understand.

Therefore, realizing the struggle of the industry RICS published a guidance note for carbon estimating using bottom-up approach. The initial guide on embodied carbon estimating was published in 2012 titled ‘Methodology to calculate embodied carbon of materials’ covering the cradle to gate system boundary. Later, RICS revised the guidance note to cover cradle to grave system boundary and published the revised guideline in 2014. RICS (2014) classifies the project into four main stages according to CEN TC 350 standard namely: product, construction process, use and end-of life stages and also explains the method step by step making it easy to understand. The methodology to be followed in embodied carbon calculations on each stage as per the guidance note is listed in Table 2. Even though the methodology is made clear it remained in reports than in application due to the amount of labour involved in estimating.

However, another substitute for this method is UK Building Blackbook produced by Franklin and Andrews (2011). Blackbook presents itemized EC dataset for standard building items in accordance with Standard Method of Measurement (SMM) which is a familiar method for QSs and a replica of pricing BoQ. This made EC estimating more approachable resulting in few reported case studies on EC estimating by using the book (Halcrow Yolles 2010b, Sturgis Associates 2010, Clark 2013), yet the usage seems to be low.

Additionally, there are various tools and softwares specifically designed for carbon estimating. Some of which are freely available on web (Environment Agency 2012, Build Carbon Neutral 2007, Phlorum 2011, Rocky Mountain Institute 2009, University of Minnesota 2014) while others are licensed (PRé Consultants 2014, PE International 2014). Online tools are useful to understand the accountability of carbon footprint of the building and sometimes of the project. However, the methodology underlying the tools is not transparent which a major drawback of such tools.

5 FACTORS TO BE CONSIDERED IN EMBODIED CARBON ESTIMATING

A major issue encountered in carbon estimating is the variations in measurements. Many scholars noticed variations in EC measurements due to several factors (Dixit et al. 2010, Clark 2013). Therefore, despite the tedious process involved in carbon estimating many other factors need to be considered when preparing an EC estimate. Major factors affecting EC measurements includes: system boundary, method of estimating, assumptions, data sources used and element classification.

5.1 System boundary

This is a distinct feature of EC estimating compared to cost estimating. EC has various system boundaries on which the measurement is based upon. EC can be calculated from cradle (earth)-to-gate (factory gate), cradle-to-site, cradle-to-end of construction, cradle-to-grave, or even cradle-to-cradle (RICS 2014). For instance, cradle-gate system boundary includes all the emissions associated with energy and processes from raw material extraction up to the final manufacturing of the product within the factory. Similarly, cradle—site

Table 2. Embodied carbon counting guide in different stages of project.

Stage	Methodology	Data source
Product	$EC_{\text{product}} = \sum \text{Quantity of material constituents in each item/element} \times \text{EC factor of the respective material}$	Inventory of Carbon and Energy (UK) (Hammond and Jones 2011), SimaPro, GaBi
Construction process	$EC_{\text{construction}} = \sum \text{Quantity of energy used for the activity} \times \text{EC factor for respective energy source}$	DEFRA Greenhouse Gas Conversion Factor Repository, GHG Protocol calculation tools
Use stage	$EC_{\text{use}} = \sum \text{Quantity of materials to be replaced} \times \text{No. of replacements} \times \text{EC factor of the respective material}$	BCIS Life Expectancy of Building Components (BCIS 2006) + product stage sources
End-of life	$EC_{\text{end-of life}} = \sum \text{Quantity of energy used for the activity} \times \text{EC factor for respective energy source}$	Construction stage sources

covers all the emission included in cradle-to-gate plus emission associated with transporting to site. Therefore, an estimate with a cradle-to-grave boundary will have higher figures than an estimate with a cradle-to-gate boundary. Therefore, system boundary is one factor to be considered when comparing studies.

5.2 Method of estimating

As discussed under the development of carbon estimating following two main methods could be employed in carbon estimating,

- Manual estimating: this can be either a bottom-up approach of estimating using ICE data source and other relevant sources or itemised estimating approach using Blackbook data. Even though Blackbook is based on ICE data there could be other variations from energy calculation for plant and machinery and additional data outside ICE.
- Automated: automated systems will have a unique in-built program for extracting quantities and retrieving carbon data. Also most softwares use ecoinvent database which is updated time to time. These can lead to varying result from manual measurements.

5.3 Assumptions

Similar to any other estimating carbon estimating also involves assumptions when data is not present. If an item description is imprecise then assumptions have to be made in order to estimate EC of the item. For instance, a staircase measure in 'Nr' has to be decomposed in to concrete, formwork, reinforcement, balustrades and finishes to get the carbon estimate of that element. In this case, assumptions play a major role in the carbon estimate.

5.4 Data sources

As explained under method of measurement, data sources other than ICE and Blackbook might vary from study to study due to difference in manufactures, suppliers, age of data source and the like. This will result in different EC figures.

5.5 Element classification

Element classification is a common variation among studies. Different studies (Halcrow Yolles 2010b, WRAP, Halcrow Yolles 2010a, Clark 2013, Sturgis Associates 2010) adopt different element classifications such as NRM, SMM/BCIS—older version, British Council of Offices 2011 and some

studies did not follow any standard at all. This makes the comparison of findings difficult.

Therefore, it is important to clarify all of the above mentioned factors in EC estimating for the knowledge to be transferable.

6 CASE STUDIES: ESTIMATING EMBODIED CARBON OF OFFICE BUILDING

6.1 Method

BoQ or detailed cost plans of the office buildings in the UK are obtained from consultancy practices and EC estimating was carried out in a similar manner of a detailed stage cost estimate. The UK Building BlackBook and manufacturer's data were used as the EC data source in the estimating process. This implies the system boundary of the study—cradle-to-gate. However, estimating was not an easy process. Sometimes, data sources do not contain some of the required data. In that case, a) similar data was used or pro-rata was done where exact match was not found; b) manufacturers' data was obtained when no similar data was found; c) item was excluded when manufacturers' data could not be obtained. After estimating the EC of each item, items were classified according to NRM element classification.

6.2 Findings

Based on the method explained above EC was estimated for the following case study offices which are presented in Figure 1.

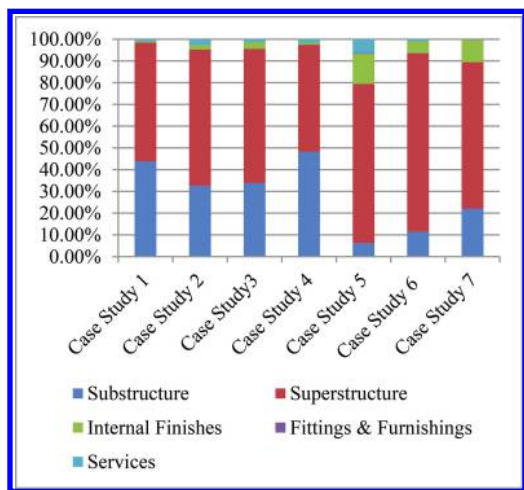


Figure 1. Elemental embodied carbon profile of case study buildings.

Accordingly, substructure and superstructure together are to contribute more than 90% of total emissions for most of the cases while finishes contribute a significant proportion to total emission in case studies 5, 6 and 7. Services element is very low than it should actually be because case studies had limitations in EC quantification of major services like electrical installations, gas installations, communication installations, fire and lighting protection installation and various other specialist installations due to lack of precise item measurements as well as lack of EC data. However, when EC of building services items are closely analysed it appears to be very small resulting in less contribution. Also as RICS (2014) claims that reduction potential is very low for services. Hence, services could be excluded from estimate which is reflected in WRAP Embodied Carbon Database. However, authors do not draw any conclusions regarding building services EC as the findings are based on few studies and this is a potential area for further research.

7 PROBLEMS IN EMBODEID CARBON ESTIMATING

There are various issues pertaining to carbon estimating which are listed and discussed below:

- Lack of standard measurement: Dixit, Fernández-Solis et al. (2012) reported that existing ISO standards for Life Cycle Analysis (LCA) lacks precision and suffers due to truncation error and subjectivity. However, in case of carbon estimating the methodology is explained through RICS (2014) guideline. Any variation in EC measurement will be as a result of variation in factors discussed earlier (system boundary, methodology, assumptions, data sources, element classification). In fact, these factors are subjective and difficult to standardize except for element classification. For instance, system boundary cannot be standardized as data available are mostly cradle-to-gate and people who are interested in cradle-to-grave will employ other ways of capturing data and it should not be restricted.
- Lack of robust database/benchmark: this is a major issue faced by the industry at the moment. There are no any other publicly available databases except for WRAP Embodied Carbon Database to support various levels of carbon estimates. Lack of EC repository is a huge barrier for EC researches and development of EC estimating.
- Lack of EC data for new products: one of the major issues in EC estimating is lack of EC data for modern products. The EC sources discussed above include most of the conventional data as

well as some modern product data. However, latest version of Blackbook is published in 2011 and it does not include products newer than 2011. Further, there is often less or no response from manufacturers regarding EC data queries.

- Lack of services EC data: case studies also witness this issue as Blackbook holds services data of disposal, sanitary, water and lift installations. Electrical installation is available for two-storey housing which is not suitable for office buildings.

8 OVERCOMING BOTTLENECKS IN EMBODIED CARBON ESTIMATING

Carbon estimating can be taken to next level only if it is treated in the same way as cost estimates. Carbon estimates should be produced in parallel to cost estimates during each successive design stage and estimates should be revisited and checked. This way carbon control can be exercised throughout the project in parallel to cost control. Further, HM Government (2010) highlight the growing concern on embodied carbon, by recommending the government and the industry to agree on a standard to measure embodied carbon to be used as a design tool. Inclusion of carbon appraisal in feasibility studies of a project magnifies the need of emission reduction to achieve a low carbon economy.

Environment Product Declaration (EPD) is a standardized way of reporting environmental impact of products. EN 15804 provides rules for the Europe-wide generation of EPD for construction products (Building Research Establishment 2015). However, not many organizations follow the standard. EPD should be made mandatory for all construction products so that the EC data is transparent and free from ambiguity. This will also stimulate EC estimating in a more positive manner.

Lack of services data can be overcome by developing dataset of elemental EC for various services. This will make the estimating process easier and help produce a holistic estimate.

Development of parametric carbon models to estimate EC during early stages of design will provide new avenue for quicker EC estimating with less design information.

9 DUAL CURRENCY APPROACH

As construction clients are becoming increasingly conscious about carbon emissions, it is important to included carbon appraisal in project appraisals. Especially, cost and carbon are now regarded as dual currency of construction projects. Clients expect not only to reduce emissions but also to achieve

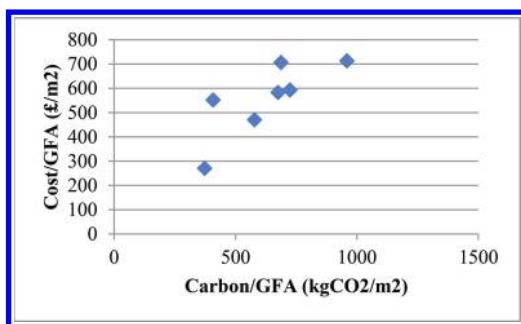


Figure 2. Cost-carbon relationship in case study buildings.

optimization in terms of both cost and carbon. This can be achieved if there is a positive relationship between cost and carbon. However, there is a gap in this regards and it is yet to be modelled.

From the case studies presented above cost is also captured in addition to EC which is depicted in the Figure 2. Figure illustrates a linear relationship between cost and carbon. However, it is based on few case studies, thus, it cannot be generalized. Nevertheless, with more case studies a conclusion can be derived.

This type of knowledge will definitely take EC estimating to next level in its development and for this to happen availability of rich EC data source is crucial.

Further, QS is the ideal person to deal with dual currency appraisal as he/she is already equipped with the right skills to do an EC estimate as suggested by RICS (2014). Therefore, it can be expected that EC estimating will become a core service in QSs' domain if EC estimating is effectively approached by the industry.

10 CONCLUSIONS

EC estimating, though considered to be a developing field it can actually be approached as similar to cost estimating. Use of EC data source dictates the method of estimating—ICE data source requires a bottom up approach, decomposing the items into material, labour, plant and building it up; UK Building Blackbook allows estimator to follow BoQ pricing approach. Both methods lead to cradle-to-gate EC estimating. However, estimates can also be prepared for other system boundary which is a tedious task as data sources are not readily available for other system boundaries. In addition to the system boundary few other factors need to be considered when estimating EC, including: method of measurement, assumptions, data sources and element classification. These factors are responsible

for reported variations in EC estimating of published studies.

Case studies of office buildings suggest that substructure and superstructure to contribute to more than 90% of emissions while services' contribution is low mainly due to non-inclusion of all the services. Accordingly, EC estimating of case studies suffers from few issues such as: lack of EC data for services, lack of precise details of the items, lack of manufacturers' data on modern products, lack of robust databases/benchmarks for EC data. In addition to that lack of standard measurement is also identified as a problem for EC estimating though authors believe it is not a major issue due to the fact that it is subjective and cannot be standardized. Identified problems could be overcome by: developing robust database (like WRAP Embodied Carbon Database) or benchmark to which industry actively contributes; EPD can be made mandatory to allow all products to declare their environmental impact which is easily accessible by estimators; and developing parametric carbon models to encourage early stage carbon estimating.

Above all, carbon and cost are considered as 'dual currency' of construction projects and there is a strong possibility of treating EC estimating as similar to cost estimating. EC management and control can be achieved in parallel to cost management and control by the involvement of QS. In this manner, dual currency approach can be implemented within the construction industry effectively.

ACKNOWLEDGEMENT

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Sustainable urban design

Improvement of urban housing blocks in China: The comparative cases of design optimisation

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ABSTRACT: In China, urban housing is the most significant part of housing sector. In recent years, the large-scale housing developments and gated housing community blocks have become key urban landscapes and housing provisions in major medium scale and large scale cities in China. The design and masterplanning of such housing developments are not necessarily sustainable. Therefore, this study evaluates key issues and potentials of urban housing blocks (or small residential districts) in China and proposes for their improvement through optimised design and planning.

This study is based on design improvement of urban housing blocks in the City of Ningbo, China. This study is undertaken as part of a research-informed teaching exercise, assessing several urban housing blocks and improving their design through evaluation of four key aspects of: a) environmental performance; b) green infrastructure; c) urban form and layout; and d) building orientation. In this study, the author will elaborate, in particular, on the importance of urban pattern and layout. The study will conclude on methods and implementation strategies for urban housing design improvement and optimisation in the context of China.

This research study is part of an on-going EPSRC-DfID funded research programme, under the 'Energy and International Development' scheme. The project is titled 'Energy and Low-Income Housing in Tropical Housing' and has a huge scope of housing and community analysis in both rural and urban areas of China, Thailand, Tanzania and Uganda. The focus of this research paper is on China's urban housing.

1 INTRODUCTION

1.1 *Urban housing blocks in China*

Urban housing is a major field in the sector of housing in a rapidly-urbanising context like China. Unlike many European countries and some other Asian countries, China's urban housing are currently built in the form of mid to high rise [tower] blocks. In the past few years, most of these blocks are developed with low density, higher buildings and less surface coverage of the urban blocks.

Although the most common practice of housing development appears in large scale blocks, there still remain many smaller housing developments that are as small as 100 m × 200 m in size. A typical two hectares urban block is still a fairly large scale in comparison to a more compact typology of urban housing in other contexts.

In the past two decades, many Chinese cities have undergone the whole process of becoming reshaped, redeveloped and restructured. Majority of this reshaping process has been the renewal of old urban fabric, particularly the urban housing. The transition of two to four storey housing towards six to eight storey housing has already taken place and is towards new mid-to-high rise

developments, most of which are gated and at a large scale (Galvez & Cheshmehzangi 2015).

In here, this study aims to identify lacking elements in China's urban housing developments and later examine ways of achieving design optimisation. This study is undertaken as a research study with a group of 20 undergraduate students with requirements of analysis, simulation and design development of selected urban housing blocks in the City of Ningbo, China.

1.2 *Context of study: Issues and challenges*

In our previous studies (Cheshmehzangi & Butters 2015, Galvez & Cheshmehzangi 2015), we have studied different typologies of urban housing in China. Based on thorough comparative analysis of 'Surface Coverage (SC)' and 'Floor Area Ratio (FAR)', we have identified a medium SC and medium to high FAR for most Chinese urban housing blocks. A typical high-rise housing block has a significantly low SC which is very much similar to the European cases, while its FAR is beyond 2.5 and sometimes reaches 4.0, which is not necessarily efficient both environmentally and socially. Consequently, low dense and [previously known as] affordable housing typologies are

subject to demolition and replacement by new mid to high rise housing developments. Apart from the three urban issues of 'limited-time land ownership', 'rapid growth' and 'large gated communities' (Cheshmehzangi & Butters 2015), China's urban housing faces the following key design challenges:

- Housing development at masterplan level, which reduces the role of architects at detailed design stage and as a result diminishes the quality of housing design;
- Lack of detailed analysis for environmental performance, greenery and landscaping, which often leads towards generic (but yet beautifully designed) green spaces and lack of consideration for urban ecology and the city landscape at a larger scale;
- Lack of integration and connectivity between the urban blocks, which results in development of large scale gated communities and encourages car use and diversity of building heights, orientation and block patterns.

By considering the three above design challenges, several housing blocks were studied across the City of Ningbo for the purpose of identifying what can possibly lead towards strategic plans for design and planning optimisation of Chinese urban housing blocks.

The overall approach to this comparative study includes: 1) a thorough analysis of an area of around 1.5 km² in the City of Ningbo (Fig. 1) with a minimum of 70% built environment (urban blocks mixed of housing and other uses), with an adequate mixture of land-uses, such as, residential, commercial, public realms and green spaces; 2) Detailed measurements of the urban features, roads, blocks and buildings as well as ratio of the built and green environments, clear understanding of land-uses, building heights, density, circulation



Figure 1. An example of studied area in the City of Ningbo with a total area of approximately 1.5 km² (Source: Baidu maps).



Figure 2. A 3D model and sectional study of the area for simulation studies (Source: IDB reports, Spring 2015).

and any environmental features/elements for the selected urban areas (Fig. 2); 3) Detailed analysis of housing urban blocks based on their current conditions; and 4) Proposal for design modifications and optimisation to initiate comprehensive design strategies based on the above.

2 CASE STUDIES OF URBAN HOUSING IN THE CITY OF NINGBO, CHINA

2.1 *The key aspects of assessment*

The assessment of selected housing blocks and improving their design is developed through evaluation of four key aspects of: a) environmental performance; b) green infrastructure; c) urban form and layout; and d) building orientation. At a later design improvement stage, the four aspects are integrated in an effective way. The importance of urban pattern and layout is embedded in all four aspects. Each aspect reflects on deficiencies of urban housing and planning that impacts how the city performs at a larger scale.

2.2 *Assessment of environmental performance*

At first, each of the selected housing blocks were assessed environmentally based on how they perform with their current design and layout (Fig. 3). This includes evaluation of insolation/lighting, overshadowing, and wind direction. By doing so, we assess basic—but yet important—condition of how housing blocks are designed with their impacts on one another, and how they environmentally perform according to issues of lighting, overshadowing and wind.

Although Eco-Tect and CFD simulations often do not provide detailed data at a larger scale, they can be used for an overall understanding of how the urban housing blocks are performing and how they may improve if to be redesigned or if a similar

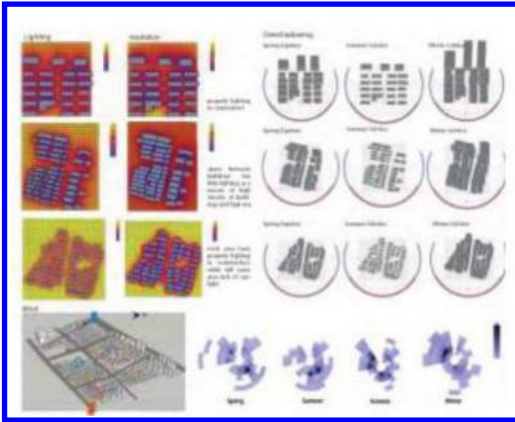


Figure 3. Environmental analysis of three selected urban housing blocks in one selected area (Source: IDB reports, Spring 2015).

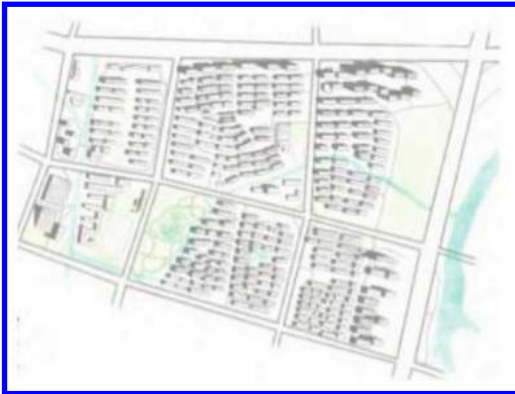


Figure 4. Assessment of available green and blue spaces in a selected urban area (Source: IDB reports, Spring 2015).

masterplan is happening at another site with a similar situation.

2.3 Assessment of green infrastructure of the urban blocks

As part of this assessment, two elements of green (and blue) spaces (Fig. 4) and mobility pattern (Fig. 5) of each site were studied at both macro and meso scales.

The availability of green and blue spaces alone does not represent much of data. However, by assessing them we can identify cooling performances at both city scale (macro level) and an urban block or building scale (meso or micro level).

At macro scale, we can identify possibilities and potentials for urban cooling corridors, urban canyons, urban cool islands and green corridors



Figure 5. Assessment of mobility pattern in a selected urban area (Source: IDB reports, Spring 2015).

(mainly ecological). Also at meso and micro scale we can assess micro-climate performance of urban blocks or units and suggest for design optimisation at a more implementable scenario.

In addition, assessment of mobility pattern enables us to evaluate network of connections and modes of movement in and about urban blocks. Such assessment identifies opportunities for social dimension of green spaces and integration of the built and green for a better performance of movement, walkability and social activities.

2.4 Assessment of urban form and layout

Lack of integration between urban blocks remains a major challenge of urban housing in China. While each block can develop differently and become an urban enclave, multiplying this to the city level would result in a mixed of city landscapes, images, and urban patterns (Fig. 6).

A variable urban pattern does not necessarily harm the city infrastructure or image as it develops diversity, but it would change the city when it leads to development of urban enclaves and large-scale (masterplan level) housing developments.

By doing such assessment, we can identify few very important elements of: urban grain, density, land-use and typology of urban blocks (Fig. 7).

2.5 Assessment of building orientation

While majority of houses are built with North-to-South facades, some housing blocks are built with larger footprints resulting in having many individual residential units with one-sided ventilation, west or east orientations only and some units with inner facades and less access to direct light.

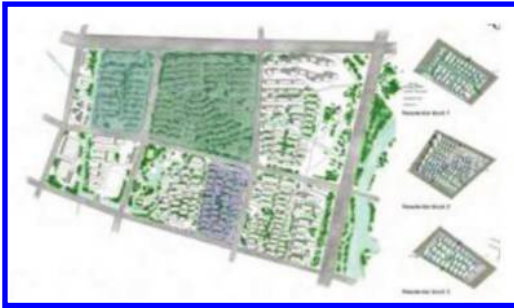
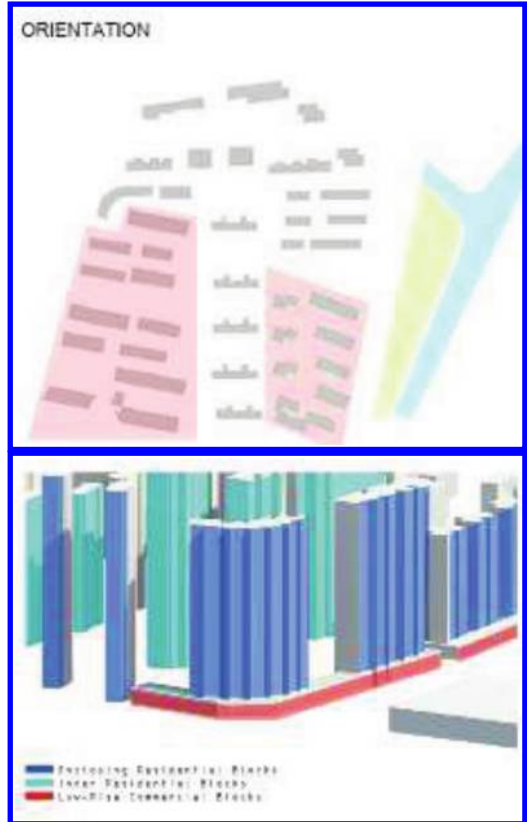


Figure 6. Assessment of urban form and layout in a selected urban area (Source: IDB reports, Spring 2015).



Figure 7. Assessment of grain study and density in a selected urban area (Source: IDB reports, Spring 2015).

Building orientation has significant impact on maximising or minimising lighting surface and solar gain, wind flow and ventilation for indoor spaces and cross ventilation for double-sided units. These represent key aspects of building performance for carbon emission reduction and energy efficient strategies for operational energy (Fig. 8 & 9). Design, being precursor to energy solutions here, can have a major impact on how residential units perform at micro scale and also how urban blocks perform at meso scale.



Figures 8 & 9. Assessment of building orientations in a selected urban area (Source: IDB reports, Spring 2015).

3 COMPARATIVE ANALYSIS OF SELECTED CASES

3.1 Improvement of urban housing blocks

Further to assessment studies in the previous section, the collated information is used for development of potential [re]design strategies of the studied urban housing blocks. This includes a comprehensive approach to combine the four main studied elements to optimise the housing blocks if they were to be redesigned. The process is undertaken by evaluation of existing conditions of the housing blocks and proposals to improve the environmental performance, green infrastructure, layout and orientation of buildings (Fig. 10).

As part of this design teaching exercise, same FAR is achieved for each of the improved/redesigned cases with revision of SC and density. Based on LSE Cities/EIFER studies (2014), a FAR of 1.5 to 2.5 is as efficient as any higher FAR; therefore, to achieve this, each revised case maintains a reasonable FAR and density, while an



Figure 10. Existing situation of a large scale housing block in Ningbo (Source: IDB reports, Spring 2015).

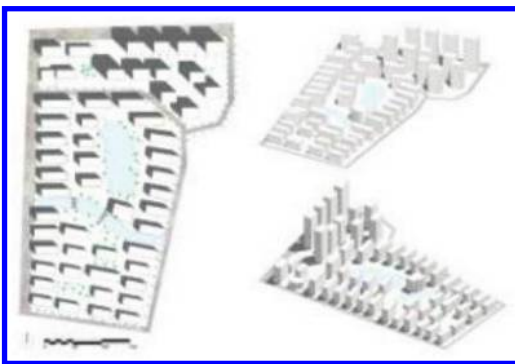


Figure 11. Proposal of new design layout based on assessment of environmental performance, green and blue spaces, pattern and layout (including density, height and configuration), and orientation (Source: IDB reports, Spring 2015).

emphasis is given to orientation, layout and spatial configuration of the blocks.

Based on simulation of lighting, overshadowing and the wind environment, the overall urban layout and spatial configuration of each block is modeled and evaluated. An example is shown below in Figure 11.

Also based on the layout design optimisation, an emphasis is given to orientation, heights and space-building ratio of each studied block. Considering the re-arrangement of spaces, the green and blue spaces are integrated as part of improving the cooling performance. As such, this improvement finds a better balance of spatial usage and green and grey ratio and integrates the two as part of re-planning the block layout.

By considering the climatic conditions (cold winter and hot and humid summer) of Ningbo, the new block layout is aimed to block cold wind in winter (from North-West) and allow a better

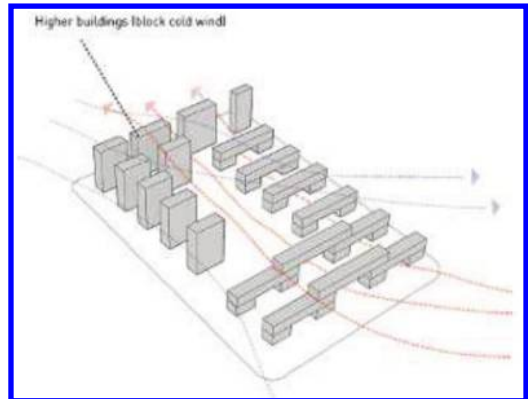


Figure 12. An example of orientation and layout of a re-modeled case (Source: IDB reports, Spring 2015).



Figure 13. An example of proposal in section and plan (Source: IDB reports, Spring 2015).

wind flow in summer (from South-East) for a better cooling performance (Figs. 12 & 13).

3.2 Discussions: Towards design optimisation

Here we have two design optimisation challenges; 1) It is almost impossible to find optimised solutions at an urban scale while implementation takes place at a smaller scale; and 2) detailed design optimisation lacks careful architectural input and analysis with a longer timeframe, which requires a more comprehensive approach than what currently exists in practice. In this study, the emphasis has been on potential improvement scenarios for existing blocks while the real scenario remains in hands of mass housing and large scale development projects. A pro-developer planning approach to urban housing has a negative impact on quality and efficiency of housing developments. Such negative impacts include: fast pace of development, low-quality design consideration, generic master-planning and lack of detailed design.

As part of this design teaching exercise, one can argue the effectiveness of design strategies for the existing contexts while planning the new is often easier and with a higher potential of better

planning and design. However, the city is comprised of districts and communities where housing provision appears in various typologies, heights and characteristics. The concern about current homogenization of urban housing in China is an alerting matter that suggests reduction of architects' role in the sector. As a result, the approach undertaken in this study and as part of the teaching exercise was to re-promote the value of architecture and detailed design for urban housing blocks in China.

4 CONCLUSIONS

This paper proposes improvement of urban housing blocks through integrated consideration of four key aspects of: a) environmental performance; b) green infrastructure; c) urban form and layout; and d) building orientation. These aspects may appear general and simple at a drawing board, but are certainly very crucial and effective at the implementation stage. The lack of housing development control and deficiency of planning regulations has resulted in dispersed and endless large-scale housing developments, cities with many enclave residential communities (i.e. gated communities) and large numbers of housing provision that are not socially viable (e.g. due to high density, poor urban layout and lack of connection to the context) and are not environmentally efficient (e.g. large grain of building blocks with provision of several units at each floor, higher dependency on air conditioning for heating and cooling, and lack of consideration of sound material use and facades).

This study is based on a research-informed teaching exercise with a focus on methods of urban housing design improvement and optimisation in the context of China. A step-by-step approach to such optimisation methods includes assessment of scale, context and climatic condition of each studied case. By integrating the four key aspects of the study, a holistic framework is developed to enhance

efficiency of urban housing and their design at a pre-design stage. The practical implications of such methods, however, require careful reconsideration of planning regulations on urban housing, review of urban block sizes and their integration at a larger scale (i.e. at meso or even macro scales). With such planning and design thinking approach, there remains an opportunity to improve urban housing in China and promote new housing typologies that are vernacular and context-specific, performative and sustainable. Some of these can currently occur at an urban block scale for faster implementation and noticeable success stories. Nevertheless, a successful practical implementation requires comprehensiveness at both planning and pre-design stages while policy changes require reflecting on the urban housing challenges that are addressed in this study.

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Planning and design strategies for sustainable urban development

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ABSTRACT: In order to create high impact low-carbon architecture it is necessary to address issues at the urban scale. This paper reviews methods adopted in the UK, Hong Kong and America and develops a series of themes that should be incorporated in modern city development strategies to produce green and sustainable outcomes. These are concerned with: planning policies and building regulations; neighborhood and site planning; and building design. Underlying themes which must be incorporated are connected to: building façade, systems design and operation; energy demand and supply; transportation systems; water supply and use; and wastes/pollution issues. To achieve success a number of support mechanisms are needed including strategic and infrastructure development, implementation of assessment frameworks, and the distribution of information and knowledge to stakeholders. The themes are compared to recent developments in the city of Kunming and proposals for future applications are explored.

1 INTRODUCTION

Cities contribute an estimated 70% of the world's energy-related Greenhouse Gases (GHG). With the rapid urbanization in China, huge stresses have been added onto all forms of public services provision, such as systems for energy, water use, transport and waste. In the 12th Five-Year Plan for 2011–2015, many Chinese cities are already committed to a high carbon-emission growth path. The World Bank estimated that about half of the world's new building constructions have taken place in China in the years leading up to 2015 (Baumler et al. 2012). In response to the emerging focus on environmentally sustainable growth, many cities are already developing eco-city and low-carbon city initiatives. At the beginning of 1990s, gradual development of the real estate market efficiently and effectively boosted China's economy, and also led to rapid development of the housing industry. Under the influence of the commodity economy, Chinese housing became a commercial product that encouraged cutting-edge technology and design theory with an emphasis on market competition. A number of housing projects which were designed with "green" concepts have appeared. Constructing green house or low-carbon dwellings has become a popular trend in much housing design practice, and estate developments with green styled names have been developed.

In order to create impactful low-carbon architecture it is necessary to address issues at urban scale. This paper reviews methods adopted in the UK (BREEAM, BREEAM for community), Hong Kong (HK-BEAM), and America

(LEED, LEED-ND), and develops a series of themes that should be incorporated in modern city development strategies to produce more green and sustainable outcomes. Based on the analysis of different sustainable design assessment methods, a case study of recent a development in Kunming city has been used to explore the issues and make a number of suggestions for future development.

2 REVIEWS: BREEAM, HK-BEAM, LEED, ASGB

Green building assessment systems have been developed in various countries with different emphases. Based on the different development situations, the contents of each evaluation systems are vary.

The first prominent green building assessment system was published in 1990 by BRE (Building Research Establishment) in the UK. BREEAM (Building Research Establishment Environment Assessment Method) became the leading and most widely used environmental assessment method for buildings and communities. Inspired by BREEAM, different versions of green building assessment methods have been launched by research organizations in different countries. HK-BEAM was introduced to Hong Kong in 1996 and is an environmental performance-based assessment scheme purpose designed for high-rise buildings. The certification scheme is completely voluntary and is owned by the HK-BEAM Society, a non-profit making organization consisting of industry professionals (Chu 2004). The United States Green Building Council's (USGBC's) (established in 1993)

Leadership in Energy and Environmental Design (LEED) is a program that provides third-party certification for green buildings. The first version of LEED was published in 1998 (Floyd & Bilka 2012). There are many other assessment systems currently available, such as GB-Tool (Canada), Australian Building Greenhouse Rating Scheme and NABERS, Green Star, SAP, Home Energy Rating System (HERS) programs, NatHERS and FirstRate, Eco-Quantum, Green Guide to Specification, ENVEST and Ecopoints, BEES (Pitts 2004). The details of different of BREEAM, HK-BEAM, LEED and ASGB (“*Assessment Standards of Green Building*”) are listed in the Table 1.

The following sections of the paper focus on the individual methodologies; the purpose of this analysis in the overall project was to enable identification of key and recurring factors and features that could be applied in the setting of the City of Kunming. This then helps set a background against which future development trends might be evaluated. Further information on design and detail is still required, though sometimes this is not easy to obtain.

2.1 BREEAM

Different versions of each assessment systems and methods have been updated for a number of times, and also included more assessment items. BREEAM consists of a series of rating systems

for a range of building types. Buildings are rated and certified on a scale of ‘Pass’, ‘Good’, ‘Very Good’, ‘Excellent’. In 2008, the BRE added a fifth category of ‘Outstanding’. There are several purposes of the rating system: firstly, it is to provide guidance to reduce the effect of buildings on the global and local environment whilst also creating comfortable and healthy indoor environments; secondly, it is to enable developers of buildings who have addressed environmental issues to gain credit for this through the rating of proposals and the award of a certificate (Pitts 2004).

2.2 LEED

The LEED rating systems address a wide variety of buildings types, including separate rating systems for new construction and existing buildings. Those include operations and maintenance, commercial buildings, core and shell, schools, retail (new and commercial interiors), healthcare, homes, neighborhoods developments. Buildings are rated at levels of ‘Certified’, ‘Silver’, ‘Gold’, and ‘Platinum’. A points based system is then used to encourage the implementation of other green and sustainable best practices, and has led to healthier, productive buildings, reduced stress on the environment by encouraging energy and resource-efficient buildings, and savings from increased building value, higher lease rates and decreased utility costs (Floyd & Bilka 2012).

Table 1. General comparison of green building assessment methods.

Assessment system	Object	Categories
BREEAM 1990 UK	New construction, existing buildings include: communities, courts, data-centres, education, healthcare, industrial, entertainment and leisure, office, other buildings and mixed use developments.	Management, energy, transport, water, materials, waste, land use and ecology, pollution, health and well being.
LEED 1998 US	New construction, existing buildings including: operations and maintenance, commercial interiors, core and shell, schools, retail (including new and commercial interiors), healthcare, homes, and neighborhood development.	Sustainable sites, water efficiency, energy and atmosphere, materials and resources, indoor environmental quality, innovation design, regional priority.
HK-BEAM 1996 Hong Kong	“New” and “Existing” building types including: offices, residential, mall, hotel, school, hospital, institutional and mixed complexes centrally air-conditioned, naturally ventilated or mixed mode.	Site, materials, resource, water resource, indoor environmental quality, innovation design.
ESGB Assessment Standard for Green building 2006 China (China, 2015)	New construction, extended buildings (residential building, public building).	Land saving and outdoor environment, energy saving and energy utilization, water saving and water resources utilization, material saving and material resources utilization, indoor environmental quality, construction management, operation management.

2.3 HK-BEAM

The Building Environmental Assessment Method (HK-BEAM) scheme used in Hong Kong was established in 1996, largely based on the UK Building Research Establishment's BREEAM. There was a significant upgrade to the previous BEAM documents in 2004. Next in response to raised concerns of occupant health in buildings especially after the outbreak of SARS in 2003, the latest HK-BEAM standards have evolved to address hygiene, health and other environmental issues in a more holistic manner (Chu 2004). In 2009, in consideration of the critical state of global environmental issues, BEAM was further enhanced to meet higher expectations of the public and community. A rating is issued to a project according to the score achieved after Provisional Assessment (PA) or Final Assessment (FA). Potential outcomes are: 'Platinum', 'Gold', 'Silver', 'Bronze', and 'Unclassified'. As with other schemes, a number of benefits can be achieved by adopting HK-BEAM, such as cost-savings through the more efficient use of energy and resources; increasing occupant satisfaction from healthy and productive accommodation, enhancing corporate profile and marketability to potential building users; providing a tool to improve purchaser choice and information; integrating local and international best practice into new designs; providing increased protection against environmental liability; and establishing a clear direction for continuous improvement and optimized performance (Society 2004).

2.4 ASGB (<绿色建筑评价标准>)

In China green building has been defined in the "*Assessment standard for green building*" (GB50378) as: "constructions that fully considered the requirements of environment protection issues during the process of design, constructing, and the utilization of building materials". It also requires assessment combining architecture with farming and agriculture, energy, environmental protection, aesthetics, and high-technology construction. It also asks for the design of buildings that not only satisfying functional demands, but which also provide healthy and environmental friendly spaces for living and working. (在全寿命期内, 最大限度地节约资源(节能、节地、节水、节材)、保护环境、减少污染, 为人们提供健康、适用和高效的使用空间, 与自然和谐共生的建筑)" (China 2015). Considering the basic needs of Chinese development, it contains sections related to the following: land preservation and outdoor environment; energy saving and energy utilization; water saving and water resources utilization; materials saving and material resources utilization; indoor environmental quality; construction management;

and operation management. Under each theme, there are 'Prerequisite items'; 'General items'; and 'Optimized items'. Prerequisite items are those which must be achieved as a necessity for green building. General items are those which are difficult to achieve and generally require high levels of attainment for each target. Optimized items denote those which are more difficult to implement in the project and with higher requirements for each target. The latest version of the "*Evaluation standard for green building*" (published in 2014) has changed 'General items' into 'Scoring Items' and removed 'Optimized items', which means higher standards have to be achieved as part of the process in order to get higher scores.

2.5 BREEAM for communities, LEED-ND

Both BREEAM and LEED have developed assessment methods for community and neighborhood scales, explained as follows.

BREEAM for Communities has been developed to enable planners and development teams to address sustainability from an early stage. It is an independent, third party assessment and certification standard based on the established BREEAM methodology. It is a framework for considering the issues and opportunities that affect sustainability at the earliest stage of the design process for a development. The scheme addresses key environmental, social and economic sustainability objectives that have an impact on large-scale development projects. BREEAM for communities awards credits in six categories:

- Governance (GO): "Promotes community involvement in decisions affecting the design, construction, operation and long-term stewardship of the development." (BRE 2012)
- Social and Economic wellbeing (SE): "Considers societal and economic factors affecting health and wellbeing such as inclusive design, cohesion, adequate housing and access to employment." (Ibid.)
- Resources and Energy (RE): "Addresses the sustainable use of natural resources and the reduction of carbon emissions." (Ibid.)
- Land use and Ecology (LE): "Encourages sustainable land use and ecological enhancement." (Ibid.)
- Transport and Movement (TM): "Addresses the design and provision of transport and movement infrastructure to encourage the use of sustainable modes of transport." (Ibid.)
- Innovation (Inn): "Recognizes and promotes the adoption of innovative solutions within the overall rating where these are likely to result in environmental, social and/or economic benefit"

in a way which is not recognized elsewhere in the scheme.” (Ibid.)

To avoid negative impacts and design problems during community development, there are three design steps that are promoted for the assessment of sustainability at the master-planning level. Master-planning should be an iterative process that involves: developing plans, consulting stakeholders and then revising plans. BREEAM for communities links assessment with the master-planning process to ensure that issues are addressed at an appropriate point in the early design stages.

LEED-ND (LEED for Neighborhood Development) was released in 2010 in the US. It aims to integrate the principles of smart growth, urbanism and green building into a national system for neighborhood design. The LEED-ND system is collaboration between the United States Green Building Council, the Congress for New Urbanism and the Natural Resources Defense Council. LEED-ND appears to be the most comprehensive of all the LEED green rating programs. It has been design to put emphasis on site selection, design, and construction elements that bring buildings and infrastructure together into a neighborhood. This approach also promotes the relationship between the neighborhood, its landscape, and its local and regional context. It has different focuses from those in BREEAM for Communities that involves more evaluation at the master-planning level. LEED-ND has three environmental categories: smart location and linkage, neighborhood pattern and design, and green infrastructure and buildings. These are discussed below:

- Smart location and Linkage: “Focuses on site selection that minimized the adverse environmental effects of development across several categories, including transportation, air quality and preservation of environmentally-sensitive lands or ecosystems. Urban sprawl and associated low density, segregated housing and commercial uses are discouraged. Preference is given to locations close to existing town and city centres, sites with good transit access, infill sites, previously developed sites and sites adjacent to existing developments. Selection of sites that are within or adjacent to existing development can minimize habitat fragmentation and also help to preserve areas for recreation. Remediation and reclamation of contaminated brownfield sites make them safer and can contribute to social and economic revitalization of depressed neighborhoods.” (Floyd & Bilka 2012)
- Neighborhood Pattern and Design: “Emphasizes the creation of compact, walkable, mixed-used neighborhoods with convenient pedestrian connections to nearby communities. Compact communities provide opportunities to reduce driving

and resultant emissions, conserve economic resources, and help reduce the spread of low density development across a region’s landscape. Public spaces, such as parks and plazas, can encourage social interaction and active recreation while helping control storm-water runoff and reducing heat island effects. Community gardens promote social interaction and physical activities while increasing access to fresh, locally grown produce. Communities with diverse housing types permit residents to live closer to their workplaces and allow families to remain in a given neighborhood as their circumstances changes.” (Ibid.)

- Green Infrastructure and Buildings: “Focuses on measures that can reduce the environmental consequences of the construction and operation of buildings and infrastructure. Including certified green buildings in project is one way to reduce negative environmental effects. Sustainable building practices reduce waste and use energy, water and materials more efficiently than conventional building practices. Site ecology damage can be minimized during construction by confining construction activities to limited areas and restriction the development footprint.” (Ibid.)

In the following section, Expo Eco-town in Kunming is used as a case study to investigate sustainable development in practice in a Chinese city.

3 EXPO ECO-TOWN IN KUNMING CITY

As mankind enters the 21st Century, the focus on sustainable development now holds the world’s attention and has been widely practiced in different fields. The building sector has been identified as that with the highest energy consumption at a time when China has a rapidly and continuously growing real-estate industry. ‘Green and low-carbon housing’ has been recognized as one of the phrases which suggests the most advanced design concepts for residential products are being exploited in design practice.

Expo Eco-town is presented here as an example of residential development that takes sustainability as a key design issue along with appropriate design practice. Beginning in 2000, the Natural Resources Defense Council (NRDC) and U.S. Department of Energy worked closely with China’s MoST (Ministry of Science and Technology) to develop large scale demonstration projects for green building in China. One of these projects is in the southern city of Kunming, Yunnan by the UK-based designers INTEGER.

3.1 Master-planning

Situated in southwest of China, Kunming is a historical city with great potential for development.

It is located on the northern side of the Dian Lake, which is in the central part of Yunnan province, and it is surrounded by hills on three sides and by water on the fourth side. Kunming is famous for its gentle climate which is not very hot in summer and not too cold in winter. Annual average temperature of Kunming is around 15.1°C, and reputed as 'spring city' and 'flower metropolis'. Kunming is a tourist city that bears little relationship to the cycle of the seasons. However, there are many challenges that the city has had to face during rapid urban development. Its geomorphic types are varied: up to 85% of the is classified as a mountainous region; 13% urban flatlands; and 2% natural water systems (Kunming 2012). Limited by natural resources and environment, sustainable land use has to be addressed: for example, in future development, mountains and hills (as the dominant terrain of the city) need to be considered as development choices. Furthermore, urbanization of Kunming city is less well supported than in other Chinese cities due to a relative lack of industrial development. This is one of the reasons leading to the overload of urban functions in the central city. Moreover, urban water environment is one of the important considerations of sustainable urban development. Water-intake, water supply, water drainage, and water resources renewal, are major challenges to meet the present needs without compromising the future. It is the water environment in Kunming that determines the scale of the possible urban economic development and sustains the population (Kunming 2012).

By considering the natural environment and sustainable development, new approaches have been explored in Kunming. Expo Eco-town has been one of the examples of adopting sustainable design concepts that match with local conditions.

The Expo Eco-town is located on the east of Kunming city. Hills are the dominant terrain, which makes the area distinctive with natural landscape features different to the built-up city centre. The site covers a total area of 255.7 ha. The planned area for the project was distributed across three surrounding areas (Fig. 1).

The total construction area of Expo Eco-town is about 460,000 m², combining eight independent districts. Plot ratios of construction in phase one development were set between 0.3–0.4. The whole project of Expo Eco-town includes the eight neighborhood units, (with housing on hilly areas), sports centre, golf course, commercial centre, exhibition hall of Expo INTEGER (UK-based company), and space for Government funded projects, such as International Conference Centre and the State Guest House. The first phase development of Expo Eco-town has been fully completed; other parts of the site are still under construction.

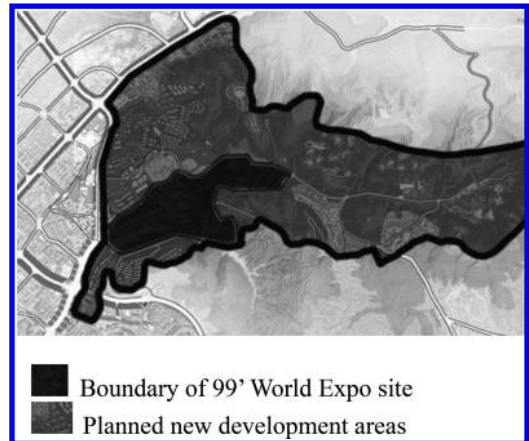


Figure 1. Planned development area of Expo Eco-town and boundary of World Expo site (Jiang 2014).

The identification of the project boundaries in Figure 1 is based on the authors' access to project materials and on site visits.

Seven research teams from different disciplines were involved in the primary studies for the project. The research fields included: geographic environment, architecture and urban planning, housing market demand, community management, evaluation of the geological environment, and analysis of policy and profit; all encompassing a wide range of research on local conditions (Eco-town 2007).

Meanwhile, on the basis of previous research, "Key point of healthy residential construction technology" (<健康住宅建设技术要点>, 2004), "Key points and technical guidance of green ecological residential district construction" (<绿色生态住宅小区建设要点与技术导则 (试行)>, 2001), and LEED (USGBC's Leadership in Energy and Environmental Design) Rating System have been used as main design references to guide the design and construction of the project. Cooperation between SWA (Design Institute of Landscape Architecture, Planning and Urban Design) and UK-based green building design agency INTEGER enabled comprehensive planning of Expo Eco-town to be developed.

3.2 Concept development

After discussions among stakeholders and researchers, several design concepts were generated and one chosen to be finalized as a design proposal for Expo Eco-town. The design proposal covered five areas of planning for the residential area which could provide more sustainable lifestyles for residents. For the whole residential area, Expo Eco-town was considered as an open area that

could establish good connections among neighborhoods and with people from other residential areas around it. For neighborhoods within the Expo Eco-town, living facilities were an important issue and were well-considered in different areas. Living resources, information resources, and energy supply have been designed to provide for a good quality of life for residents. Waste recycling, sewage treatment, and air cleaning systems have been introduced into the residential area along with Medical facilities. Earthquake resistant design has also been used to ensure a safe living environment. Further, Expo Eco-town has been designed as an integrated community which generated various urban functions to include tourism facilities, a research centre, educational facilities, a conference centre, a sports centre, eco-homes, and other municipal facilities. To enhance the natural environment in Expo Eco-town, the design proposal put emphasis on the combined consideration of the local climate and landscape in order to provide a good living environment for residents and also meet the needs of other species that inhabit there. For instance, the natural water system, the natural mountain landscape, and the forest on site, have each been well protected to avoid destroying the natural habitat for other species.

“Long-lived residential dwelling” is a slogan that delivers the message to residents that the dwellings have been designed using local natural resources for and for sustainability. To achieve this aim, building materials with less embodied energy have been promoted in the project, and ways of recycling and reusing the available resources also have been well considered based on local conditions. Furthermore, solar energy as the main renewable energy source in Kunming has been well-developed in this project.

A concept of “self-renewal” has also been set to improve and renew facilities in time to make sure of longer term use. To provide alternative ways for residents to travel, a transport system was designed to encourage residents to use public transport, walk or cycling; and to regulate the use of private vehicles. An efficient management system can ensure the whole community is running well. The design proposal for Expo Eco-town requested an optimized management system which could respond to demands from residents in good time. The management system is still committed to maintain a normal order for residents to create a community with a strong sense of belonging. Furthermore, community regulations set up a number of rules for residents (for example: residents cannot cut trees or other plants, hunt or kill animals, discharge sewage or waste, and so on). Green education has also been considered, for example, how to strengthen residents’ environmental consciousness,

how to encourage residents to use green building materials, and how to deliver a basic understandings of recycling waste.

The whole design proposal covers different study fields of constructing in a sustainable residential area. Starting from the planning theme through the detailed living facilities that were implemented at the end, Expo Eco-town was seen as a cutting-edge approach that explored ways to create a vibrant residential area with more consideration for sustainability. The design proposal generated different methodologies which could enhance the level of sustainability in residential area; however methods which could implement intangible concepts in a more tangible way were not well-considered at the planning stage. Therefore, after residents have moved into the community, a number of original design concepts have lost their meaning. This is design understanding gap which needs to be filled and requires further in-depth and empirical research on understanding the links.

3.3 *Energy-efficient design and technology*

As a pioneer project of the “Green residential area” approach in Kunming, a number of intelligent, green and energy-efficient building technologies have been explored and implemented. One of the principal features of this project is the site plan strategy. In order to ensure there will be less damages caused by construction on site, the building land-coverage proportion has been controlled within 10%. This means there will be more greenland sustained on the site and the natural water system has been preserved to some extent. Furthermore, there are five drainage channels for collecting rainwater for recycling and utilization. Moreover, water seepage paving stones have been used for out-door walking paths which can sustain the rainwater within the site and provide more water for irrigation. After being filtered and collected, rainwater can be used for plants or stored in a reservoir. [Figure 2](#) shows the location of reclaimed wastewater treatment systems of Expo Eco-town (phase 1) and is based on site visits by the author and information from the Design Institute.

Each residential dwelling, including those buildings on hillsides, is orientated to face south to gain as much as natural light and heat as possible. Traditional means of ventilation in dwellings have been used in order to reduce the extra energy consumption for mechanical cooling and ventilation systems. Water-saving faucets, showerheads and toilet tanks have been used. Furthermore, domestic sewage can be recycled through residential wastewater treatment systems which have been in-stalled in each house. Kunming is rich in solar energy resources, and in Expo Eco-town, solar panels are

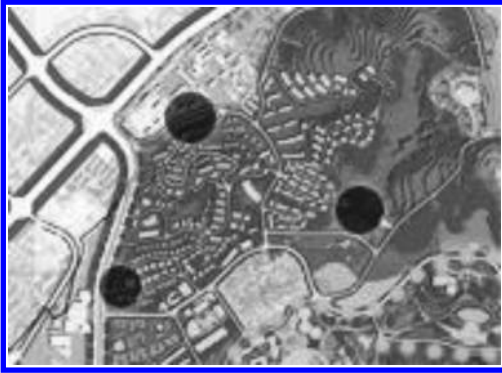


Figure 2. Distribution of reclaimed wastewater treatment system in Expo Eco-town in first phase of development, (using data from Kunming 2005).



Figure 3. Redesign the natural water system on site (Jiang 2007).



Figure 4. Permeable pavement design in Expo Eco-town (Jiang 2007).

mounted on the south-facing roofs to heat water for everyday use. All the residential dwellings have Low-E glass windows installed with the purposes of reducing energy use and CO₂ emissions.

3.4 Discussion

From master-planing to detailed housing design, Expo Eco-town as a pioneer project in Kunming



Figure 5. Solar energy ground lights and solar heat-collecting panel on the roof (Jiang 2007).

has explored and has practiced cutting-edge designs and technologies. New approaches and technologies brought together development of sustainable residential designs, yet also led to higher cost at the initial construction stage. The design concepts and green products have verified the development of sustainable design in residential dwellings, but the impacts on residents in and around Expo Eco-town in the long-term still needs further research. For instance, design concepts and management strategies identified Eco-town as a project to demonstrate sustainable design to the general public. It has also been considered as an area that aims to establish good connections amongst neighbourhoods and also with other residential areas around it. But as an expensive high-end development, it has already been isolated due to residents ability to afford to live there; by contrast, residential districts around the Expo Eco-town have accumulated residents from different social groups. The management strategy for the Expo Eco-town is to ensure the living environment to be more safe, quiet and comfortable. As a result "good connections" with other residential areas is hard to achieve. Furthermore, as a demonstration project with educational functions, easy access for other residents of the city to visit the community is important in order to disseminate knowledge and to share the well preserved natural environment. But the gated community makes it difficult for others to visit the place. Overall, Expo Eco-town has demonstrated possible ways for residential districts to be designed and used more sustainably in Kunming. But more detailed research and observation are needed to improve intangible design concepts, which means not only the designer conceives what the residential district will be like, but also the end users also know what the residential district should be like.

3.5 Challenges and successes

Arising from the post-development phase of Expo 99' Kunming, Expo Eco-town is a new attempt for residential development in Kunming. It is different from other residential projects implemented in Kunming; the project received a lot of attention

and support from government. The project has put great effort into integrating design resources and establishing connections among different disciplines. The new approaches to urban development, adopting hilly and mountainous areas in Kunming, have been implemented with the manner of sustainability. In 2000 a research team to consider post-development of Expo 99' Kunming was established and for several years examined natural resources and development strategies for land around Expo 99'. The crossover research among different disciplines provided detailed information on site, and has also done much to support decision-making during the later design processes.

But there are also challenges associated with this project. As the first demo-project for green residential development implemented in Kunming, providing the environmental-friendly living space is not the only target. More significantly, this project should have provided opportunities to spread knowledge and information that could improve consciousness of sustainability in residents' daily life. For example, Expo eco-town has well connected inner residential neighbourhood, but the whole project basically exists as a huge "gated" residential district. Good design concept and green lifestyle cannot therefore be shared easily amongst the wider group of city residents. Significantly, a high profile demo project such as this should not be developed for an isolated community.

3.6 Conclusion and suggestions

According to the reviews of assessment methods set up in the UK (BREEAM, BREEAM for community), Hong Kong (HK-BEAM), and America (LEED, LEED-ND), there are a series of themes that should be incorporated in modern city development strategies to produce green and sustainable outcomes. A numbers of common principles from all the assessment systems introduced in this study are suitable for referencing in creating impactful low-carbon architecture and also to address issues at urban scale. In addition, good practices for sustainable urban design in China can be summarized based on the analysis of the Expo Eco-town case study as follows:

1. Include design teams from different disciplines and users groups in the master-planning process to ensure that issues are addressed in early design stages.
2. The post-construction stage of the project is equally important to ensure that measures of sustainable design are implemented in the occupation stage and the good connection between neighborhoods are maintained and managed.
3. Good design concepts and practices of sustainable projects need to be disseminated to the general public in various ways and easy access to those projects needs to be provided for visitors.

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Noise mapping approach for the acoustic environment of mountainous cities: A pilot study in Chongqing, China

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ABSTRACT: Chongqing is a typical mountainous city in Southwestern China. Its undulating terrain creates a unique urban structure and brings diversified features to the compact urban open spaces. The increased density of high-rise buildings makes urban sound environment complicated. Integrated with a number of necessary environmental variables, noise mapping is a useful tool for the assessment of noise pollution at different urban scales. This paper aims to investigate the relationships between the terrain characteristics and the acoustic environment of high-rise residential buildings close to urban expressways in Chongqing, using noise mapping approach. 35 simulation models, specified with various mapping parameters, building and terrain properties, were compared and analysed to investigate the noise level distributions of different building layouts for road-side apartments, and the sound attenuations along varied road cuttings in mountainous cities. It has been shown that the rectangular-shaped apartments on roadside obtain better noise reduction capacities than tower blocks. The acoustical performance of road cuttings, as an appropriate designed earth mounds, is improved along with deeper vertical alignment and bigger slope angle towards the right-angle.

1 INTRODUCTION

It is widely accepted that the residential areas in modern cities should have good acoustical environments for local residents (van Kempen et al. 2014). Noise affects exposed people, as it disturbs communication, concentration, and activities and provokes adverse emotional reactions (Miedema 2007, Xie & Kang 2010). Traffic noise, as the primary urban noise type, could make obvious interference to the residents in roadside buildings within a dense urban morphology (Brown et al. 2015, Paunovic et al. 2014). To date, noise pollution has been one of the major urban environmental issues for more than a decade in China (Xie et al. 2014).

Noise mapping is a good tool intended for the assessment of noise pollution at urban scales. To consider both topographical and acoustical complexities, a useful noise mapping approach is adopted to produce a series of three noise maps: for the present environment, the planned construction works and the future. EU Member States were required to determine the exposure to environmental noise through strategic noise mapping and to elaborate action plans in order to reduce noise pollution. A common framework for noise assessment methods (CNOSSOS-EU) has been developed by the European Commission in co-operation with the EU Member States for strategic noise mapping

in every five years, as required by the Environment Noise Directive (2002/49/EC) (Kephelopoulou et al. 2014).

Chongqing is a typical mountainous city in Southwestern China. The natural mountainous terrain, where a large number of road projects and compact high-rise buildings in Chongqing are built on, makes the urban sound environment complicated, in conjunction with the significant street canyon effect (Kang 2000). The sound propagation in a valley-slope configuration showed that the terrain profile was responsible for an increase in sound pressure level at distant, elevating points up to 30dBA compared to a flat ground situation (Van Renterghem et al. 2007). A Spanish study obtained that different accuracy levels (Kappa index values from 0.725 to 0.987) depended on the terrain and noise source properties (Iglesias Merchan and Diaz-Balteiro 2013).

In recent years, noise mapping has been encouraged for the environmental noise evaluations at several big Chinese cities. Nonetheless, this type of assessment approach is rarely applied in mountainous cities. The information presented in 2D still limits the effective and realistic understanding of the complex urban environment in Chongqing, comprising of many high-rise buildings crowded in close proximity together with complex road and flyover layouts in the hilly terrain. 2D noise

mappings are simply not good enough in accurately depicting the noise environment. This paper aims to investigate the relationships between the terrain characteristics and the acoustic environment of residential areas close to urban expressways under complex circumstances with the hilly terrain, considering the terrain effect on noise propagation, through a series of noise mapping simulations.

2 METHODOLOGY

2.1 Case study areas

A residential area (1 km²) close to the urban expressway in Shapingba District, Chongqing was chosen as the case study site. Obvious undulating terrain exists between the urban expressway and housing estates in this study site, including the interior areas of each housing estate as well. For those two commercial-residential mixed housing estates (A and B), the nearest distances between the apartments and urban expressways are only 30 and 40 m respectively, as shown in Figure 1. The field acoustic measurements were conducted on weekdays, according to the Chinese Standard GB3096-2008, with both typical daytime (15:00–17:00) and night-time (23:00–1:00) considered, in order to avoid the peak traffic hours. The traffic flows in terms of different vehicle types for relevant roads in the study areas were recorded as well.

2.2 Noise mapping

This study firstly established 9 models to assess the influence of building layouts on the road traffic

noise under the context of undulating terrain, taking into account the spatial and time distributions of noise levels. Then another 26 small-scale models used different deepness or angles of road cuttings, to explore the relationships between the mountainous terrain characteristics and urban acoustic environment. Traffic flow parameters and building models were validated based on the field measurement results, and the average error between testing and prediction results was normalised under 3 dBA to ensure the reliability of original noise mapping model. Predictor-LimA software suite (version 10.10) was adopted for the noise mapping simulation. The calculation standard is based on the relevant Chinese Standard HJ2.4-2009. All calculations were performed for the changeless propagation conditions according to the homogeneous atmospheric conditions.

3 RESULTS

3.1 Field measurement

In the field measurement, 28 receiver points were arranged in Housing estates A and B. The averaged daytime and night-time noise levels at all the receiver points were 61.8 dBA and 56.1 dBA for Housing estate A, whereas 57.5 dBA and 52.9 dBA for Housing estate B. In summary, 33.3% of the overall receiver points exceeded the recommended values (55 dBA) at night, given 57.1% and 14.3% of receivers at Housing estate A and B over the national standard. Clearly, in average, the acoustic environment of Housing estate B was quieter than Housing estate A. The relationship between normalised sound levels at the inside receivers and the nearest distances to the urban expressway was illustrated in Figure 2. The night-time noise levels had a significant negative correlation with



Figure 1. Photographs of case study areas; (a) panoramic view, (b) road-side buildings of Housing estate A, (c) road-side buildings of Housing estate B.

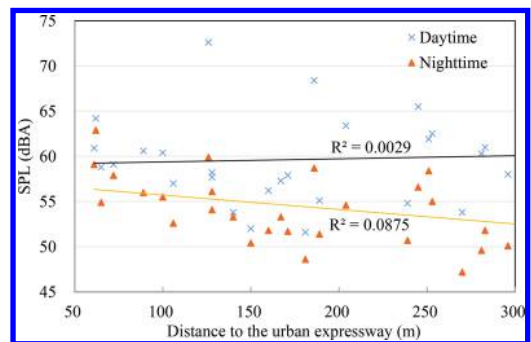


Figure 2. The relationship between normalised sound levels of the receivers ($n = 28$) and the nearest distances to the central line of urban expressway, for the daytime and night-time.

the distance between receivers and urban expressway ($p < 0.01$), whereas the day-time data showed insignificant positive correlation with the distance ($p = 0.812$), indicating the possible influences of other noise sources in this mountainous site, apart from traffic noise.

3.2 Building layout

Property developers in Chongqing normally prefer the tower blocks in the design of residential areas, due to the shortage of proper construction lands in mountainous cities. Although tower-block apartments seem to have a better visual view, its noise reduction capacity might be less than rectangular-shaped apartments (Shao 1991). As shown in Figure 3, the spatial and time distributions of noise levels were simulated in terms of three forms of

the roadside building layouts in Housing estate B, meanwhile the overall plot ratio, total building areas and total heights of the buildings were the same for all the simulation models. The receivers in those maps were placed at the height of 4 m above ground, amount to 12,252 recalculated positions, as recommended by the relevant Chinese Standard (55 dBA limit for daytime and 70 dBA limit for night-time). The averaged L_{den} of three building layouts in Housing estate B were 53.0 dBA, 58.2 dBA and 57.9 dBA, with regard to the rectangular shaped apartment, 2 tower blocks and 3 tower blocks, respectively. As expected, more scattered layout might contribute to higher average noise levels of the interior areas of the housing estate, but also better degrees of homogeneity upon the noise impacts.

After the comparisons of environmental noise data and building layout of Housing estate A and B,

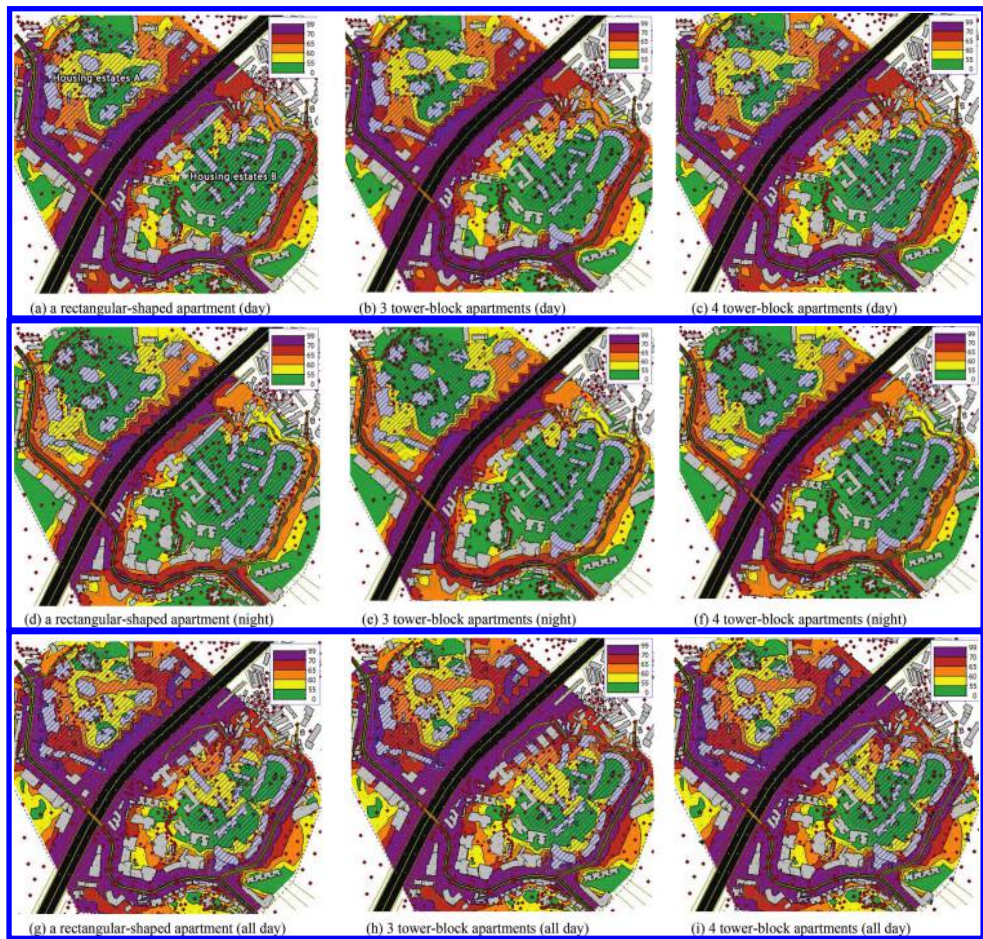


Figure 3. Noise maps for the traffic noise levels in the studied residential areas, with the considerations of 3 time periods, namely daytime, night-time, and all day (L_{den}).

situating on opposite sides of the urban expressway, it is noted that the outdoor acoustic environment was likely to be more influenced by the building layout, rather than the scale of road network grids or the distance to expressway. Three tower-blocks (40 m-length, 99 m-height) and a rectangular-shaped department (100 m-length, 50 m-height) were built in the periphery of Housing estate A and B, separately. The result shows that the interior zone in Housing estate B was quieter, though the plot ratios were quite similar in two housing estates, given 3.2 for Housing estate A and 3.0 for Housing estate B. A possible reason might be the terrace apartment in Housing estate B, as it could be regarded as a noise barrier for protecting the acoustic environment of the interior zone. This is also consistent with previous studies that the shape of building blocks had a large effect on the sound level at the least-exposed facade (quiet facade) of a building, and a smaller effect at the most-exposed facade (Salomons and Pont, 2012).

3.3 Road cutting design

Appropriate earth mound design could be effective in reducing noise. Factors affecting the performance of the earth mound include effective barrier height, scattering diffraction losses on the barrier top, absorption effects of grass-covered slopes, and the slope angle of the wedge. Similar to the above, the road cuttings above the surrounding road surfaces can create acoustic shadow zones. A series of typical configurations of road cuttings were considered, including the vertical alignment (from 0 to 20 m) and the slope angle (from 15 to 90 degrees), as shown in Figure 4. For each case, the sound pressure levels at 9 typical receiver points were calculated at 50 m, 100 m, and 150 m far from the road side, as well as 10 m, 30 m and 50 m high above the ground. The traffic noise source was set at a height of 1m above the road surface.

All the simulation results regarding road cutting design are presented in Figure 5 and Figure 6. It can be seen that in some specific cases, taking receiver

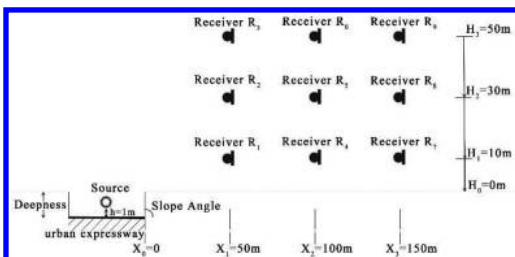


Figure 4. Typical configurations of road cuttings with deepness and slope angle, and receiver locations.

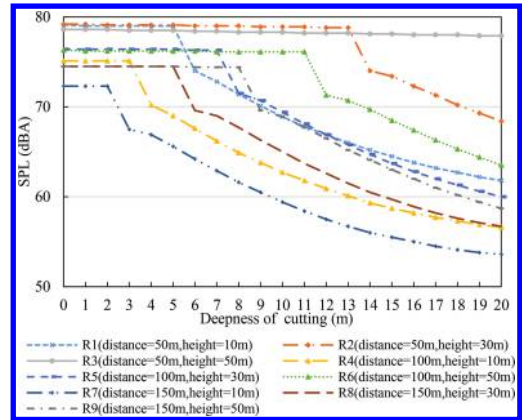


Figure 5. Sound attenuations along with deepness of the road cuttings (0 to 20 m), under varied distances between receivers and road (50 m, 100 m, and 150 m), and heights (10 m, 30 m, and 50 m).

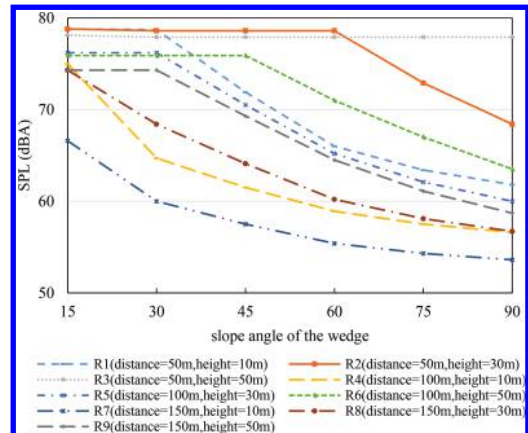


Figure 6. Sound attenuations along with slope angle of cuttings (15 to 90 degrees), under varied distances between receivers and road (50 m, 100 m, and 150 m), and heights (10 m, 30 m, and 50 m).

R7 (distance = 150 m, height = 30 m) as an example, the effect of the cutting deepness on noise reduction could be as significant as 4.8 dBA for only 1m deepness variation (from 2 to 3 m), as a result of shadow zone influence (see Figure 5). After the calculations on the sound attenuations for the slope angle, 90 degrees might be working more effectively in noise reduction, as shown in Figure 6. As the case of receiver R8 (distance = 150 m, height = 30 m), there would be 1.4 to 5.9 dBA reduction when the slope angle increased by every 15 degrees.

A noise barrier may be erected on top of an earth mound to reduce the horizontal land take (Bougdah et al., 2006). In this study, as simulated,

the 4 m basic form of noise barriers on the urban expressway side resulted in poorer noise reductions for the high apartment levels, therefore the road cuttings or earth embankments, as the particular landform condition of mountainous cities, would be good natural noise barriers. Moreover, tree belts could decrease traffic noise owing to the diffusion effect (Kang 2000, Heimann 2010). Thus, nearby green areas might be helpful to escape noisy surrounding, and to lessen stress-related psychosocial symptoms and other adverse effects from traffic noise exposure (Guastavino 2006, Gidlof-Gunnarsson and Ohrstrom 2007).

4 CONCLUSIONS

It has been shown that the buildings layout of roadside residential buildings has close correlations with spatial distributions of noise levels. The rectangular-shaped apartments on the roadside provide a better capacity of noise reduction than tower-block apartments. The acoustical performance of road cuttings, as an appropriate designed earth mounds, is improved along with deeper vertical alignment and bigger slope angle towards the right-angle. Based on these pilot findings, future research would be needed to further explore the quantitative relationships between urban acoustic environment and other possible urban morphological factors for mountainous cities. It is also possible to optimize the noise modelling in hilly terrain, in order to generate practical design and urban planning guidance in mountainous cities.

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Integrating environmentally sustainable design principles into livable neighborhood

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ABSTRACT: To respond to the environmental concerns raised by patterns of human development, changing the way that land-use decisions are made, to adopt a sustainable framework is inescapable. In view of this fact, to limit the built environment impact on the natural environment, and therefore to produce a built environment that mimics and complements rather than conflicts with nature is emerging as a vital goal for those who design cities, neighborhoods, and buildings. At the same time, the second important factor for land-use planning is the term “livability” that describes quality of life and the characteristics of cities that make them livable. Traffic, health, safety, the environment, and many other factors are important to quality of life, and all are influenced in some way by the built environment. The key question of this paper is: whether and to what extent we can integrate environmentally friendly design concepts into our livable neighborhood. To find the relationship between “livability” and “sustainability” in neighborhood design, this paper first highlights the key elements of livable neighborhoods, and their design components. Second, the environmental consequence of the elements will be determined. For example; how design for physical activities—like providing walking paths—can influence directly or indirectly on the environment. Moreover the paper seeks to find ways to add efficiency to sustainable design solutions. The results indicate the benefits and problems with environmentally designed livable neighborhoods.

1 INTRODUCTION

Many of the acknowledged environmental challenges are directly or indirectly caused by land use and our built environment (Kibert C. J., 2002) that is generally defined as any non-natural development that have taken the place of the natural environment. The urban form directly affects habitat, ecosystems, endangered species, and water quality through land consumption, habitat fragmentation, and replacement of natural cover with impervious surfaces. Certain patterns of development also encourage increased use of motor vehicles, which is associated with growth in carbon emission and the greenhouse gases in atmosphere that contribute to global climate change, air pollution, water pollution and the like (EPA, 2001). Consequently, to limit the built environment impact on natural environment raises a challenge for designers and planners, and to produce a built environment that mimics and complements rather than conflicts with nature is emerging as a vital goal for those who design cities, neighborhoods, and buildings (Wilson, 1999). In sustainable urban infrastructure, the incorporation of sustainability principles in neighbourhood design is important because many of the problems encountered at the macro-city scale are in fact cumulative consequences of poor planning at the micro-neighbourhood level (Engel-Yan, 2005).

This paper therefore looks at a strategic framework that provides us insight into environment-friendly techniques, for neighborhood pattern and design in particular.

Another important point worth considering is the land-use planning in terms of “livability” (Godschalk, 2004) that is increasingly used to refer to quality-of-life issues important to the long-term well-being of people and communities (Wheeler, 2001). Livability is also best defined at the neighborhood scale and is keenly aware of its character. A livable community identity can be embodied in its physical features (from highly recognizable topography and climate-specific vegetation to public plazas and architectural styles) and in the actions of its residents (from public events to social programs). Traffic, health, safety, the environment, economic development, a sense of community all these and many other factors are important to quality of life, and all are influenced in some way by the built environment (AIA, 2005).

The notion of both sustainable communities and livable neighborhoods leads us to some interesting explorations about the neighborhoods and communities of which we are a part, and that we have to engage with (Smith M. K., 2008).

The terms livability and sustainability in several studies are used interchangeably, while in some

other a conceptual overlap has been emerged between livability and sustainability. Nevertheless, a greater number of resources referred to livability and sustainability as separate and discrete concepts. This paper accordingly considers these two terms as separate concepts and intends to find their potential to be integrated in neighborhood design. The key question of this paper is: whether and to what extent we can integrate environmentally sustainable design concept into our livable neighborhood. Based on literature reviews, this paper explores the fundamental elements of a livable neighborhood. The AIA's Livability 101, as an approach to build more livable communities (AIA, 2005), helps this study to indicate the principles. To determine whether or not the livability principles cause environmental impact, this paper refers to "*Our Built and Natural Environments*" (EPA, 2001). The book "*Our Built and Natural Environments*" is a resource for technical analysis on the implications of land use and transportation for the environment in state, local governments, academics, and those studying the environmental impacts of urban form (EPA, 2001). In addition to these resources, other studies are also reviewed in support of the paper arguments. The paper then concludes the role of environmentally sustainable design principles in protecting and improving natural environment in ways likely to improve the neighborhood livability too.

2 LIVABLE COMMUNITIES: THE ELEMENTS, THE ROLES, DESIGN PRINCIPLES & ENVIRONMENTAL EFFECT

"Is there an integrated design of neighborhood that not only enhances the quality of life and livability, but also diminishes the environmental impact of our built environment?" There are several studies that seek to answer this question and to understand the relationship between environmentally sustainable design of neighborhoods and livability. The existing studies mainly conducted to focus on human perceptions of the local built environment (Boeing, 2014) and to evaluate how the neighborhood design and planning criteria in environmental assessment tools such as LEED-ND aims to foster a sense of livability in a project area (Clark, 2013). This paper reviews the literature on the connection between livable neighborhood design elements and environmentally sustainable design principles to examine their potential to be integrated and also to determine their negative or positive correlation.

This paper first highlights the key elements of livable neighborhoods, their importance and design principles (AIA, 2005). Second, the environmental

consequence of the elements will be determined and finally, the paper explores how various development techniques and alternatives designs can minimize environmental damage (EPA, 2001).

Livability 101 offers eight fundamental planning principles that can or should be adopted in design for livable neighborhood. Below this paper only considers four major principles including "Mixed-use development", "Density", "Street-savvy design", and "Physical health and community design". The elements will be analyzed in detail and their relationship with sustainable design principles will be detected.

2.1 Mixed-use development

Mixed-use development is a variety of appropriate uses within walkable distances that provides convenient access to services while creating a vibrant backdrop for commercial and social exchange. At the neighborhood level, mixed-use development refers to the arrangement of different uses across several blocks or acres of land so that they are not physically isolated from one another (EPA, 2001). The shift away from the segregation of land uses and back toward the integration of them has established mixed-use development as an important paradigm in the planning and development worlds. Consequently, the distances that people were able to travel on a daily basis were limited to what could reasonably be traversed on foot in the small amount of time that was available between sleeping and working. This limited both the overall size of the city and the amount of space that was available for each necessary function. (Herndon, 2011). According to Niemira (Niemira, 2007) a mixed-use development is a real estate project with planned integration of some combination of retail, office, residential, hotel, recreation or other functions. It is pedestrian-oriented and contains elements of a live-work-play environment. It maximizes space usage, has amenities and architectural expression and tends to mitigate traffic and sprawl.

The benefits of mixed-use development include safer, vibrant neighborhoods, less traffic, reduced air pollution, improved public health, and increased economic activity, among many other things. For example, having numerous uses within a close distance creates a situation where it is reasonable for people to walk or ride a bicycle to their desired destination. This reduces people's need to rely solely on their automobiles for transportation, and fewer automobiles on the road will likely lead to less traffic congestion and lower volumes of air pollutants entering the atmosphere. This in turn has the potential to improve air quality and benefit public health. However, each of these benefits which are not guaranteed to occur because each is

reliant upon factors external to the mere presence of a mixed-use development (Herndon, 2011).

The components of mixed-use development that are generally agreed upon are; first, the project must consist of multiple uses that are physically and functionally integrated and which are substantial enough to attract their own markets. Second, the project must maximize space through intensive land use and be oriented toward the pedestrian. Third, each component of the project must conform to an overarching, coherent plan. These three principles make up the core of the consensus regarding mixed-use development. Parking, public spaces and street-scapes, the integration and interrelationship of elements, pedestrian circulation and connectivity, and creating a sense of place are just a few of the design elements that are crucial to the success of a mixed-use project (Herndon, 2011).

Mixing land uses can have direct and indirect effects on habitat loss and runoff since mixed-use developments have the potential to use surface parking lots and transportation infrastructure more efficiently, requiring less paved area (EPA, 2001).

For example, when office buildings contain retail shops and restaurants, the infrastructure that supports the building—the roads and parking lot—is in use for more of the day.

Several empirical studies have found that in most cases, the average distance per trip driven by residents of mixed-use neighborhoods was half that of those living in the single-use neighborhood. By encouraging people to walk, bike, and use transit rather than drive, mixed-use development patterns reduce VMT and consequently lead to decreases in automobile emissions, thereby improving regional air quality. Furthermore, mixed use has environmental benefits resulting from the form of less stormwater runoff because of reducing the impermeable surface and lots (APA, 2004).

2.2 Density

Increasing the number of housing units per acre goes hand-in-hand with mixed-use development. Planners must carefully consider the density required to foster lively main streets. In a dense neighborhood, a large number of services can be located within walking distance from dwellings and in close proximity to each other.

Higher densities can help by providing opportunities to use more expensive units to subsidize lower-cost units in the same development or neighborhood. In addition, increased densities can provide the resources to pay for parks, schools, and other amenities essential to building livable neighborhoods. The public sectors can no longer afford to subsidize lower-density, sprawl-dominated development. Total vehicle-miles driven in the

Boston region have increased 15 times faster than population since 1970. Moreover, gasoline consumption rises in direct proportion to decreased density among American cities. Hence, higher densities are essential to halting escalating traffic congestion and offering people greater options. (Dixon & FAIA, 2005).

Higher densities represent one of the most effective ways to create walkable, mixed-use districts.

Density is a number of units—people, dwellings, trees, square feet of building—in a given land area. (Forsyth, 2003). The kind of design that builds a community is not a function of density. In truth, one can find both good and bad examples of architecture and planning at various levels of density (Dixon & FAIA, 2005).

Compact metropolitan development generally means that the space needs of a population can be satisfied with less land area. Communities can develop more compactly by using three techniques (EPA, 2001):

- Infill development
- Brownfield redevelopment
- Cluster development.

Infill development occurs in locations where some development has already taken place and infrastructure is already in place.

As a particular kind of underdeveloped land, brownfields have received significant attention as both a problem and a potential source of multiple urban benefits. Because of uncertainties about environmental liability, clean-up costs and potential presence of contaminants, site owners, developers, and lenders often avoid investing in brownfields. Rather than developing brownfields, firms and investors instead turn to surrounding areas and undeveloped greenfields or relatively untouched and uncontaminated land (Brownfield).

Cluster developments are built at gross densities comparable to conventional developments but leave more open space by reducing lot sizes (Quality Growth, 2014).

Compact development has some clear effects on the environment that are a direct result of reduced use of land and the nature of development or redevelopment (EPA, 2001):

Reduced disruption and fragmentation of habitat: Compact development minimizes land conversion to urban use, and maximizes retained natural habitat. Infill development of all kinds, including brownfields redevelopment, reduces development in more pristine areas since an acre built in town is often at least an acre less that is developed on a greenfield. When existing infrastructure is renovated, infill development can spare natural resources such as the wood and metal that would be needed for the construction of new facilities.

Reduced impervious surfaces, resulting in improved water quality: Infill development accommodates new growth with significantly less impervious surfaces per unit of development than development on undeveloped land does. By reducing the need for impervious surfaces, compact development significantly reduces runoff and water pollution. Studies show that the impervious surface area of a clustered development site is often 10 to 15 percent less than that of more dispersed development, depending on the size and configuration. In addition, less storm water runoff and pollutant loads are found in cluster developments, due to reduced impervious cover.

Clean-up of contaminated waste through brown-field redevelopment, which can reduce water pollution and community environmental risks: Past industrial activity bequeathed a legacy of soil and water pollution at formerly industrialized sites. Thousands of these brownfields are located in densely populated, urban areas where residents tend to congregate and children play. To clean-up and redevelopment these sites can lead to substantial environmental and human health benefits. Potential public health benefits of brownfields clean-up include reduced blood lead levels, decreased risk of cancer, and minimized respiratory problems. The encapsulation (“capping”) of contamination on brownfields can reduce runoff of toxics into nearby bodies of water. This reduced runoff can lead to improvements in overall water quality and, consequently, improvements in habitat. Brownfields redevelopment also benefits the environment; removing toxics from the site can result in cleaner soil and improved water quality. In addition, many brownfields are located in urban areas that are already served by existing sewer lines, roads, and other infrastructure. Redeveloping these properties can consequently minimize greenfield development. In addition to direct effects on habitat, water quality, and brownfields, compactness in a metropolitan area also affects travel activity. Increased compactness is viewed as a means to reduce vehicle travel because it affects travel demand in the following ways:

- Compactness increases the number of activities accessible within a given area.
- Locating activities closer together allows trips to be made by walking and bicycling rather than motor vehicle.
- Vehicle ownership—Dense areas reduce the need for vehicle travel for personal mobility, so people are less likely to own as many vehicles. Dense areas also tend to be associated with limited parking availability and higher cost.

2.3 Street-savvy design

As the automobile prevailed, other transportation modes that are more conducive to face-to-face

interaction—such as walking, bicycling, and public transit—gradually declined. The speed of automobile travel brought about the need for larger signs and signals, and a demand for convenient parking. Fortunately, today’s urban planners and city leaders have learned from the experience of the last century. People, by nature, are drawn to vibrant, sociable venues. To ensure a healthy neighborhood, this dynamic must begin at the level of the street. Today, there is a growing movement to reclaim our streets from the tyranny of the car—and the isolation it generates—by offering more transportation choices and making roads more-friendly to people (Vanderslice & AIA, 2005).

Replicated thoroughfares over and over across the cities the same as ribbons of asphalt lined by parking lots and giant illuminated signboards, lacks any sense of place and fosters a feeling of estrangement among the citizenry. An interconnected network of streets makes it possible for people to reach destinations by the shortest possible trip. In contrast, street systems with many cul-de-sacs discourage walking and biking and create unnecessary traffic. Connectivity is important for good transit as well. Transit networks that offer frequent service over a widely connected network draw more riders than those with limited service and out-of-direction routes. Trails and paths are not a substitute for on-street facilities, since most destinations and transit methods lie on the street system, but they can provide short-cuts or alternate routes of interest. Trails or paths through parks or along waterways can add to the quality of a trip as well as shorten it.

Sidewalk space and bicycle lanes should be added or enlarged adequately. Sidewalks should provide a walking path that is safely separated from motorized vehicles. Good side-walks are made up of at least three distinct zones: a *through zone*, a *curb zone*, and a *building zone*. The through zone in residential areas should be wide enough to accommodate two people walking together with room for a third to pass. No obstructions should be allowed within this circulation path. The curb zone serves as a buffer between walkers and the roadway. It may be landscaped or paved. This is the place for street trees and furnishings, such as pedestrian-scaled streetlights, signs, utility poles, bike racks, and parking meters. The building zone is the area where storefront activities can spill onto the sidewalk without impeding passers by, whether on foot, in wheel-chairs, or moving with some other means of assistance. This zone plays a particularly strong role in luring people onto the public streets and engaging them with their community. Other features are; bicycle lanes, bicycle boulevards, safe and convenient bicycle parking, safe zebra crossing and street corner (where walkers congregate and then cross the street) and it is the location for

traffic hardware such as street—name signs, traffic signals, and utility poles.

Cities can expand transit supply through the construction of new transit systems or the expansion of existing facilities. Transit systems that are well designed and operated can reduce vehicle travel, resulting in reduced criteria pollutant and greenhouse gas emissions. For some areas, the most cost-effective way to improve transit access may be to increase development around existing stations, rather than extend transit service. This type of development is typically referred to as Transit-Oriented Development (TOD). The premise of transit-oriented development is that locating residential development and employment near transit stations increases the market for transit services and yields greater ridership than is achieved at stations (or bus stops) surrounded by low-density development.

Effective transit systems require supportive land use patterns. Locating high-density commercial and residential development around transit stations is expected to reduce vehicle travel by providing mode choice and substitute for vehicle travel and is expected to reduce the need to own personal vehicles for mobility.

2.4 *Physical health and community design*

Most of the routes we travel along daily—between home and work, school, stores, and recreational venues—are not designed for safe walking or biking, even for those who happen to live close enough that they could, at least in theory, leave their cars at home. The lengthy distances to our daily destinations mean that many people spend a greater part of their day in their car, which leaves less time for engaging in the minimum recommended amount of regular physical activity. As a result, physical activity is no longer an integral part of daily life for all but instead is a distinct, programmed event for only those who are both highly motivated and have the time.

There is growing evidence that our physical health is directly tied to our physical activity. A surge of studies presented in public health journals and conferences, plus increased funding for research into links between physical activity and health, indicates the growing interest in the health professions in how the design of the built environment influences health. People of all ages suffer from an environment that is poorly designed for physical activity and mobility. However, the young and the old, and those who care for them, bear the brunt of the problem. Childhood activity today is mostly a scheduled and transported event and children are more physically isolated and inactive than those of previous generations and are suffering in larger numbers from obesity and other

chronic health conditions related to the lack of physical activity. And the very old, who typically lose their ability to drive, are left with options that drastically diminish the quality of their lives: They either remain home—bound and isolated, or must move out of their homes and communities for unfamiliar age-segregated retirement communities or institutions. These settings remove them from lifelong social networks, which can greatly affect their mental and social health (EPA, 2001).

Providing networks of limited-traffic residential streets, sidewalks, bike paths, and greenways that connect adjacent residential neighborhoods to each other and to nearby commercial and civic services (e.g. public schools, libraries, learning resource centers, recreation and gyms) so that a greater number of people, young and old, can more easily walk to places and avoid busy connector and arterial streets. The next is, providing greenways, bike paths, and jogging trails within floodplains and utility rights-of-way between existing, isolated neighborhoods and landscape buffer should separate sidewalks and bike lanes from vehicular traffic on all streets. Provide deciduous street trees between the road and sidewalks to make walking more pleasant and tolerable in hot and cool weather. Street trees can also provide pedestrians with a real and perceived sense of security and separation from vehicular traffic. Furthermore, canopies on commercial storefronts for sun and rain protection, adequate lighting of pedestrian pathways to ensure safety and security at night, front porches on residence are of effective factors to promote walking and biking instead of driving.

Aspects of the built environment such as building orientation, street connectivity and design, and building design all contribute to the relative friendliness of that area to pedestrians and bicyclists, and to the general aesthetic appeal of an area. Several studies have found that higher levels of pedestrian and bicycling activity occur in areas with street connectivity and pedestrian amenities than in those without those features. The most significant environmental benefit of good urban design is that it reduces auto travel by enabling other travel choices and less air pollution generation.

3 IMPLEMENTATION OF SUSTAINABLE SOLUTIONS: EFFICIENCY AND QUALITY OF LIFE

The above design principles have demonstrated positive environmental implications by reducing infrastructure requirements, reducing vehicle travel, reducing land and habitat consumption, reducing water consumption and pollution. However, the efficacy of these practices depends on how well

they are implemented. Below are some alternatives and potential solutions in neighborhood design that their combination with the strategies above can be applied to improve efficiency of the design principles in neighborhoods.

3.1 Porous pavement

In developed environments, impervious surfaces are quite possibly the most ubiquitous structures created by humans (Frazer, 2005). Impervious surfaces are generally understood to be any material, natural or man-made, that prevents the infiltration of surface water to the underlying strata. These surfaces not only indicate urbanization but are also major contributors to the environmental impacts of urbanization (Kampouraki, 2010). As a result, in built areas with extensive impervious surfaces, water can't soak naturally into the earth, causing a host of serious environmental problems such as; increased polluted runoff (Runoff, 2003), surface water flooding and streambank erosion (Gibbons, 1998), reduced groundwater recharge (NHEP, 2007), formation of stagnant water, heat island effect (Permeable vs. Impermeable Surfaces, 2009), and habitat loss (EPA, 2001).

Despite all negative relationship between impervious surfaces and the environment, no one would suggest not to pave streets, parking lots or sidewalks. So, is there any solution? Fortunately yes! There are several alternative potential solutions that reduce the adverse impact of paved surfaces on environment. They are discussed as followings;

3.1.1 Permeable paving

Provide maximum permeability of surfaces in landscape. Permeable pavements allow water to infiltrate or pass through them (NCDENR, 2007) where water tends to gather and become stagnant.

Numerous type of permeable pavements are available including pervious concrete, pervious asphalt, Permeable Interlocking Concrete Pavers (PICPs), concrete grid pavers, and plastic reinforced grass pavement.

These pavements are all similar in several ways. They usually contain a gravel storage layer underneath the surface pavement, which often doubles for structural support.

Permeable pavements are typically targeted for paved areas with low traffic volumes. Some examples of appropriate uses for permeable pavements include patios, residential parking pads, driveways, fire lanes, overflow parking areas, and some daily parking areas. For runoff reduction, a permeable pavement on flat slopes tends to work better. Deeper gravel layers under the pavement allow for more storage of rainwater. Permeable pavements will function for up to 20 years if they

are constructed in areas free of disturbed soil and regularly maintained (Muench, 2011).

3.1.2 Permeable interlocking concrete pavements

Interlocking concrete pavements have patterns that create openings or drainage holes for rainfall to enter, while maintaining high side-to-side contact among the units for stability under vehicular loads (Smith D. R., 2006). The openings are filled with pea gravel aggregate, but can also contain top soil and grass (Oluwapelumi, 2012).

3.1.3 Green roofs

In dense urban areas with flat roofs, green roofs—planting the roof with either low-maintenance native plants or growing food—can be a key strategy (Polycarpou, 2010). Roof greening presents a feasible design strategy for microclimate amelioration and energy conservation in cities (Peng & Jim, 2013) and would cool the climate in all of the cities, with the greatest effects in the hottest cities (Alexandri & Jones, 2006).

Greening of rooftops, reduce the impacts of stormwater runoff by reducing the impervious surface within a developed zone (Weiler & Scholz-Barth, 2009).

Plants reduce carbon dioxide in the atmosphere and produce oxygen. Plant roofs remove heavy metals, airborne particles and volatile organic compounds. Being absorbed into the green roof system these polluting particles do not enter the water system through surface run off leading to improvement in water quality.

Vegetation, including green roofs, limits the Urban Heat effect creating a greener and hence more livable appearance and atmosphere, and helps enable more optimal use of urban space (2014). Furthermore, well-designed green roofs can provide habitat compensation for rare and endangered species affected by land-use changes (Brenneisen, 2003).

3.1.4 Rain gardens

A rain garden for stormwater infiltration is a landscaped garden in a shallow topographic depression of small area that receives stormwater from a roof or other connected impervious surface.

The garden plants, usually native species with aesthetic attributes, provide a biologically active root zone that helps maintain soil infiltrability through macropores. (Dussaillant, Wu, & Potter, 2004).

4 DISCUSSION AND CONCLUSION

This review studied the literature concerning the interaction between environmentally sustainable design and livable neighborhood design. It is aimed

at finding whether a relationship exists between livability and environmentally sustainable design principles.

This study after selecting four key features of livable neighborhood, investigates how this features touch the natural environment through combining different reviews.

The results showed that, there was a direct relationship between environmentally sustainable design principles and four key factors of livable neighborhood design; mixed-use development, density, Street-Savvy Design, Physical Health and Community Design.

Mixed-use development promotes safe and vibrant neighborhoods and has potential to use parking lots and transportation efficiently and therefore less paved area, and because of that less stormwater runoff generation. Additionally the average distance per trip driven has dropped by half in mixed use development pattern that encourages people to walk. Consequently leads to reduced automobile emission, thereby improves regional air quality.

Density as another factor in livable neighborhood design, along with mixed-use development, leads to a number of services that can be located within walking distance that consequently increases the physical activities and therefore reduces automobile emission. In addition, dense areas are targeted on infill and brownfield development and their revitalization that benefits for both environment and human health. Cluster developments, by reducing the built-up area and impermeable surfaces, and leaving more open spaces, diminishes disruption and fragmentation of habitat.

Street-Savvy design, by well designed and operated transit systems offers more transportation choices and makes roads more friendly to people. Also locating high-density commercial and residential development around transit stations reduces vehicle travel by providing mode choice and substitute for vehicle travel and expects to reduce the need to own personal vehicles for mobility. In addition to increasing residents' physical activity, street-savvy design reduces vehicle travel, resulting in reduced criteria pollutant and greenhouse gas emissions.

Physical health and community design, focus on separating sidewalks and bike lanes from vehicular traffic on all streets, and also providing limited-traffic residential streets, sidewalks, bike paths, and greenways that connect adjacent residential neighborhoods to each other and to nearby commercial and civic services so that a greater number of people, young and old, can more easily walk to places and avoid busy connector and arterial streets. As a result, physical health and community design by increasing physical activities such as walking and

biking and by reducing traffics and need for automobiles will benefit the environment in many ways.

Furthermore, this paper explored some alternatives and potential solutions in neighborhood design that their combination with the strategies above can improve efficiency of the design principles in neighborhoods. The results show that, alternatives such as porous pavements, green roofs, and rain gardens, not only enhance the efficiency and improve environmental impact of built environment, but also create cleaner and aesthetically pleasing places for residents.

This paper concludes that the need to create a livable neighborhood does not necessarily mean that we should neglect to consider environmental issues or vice-versa. Surprisingly, the results show that not only creating a livable environment poses a threat to natural environment, but also they go along with each other to develop a human-oriented sustainable environment.

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Green building cluster: Key to China's green building industrialisation?

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ABSTRACT: Green building has become a primary development goal in China, but how to promote the scaling development of the green building industry remains challenging. This paper overviews the development of the global green market and identifies that green building has already become a global trend with important business opportunities. It further utilises the concept of 'Industrial Cluster' (Porter 1990) and discusses its adaptation to green building cluster since the late 1990s overseas. Cases in Europe and America prove its multiple benefits to the cluster members, cluster management, industrial development and regional economic growth. An effective green building cluster combines the traditional building sector with emerging areas of opportunities, and adapts to the local industrial basis and development conditions, including demand, factor, context, and related supporting industries (Diamond Model by Porter 1990). It argues that by extending the industrial chain, developing new knowledge, products, services and business relations, as well as inducing scaling impacts within and out of the cluster, the green building cluster conforms to the primary requirement of sustainable development and industrial upgrading. This paper concludes that the green building cluster has the potential to identify new growth points and promote large-scale green urban development in China, which further contributes to the reform of the prevalent yet unsustainable urban development in China which is property-led, investment-driven and energy-intensive.

1 INTRODUCTION

In most developed nations, building energy consumption accounts for about one third of total energy consumption (CCICED 2009). The centrality of green buildings to energy efficiency and sustainable development has been increasingly recognised in literature (e.g. Kats et al. 2003, Nelson et al. 2010, Liang et al. 2007, Bardhan & Kroll 2011), although the path towards pursuing such progress has been varied and challenged. Recently, green building has fast risen to be a national development strategy in China (World Bank 2001, Zhao & Wu 2009, Li & Colombier 2009). In 2012, the Chinese government put forward the strategic goal of large-scale development of green building through financial incentives and mechanisms, marking the first serious government response at the central level (MOF & MOHRUD 2012). In less than one year, the State Council (2013) stipulated *Green Building Action Plan*, imposing a one billion m² of new green building spaces, accounting for 20% of total new built stocks by end of 2015. Such proportion has been further increased to 50% by 2020. Moreover, in 2015, a stricter version of *Green Building Assessment Standard* has been implemented (Xinhua News 2014a), indicating an important transition of government strategy from promoting towards nationwide mandating.

Given the extensive industry chain of the building sector and the massive scale of new construction in China, averaging 1-2 billion m² annually (CCICED 2009), this ought to be translated into enormous business opportunities and a major step forwards towards sustainability. However, according to government statistics, in 2013 alone, among the 1.7 billion m² new starts, green building only amounted to 86.9 million m², accounting for about 4.9% (163 News 2014). It thus becomes a pressing need to identify pertinent solutions to coordinate public and private initiatives and industrialise the green building sector. In this regard, the notion of green building cluster has been attempted in some western countries for the past one or two decades, indicating its capacity of expanding business networks, deepening industrial development and promoting regional comprehensive development (Allen & Potiowsky 2008, Berk & Hodgins 2005, ECOPLUS 2010, Barsoumian et al. 2011). Debates and experiments (e.g. Wuxi & Shenzhen, China) have also been found recently on whether green building cluster could be contributing to China's green campaign, which tends to be more massive in scale and more limited in time. This paper thus offers an overview of the trend and potential of global green building market. It then introduces the concept of cluster and application of green building cluster, and finally analyses the multiple benefits of

green building cluster through cases in the US and Europe. It argues that an in-depth understanding of the forces and factors that underpin the multiple impacts of green building clusters overseas brings valuable insights for China, where total the green building market is estimated to have a RMB 1000 billion market scale (Xinhua News 2013b). This sheds light to the exploration of its potential to industrial upgrading, energy security and low-carbon development in China, which is key for its next stage development.

2 GREEN BUILDING MARKET

In response to the global challenge of climate change and energy crisis, the concept of green building was initially put forward to reduce greenhouse emission and increase efficiency of energy use. It is mainly featured with saving energy and resource, protecting environment and creating spaces suitable for human activities (Nelson et al. 2010, World Bank 2001). This conforms the essential objectives of sustainable development and represents the future development of buildings. However, since its initial conceptual establishment in the 1960–70s (e.g. McHarg 1969), its practical development has not been straightforward but rather complex and challenging. After a few decades' efforts, mainly ascribed to the public compulsory regulation, such as those stipulated by the European Commission, the vicious loop of blame (Ratcliffe et al. 2003) has started to be reversed towards a virtuous one [Figure 1](#). In recent years, tenant demands for green spaces and its positive environmental performance are increasingly becoming the mainstream market drivers, releasing its market potential in scale. Actually, the public policies in Europe is already deemed to make green

building a common standard for both new and existing buildings in the coming decade (Nelson et al. 2010). According to the estimates by McGraw-Hill Construction (2013), the industrial scale of green building is expected to largely expand from \$90.6 billion to \$140 billion during 2009–2013, indicating the accelerating and explosive development of global green building market. It is worthwhile to note that such massive development is likely to be a major trend with limited negative impacts affected by the financial crisis since 2008.

In spite of the higher initial investment of green building than traditional building (R. and Vin), the commercial value of green building has become to be fully understood. In this respect, World Green Building Council (World Green Building Council 2013) conducts a comprehensive analysis to elaborate the evident commercial returns of green investment in five main aspects. These include cost reduction of whole life cost, covering design, construction and operation stages, asset value appreciation, enhancement of work productivity and health, reduction of energy consumption, as well as risk mitigation. This is due to the expectation that not investing in green would be deemed as a risk when it becomes a compulsory requirement by government. This has been a step forward that is crucial to a fuller understanding of the benefits of going green in the building sector, central to stimulating further expansion of market demands.

Hence, a gradual transition from mainly government mandating towards a public-private commitment to green building is increasingly notable. This is largely reflected in the global survey conducted by McGraw-Hill Construction (2013). It involved 803 enterprises and institutions in 62 countries, ranging from design, construction, building, professional companies as well as manufacturers and suppliers. By covering most of the industrial chain, the survey results revealed that over 51% of them expect more than 60% of their business to be related to green building business. This has been a substantial increase compared to 2009 when only 13% of them considered alike ([Fig. 2](#)). Therefore, private sector has become an important input for pushing the green buildings business, driving market forces to have a fuller play for the next stage development.

A global green market is being shaped. According to the survey by McGraw-Hill Construction (2013), a strong green activity has been found as statistically significant sample responses from the nine countries ([Fig. 3](#)) have reported more than a quarter of their project work was green in 2012. This trend of increased green building activity is not limited by location, economic conditions or culture, although at somewhat different rates and with different focus and status. Many developed countries in the West have formed a relatively mature market such as

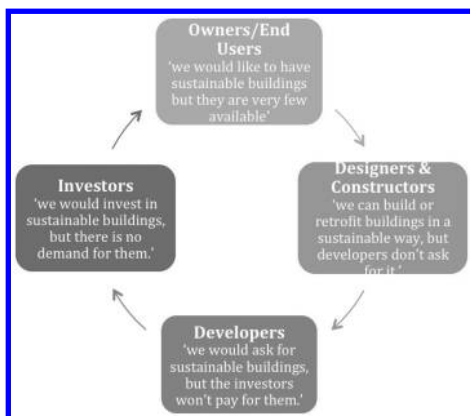


Figure 1. Vicious circle of blame of development of green building development.

Source: Ratcliffe et al., 2003.

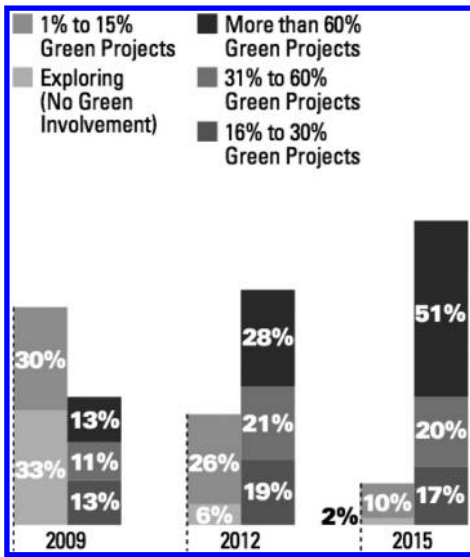


Figure 2. Levels of green building activity by firms around the world (2009–2015 expected).
Source: McGraw-Hill Construction, 2013.

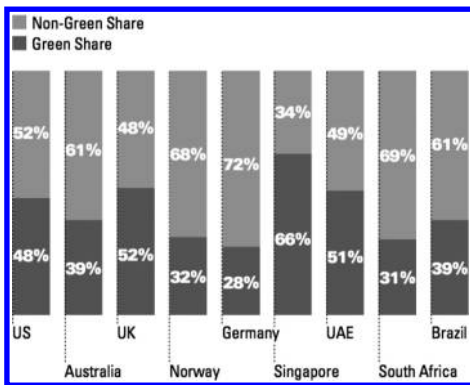


Figure 3. Average 2012 green share of building project activity (by location).
Source: McGraw-Hill Construction, 2013.

UK, US and Australia. Yet some developing countries have seen rapid growth in recent year, such as Brazil. The focus of development also varies when most European countries concern the retrofitting of existing buildings, many developing areas tend to more emphasise on new building market.

Growth of green activity is to be further expanded in both developed and developing regions. According to survey respondents, most of the nine countries are likely to have over 20% increase of firm involvement in green business, some of them even about 40% (Fig. 4). In spite of this, Xue (2010) argues that, for the next five years, Asia is expected to see the largest

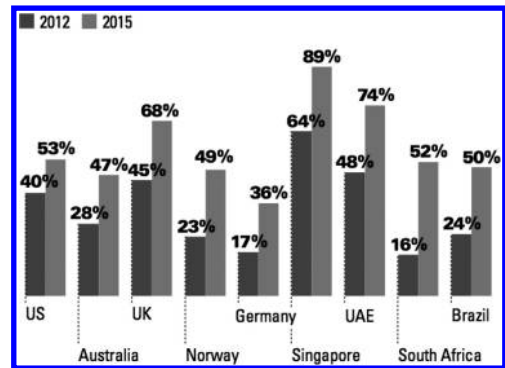


Figure 4. Percentage of firms with more than 60% of work green (2012 and expected for 2015).
Source: McGraw-Hill Construction, 2013.

growth of green building businesses from 26% to as many as 73%, where China is likely to play an important role. Therefore, green market is not restricted to a geographic place or economic environment, but rather a fast growing global trend that spread across countries. How it is approached largely relies on local features and market conditions.

In China, building energy consumption accounts for about 28% of total energy consumption, lower than the average level of developed nations. This is partly due to the massive energy consumption for industrial development. With a total stock of about 40 billion m² existing buildings and about 2 billion m² new buildings annually, China has already become the world's largest construction site (CCICED 2009). Such a trend is expected to continue under the massive acceleration of urbanization that is about to make urban population surpass 1 billion by 2020 (McKinsey Global Institute 2008). However, about 80% to 90% of existing buildings are of high-energy type. Therefore, given its grand scale, the building sector is expected to see an upward pressure on improving energy efficiency. In this respect, China, with its current strong government commitment has massive potential to be released. This should be incorporated into the on-going national campaign towards industrial upgrading and transformation, promoting the long-term urban resilience of cities and regions.

3 GREEN BUILDING CLUSTER

3.1 Cluster theory and 'diamond model'

The concept and application of cluster has been put forward by Porter (1990) when discussing competitive advantages of nations. He contends that cluster commonly exists in developed economies, playing a critical role in economic growth and

competitiveness. Economic clusters refer to “geographic concentrations of interconnected companies, specialized suppliers, service providers, firms in related industries, and associated institutions (for example, universities, standards agreements, and trade associations) in particular fields that compete but also cooperate” (Porter 2000). As they all belong to one or multiple industrial chains, it not only represents a geographical concentration, but also involves a new form of economic agglomeration and network activities. Such important and dynamic exchanges within an industry or sector was scrutinised by Markusen (1985) through use of an input-output model. Clusters were also claimed by Storper and Christopherson (1985) as able to provide greater resilience to a regional industry, allowing it to adapt to changing markets trends.

Compared to traditional way of business, economic cluster is able to capture the interaction, complementarity, and technology spillovers (*ibid*) as well as the interactive relations between customer needs, marketing and information (Porter, 1990). It opens up communication channels between enterprises with the government, the supplier and other institutions, and thus builds up cooperation among them based on common interests. In this way, the cost of exchange could be reduced and efficiency of economy enhanced, which is the core for economic growth according to new institutional economics theory (North 1990). This enhances the productivity and innovative capacity, central to technology advancement and industrial upgrading. As a result, the industrial chain could be extended and adapted to market changes and new demands, towards producing high value-added sectors in the long run. In this regard, cluster has been argued by Porter (1998a) as a “system of interconnected firms and institutions whose value as a whole is greater than the sum of its parts”. It could help create a symbiosis

status for enterprises, being a useful way to promote local economic development and an important indicator for city and regional development.

Clusters differ in terms of size, type of member (large, small and medium-sized businesses), source of finance and area of focus. Yet successful clusters usually have common features, which is summarized by Porter in his ‘Diamond Model’ (1990) (Figure 5), i.e. demand conditions, factor conditions, related and supporting industries, and context for firm strategy and rivalry. This provides a well-structured framework for analysing the components and factors that make clusters work as a whole. It also indicates the need for both combining business conditions and adapting to local industrial basis, and developing new supply chain relationships and institutional environment.

3.2 Green building cluster

The concept of cluster has been employed to the building sector since the late 1990s when some of the developed nations such as US, Europe and Australia started to explore the idea of green building cluster (e.g. European Green Capital 2010, Helmut Berrer 2011, Berk and Hodgins 2005, Allen and Potiowsky 2008). This was approached under the general background of increasing international call for sustainable development and the need to combine this with industrial advancement. Such a cluster represents a complex collection of products, services, policies as well as other related factors in relation to the green building industry. It is much more than a collection of companies commonly referred to as ‘sustainable’ or ‘green’. Rather it is composed of four over-arching types of activity: production, provision, consumption and facilitation (Berk and Hodgins 2005). In Figure 6, Producers are mainly concerned with a green process of materials generation. Providers focus on the supply of materials, designs, services and production that connect finished buildings with end users. Consumers are various individuals and institutions that pay for green buildings, services and materials. Finally, facilitators close the loop by regulating, innovating and educating other parts of the cluster (*ibid*).

The practice of green building cluster in US and Europe tends to have a few common characteristics. First, it becomes important for enterprises and institutions to have frequent interactions between them. In addition to traditional business relations, new business networks need to be created and developed by green building practitioners. For instance, a teamwork relationship needs to be established among developers, architects, engineers and landscape designers, throughout the entire construction process, ensuring their early involvement and holistic planning for projects. Second, new supply

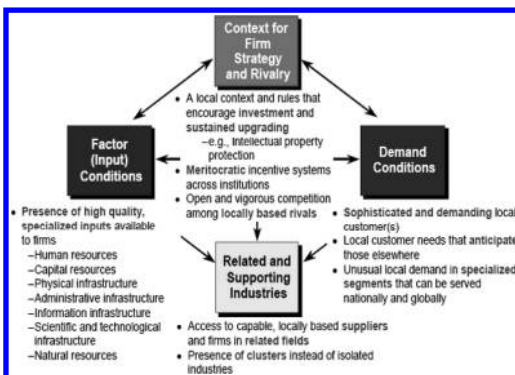


Figure 5. ‘Diamond Model’ of economic clusters. Source: (Allen and Potiowsky, 2008, Porter, 2006).

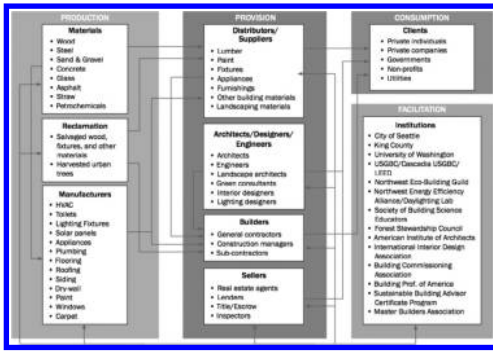


Figure 6. Green building cluster map.
Source: (Berk and Hodgins, 2005).

chain relations and networks are to be created by existing suppliers for green building products and services. This is mainly through institutional setting and rearrangements. Third, there requires a development of new assessment standards for new materials and technologies to be more effectively applied to the green building development.

3.3 Green building cluster applications

To reach a fuller understanding of the suitability and potential benefits of clusters to China's green industry development, an investigation of its multiple contributions is useful. Such investigation is to be conducted from three dimensions, i.e. within the cluster members, towards cluster governance and external impact on regional scale in the following session. This is supported by two case examples, i.e. Seattle (USA) and Lower Austria (Austria). Both clusters have been set up since 2000s and initiated by government commitment. As the largest industrial city in west Pacific region, Seattle has laid strong foundation in the fields of aerospace, information technology, timber processing etc. In the 1990s, Seattle government set up an important economic strategy, which placed developing green building cluster as central to promoting local sustained economic growth (Berk and Hodgins 2005). Its Sustainable Building Policy 2000 stipulated that public buildings over 5000 square feet must obtain LEED certification. As the first of its kind in US, this policy became a catalyst for rapid development of green building industry and cluster. Estimates done in 2005 showed that Seattle has already ranked number one in LEED accredited professionals, and had 230 firms with clear presence in the cluster, affecting nearly 34,000 companies in building and construction (*ibid*).

Lower Austria is the largest federal state in Austria and has built up a solid industrial basis in heavy industries. Since 2001, state government successively initiated five economic clusters, in five

economic areas deemed as most promising to be core of local economic drivers, i.e. green building, plastics, food, logistics and mechatronics. By 2011, in total, the five Lower Austrian clusters have 590 member companies and a combined turnover of €23 billion (Helmut Berrer 2011). Cumulatively a total project volume of €52.2 million has been realized, among which green building cluster, as the largest cluster, has 273 projects with a project volume of €34 million (*ibid*).

4 CONTRIBUTION TO CLUSTER MEMBERS

By geographically close to each other, companies and institutions with related business fields are able to share information, exchange business and expand connections on a more regular and dynamic basis. Deeper understanding of the market trends and development of new technology and techniques could be reached through various and more frequent trade fairs, research symposiums, as well as training programmes organized at the cluster level. New knowledge can be disseminated at a quicker pace and greater scale among cluster members, providing a favourable environment and driver for enterprise innovations. This could further be linked with potential funding opportunities and collaboration business for cluster members, with the assistance of cluster management entities. Take Lower Austria clusters for example, with the effective management of ECOPLUS, three-fourths of the total project volume, equal to €37.6 million, is made up projects receiving financial assistance. Public sources account for as many as 49% of the total funding of these projects (Helmut Berrer 2011).

Being located within the cluster, and connected with other members through business ties, companies could also manage its brand building and win a leading position within the industry. The industrial chain is not static but rather dynamic in development as new products and services are created and new market demands established. Therefore, instead of traditional individual business, cluster members have more instant access to new areas of development and tend to be open-minded and develop a regional market perspective. Being aware of the key development areas help companies to develop its own medium to long-term business strategy and goals.

5 CONTRIBUTION TO CLUSTER MANAGEMENT

Successful clusters often have an efficient cluster management institution to guide and facilitate its operation at the cluster level. The experience of cluster management and relevant institutional

setting are transferrable and beneficial to future development of clusters. For instance, ECOPLUS is a government-authorized business agency, responsible for cluster management (Helmut Berrer 2011, ECOPLUS 2010). ECOPLUS was able to focus on a few key issues: 1) bring together member companies and institutions within and across the industrial chains. 2) Promote R&D and innovation through facilitating the development of training and qualification institutions. 3) Connect cluster members with funding institutions and investors through various ties and organised events. 4) Supervise and coordinate the various projects conducted in the cluster. Such dedicated service has not only created favourable environment for companies especially small and medium companies to grow, but also coordinated the multiple resources of government, enterprise, clients and investors with high efficiency. Within the first decade of 2001–2011, ECOPLUS has attracted 590 members, generated 300 projects, and created 72000 jobs (*ibid*).

The significance of cluster management has been recognized by Berrer (2011) who argued that at least majority of the cluster projects would not have to exist without it, let alone to say the value added and employment effects. Furthermore, the value of ECOPLUS is not restricted to the growth of this particular cluster, but also to the advancement of cluster management and governance experience. Effective management underpins the speed and quality of entire cluster development, and is beneficial to large-scale industrialization development.

6 CONTRIBUTION TO INDUSTRIAL DEVELOPMENT AND REGIONAL ECONOMY

The significance and urgency of developing green building industry has been increasingly recognized as a new growth point for the next stage of development that is prioritised on sustainability. Places and regions with forward-looking have been found to incorporate the green building cluster as their important development strategy. This is largely related to its simultaneous benefits to the development of diverse businesses along the industrial chain. It is also driven by its contribution to establishing new knowledge, new technology and new business networks through clustering, and promoting regional economic development and competitiveness. Its contributions to industry and economy have been illustrated below with examples.

First, green building cluster enhances productivity and competitiveness of enterprises and the industry, as well as the regional competitiveness of the cluster as a whole. This is mainly ascribed to the improved market specification, shared information, diversified selling channels, and professional

training and labour supply. As a system, cluster as a whole is more than the sum of all its parts, bringing market recognition and new opportunities for each member. New knowledge and information can be more extensively disseminated, which further attracts high-quality talents to form a cluster of human resources at the same time. In this respect, the green building cluster in Seattle (USA) has already become the centre for first-class green building professionals, as an additional but crucial advantage of the cluster. This helps solve the common problem as lack of talent in green industry, and providing an important driver for the development and innovation of technology. A survey conducted by the Seattle government indicated that more and more green building design and construction companies desire to set up headquarters in Seattle, as an effective way to consolidate their leading position in business, build up market confidence and demand, and win more regional and international businesses (Berk & Hodgins 2005).

Second, green building cluster not only promotes the cooperation among enterprises but also connections between green building and traditional building industry, so as to advance the entire industrial chain. Building industry is featured with long industrial chain, covering design, research, material production and supply, construction, marketing and sale, building operation, services and maintenance. Development of green building related business further extends the industrial chain with new products and services as well as new relations between producers and customers. While green building gradually becomes the market trend, traditional building sector is also making efforts towards green production process and strategies. This makes it possible and important to have dynamic exchange between the two sectors, integrating the power and resources for the extension and upgrading of the entire building industry.

Based on the data of percentage share of LEED certified commercial buildings, it was estimated that green building industry represented between 4% and 12% of the entire building industry (Berk and Hodgins, 2005). While 230 firms claimed their clear presence within the Seattle Cluster, there was a high possibility that nearly 34,000 companies involved in building and construction industry in Seattle were affected and involved in green building business in some way (*ibid*). Meanwhile, both the clusters in Seattle and Lower Austria have been established on the basis of traditional industrial foundation with existing local and export demands. There has been an effective combination between new green business and existing factor conditions, demand conditions, supporting enterprises and favourable local business and policy context.

Third, green building cluster improves innovative capacity, creates high value-added sectors,

and generates local fiscal revenues. The development of green building industry largely relies on technological and methodological innovation, and the ever-deepening and more sophisticated market demands further require innovation at various sectors of the industrial chain. Its economic impacts could often be reflected by job creation and wage payment, and local tax incomes. Take Lower Austria as an example, the leading innovative companies and institutions in the fields of energy management, energy technology and green building retrofitting are gathered together to collectively push forward innovation and share the value-added impacts. The value-added impact of green building cluster has been the largest among the five clusters, accumulatively reaching 14.6 million euro and contributing to 70% of the total value-added (Helmut Berrer 2011). It created 434 direct and indirect jobs every year, accounting for 69.6% of all the jobs created by the five clusters. The five clusters in total made substantial tax contribution, including €2.34 million to federal government, €3.4 million to Lower Austria State Government, €7 million to City Council, and €6.15 million to social welfare institutions (*ibid*).

An estimation made by the Seattle government shows that every year economic activities related to green building creates about \$320 million to \$1 billion, creating 1370 to 4160 'green jobs', and pays \$60 million to \$190 million wages (Berk & Hodgins 2005). It also generates about 900,000 to 2.9 million local consumption tax and 650,000 to 2.1 million business and operation tax (*ibid*). Further, increased wages further encourage higher consumption and related taxes too.

Fourth, green building cluster creates new market demands for green building products and services at regional and international level and stimulates the development of new professions. A cluster creates favourable conditions for the emergence and growth of new industries. It has been found that the new business and commercial demands generated at clusters, with policy and regulatory guidance, often provide new vitality and potential for the region (Sternberg 1991). Such new market demands act as a breakthrough to the traditional business, effectively helping solve the problem of market failure and information asymmetry. Research results indicate that green building accreditation, such as LEED first developed in US and BREAM in UK, tends to bring extra economic value to the buildings (World Green Building Council 2013). Hence, there has been an increasing enthusiasm among tenants and builders for having buildings assessed and accredited, so that its potential market value to be formally recognized. This, as a result, provides new opportunities to the profession of green building assessment, training, and researching.

7 CONCLUSION

Green building represents the future of building in terms of better energy efficiency, reduction of greenhouse emission and improvement of human uses. While its significance to sustainable development has been repeatedly addressed by policy-makers, its commercial value is also growingly recognised by market players. This paper discusses the current conditions and development trend of the global green market and indicates that green building has already become a global development trend that brings important business opportunity in the long-term. Therefore, the key question arises as how to release such grand potential and promote large-scale development of green building at an industrial level.

In this regard, this paper further analyses the concept of green building cluster and its practical application in some developed nations. Its contributions have been investigated through three perspectives, i.e. to the cluster members, to the cluster management, and to the industrial development and regional economy. It has been found that green building cluster is way more than geographic agglomeration of green building enterprises. It reduces costs of information and exchange, promote knowledge creation and technology advancement, develop green industry via creation of new products, services and new business relations and networks. Its combination with green building industry conforms the essence of sustainable development, and is closely related to releasing the commercial potential of green building through scaling development.

This, to a large extent, is in line with the major goals of China, which is at the strategic crossroad of growth model change and industrial transformation. The need for improving energy efficiency, challenging goals set up by the government in making buildings 'green', the large-scale of new construction, as well as the unsustainable problems of the current urban development pattern all point to the opportunity for developing green building clusters. It has the potential to promoting low-carbon development and driving economic growth at the same time. However, its applicability to China needs further research on suitable modes of financing and management. Its sustained success relies on an integrated feasibility study at early stage, incorporating multiple factors such as industrial bases, financing tools, policy supports and existing and potential resources at various levels. Studies on the international cases all indicate the necessary combination with local industrial conditions and important foundation of the four main factors of the 'Diamond Model', i.e. demand, factor, context, supporting industries. These by all means need to be taken full account and effectively combined with Chinese local characteristics and institutional environment. Green building cluster could be

a key to opening the green industry in China, yet the way ahead is yet to be explored.

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Promoting low carbon building industry development through PPP procurement system in China

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ABSTRACT: The main driver of Chinese rapid economic growth over last three decades relies on fixed asset investment and infrastructure development. However, the environmental cost of economic development in China is high, which has significantly damaged China's economic achievements. The Chinese Central Government has noticed the challenges and it has responded to fix the problem by increasing investment for environment protections particularly, low carbon building development. Public Private Partnership (PPP) is an effective procurement tool for the government to deliver the provision of public services, which has been adopted worldwide. There is a potential for using PPP to incorporate a sustainability agenda and thus support the low carbon building industry development. This paper aims to identify the business case of using PPP to promote and accelerate the low carbon building industry development and concludes that PPP could provide financial incentives for low carbon technologies development and remove economic barriers for low carbon building projects.

1 INTRODUCTION

China has been one of the world's fastest growing economies (Demurger 2001, Sahoo et al. 2012). The dominant economic growth model in China is investment-led model, while China is always keeping high proportion of urban fixed-asset investments such as infrastructure and real estate which contributed around half GDP growth in 2000s (Ahuja & Nabar 2012). Meanwhile, the public infrastructure development is the main engine of China's investment-led growth. The positive impacts of infrastructure development improve productivity, provide better investment conditions, reduce the unemployment rate and also help China avoid the global financial crisis between 2007 and 2009. As an alternative procurement tool for the public infrastructure provision, Public Private Partnerships (PPPs) brings more private finance into public sectors. Public Private Partnership (PPP) is a procurement mechanism by which the public sector contracts to purchase quality services on a long-term basis so as to take advantage of private sector management skills incentives by having private finance at risk. There is a potential for using PPP to incorporating sustainability agenda and support low carbon economic development. However, little research has been conducted to demonstrate the benefits and advantages of PPP procurement system to support low carbon industry.

In China, the promotion of green building and Low Carbon Building Technologies (LCBT)

are inevitable slow. One of the key barriers is the cost. Property developers' investments in the environmental friendly building technologies are far less than social expectations. The risks & uncertainties of short-term ROI (return on investment) and misalignment between owner costs and tenants benefits stop developers' extra investments; the continuing economic recession and austerity plan shrink public financial supports;

This paper aims to identify the business case of using PPP to promote accelerate low carbon building industry development and concludes that PPP could provide financial incentives for low carbon technologies development and remove economic barriers for low carbon building projects. It initially reviews the development of PPP and the barriers of low carbon building in China. The following section assesses how PPP acts as an alternative procurement tools to support and stimulate the implementations of low carbon technologies in the construction industry and the lesson learns of sustainable PPP projects in the UK. It finally discusses the facing challenges and future prospects of sustainable PPP projects in China.

2 PPP DEVELOPMENT IN CHINA

Globally, PPP has been seen as a world solution to involve greater private sector participation in the development of urban development projects. PPP has been widely adopted by many countries

because of its attractive characteristics to transfer risks to the private sectors, reduce public sectors administration cost, solve the problem of public sector budget restraint, provide higher quality public products and services, and save time in delivering the project (Li et al. 2005).

The concept of PPP is not new in China. They contribute to the national and local infrastructure development by providing investments, advanced technology, and management skills (Adams 2006, Ho 2006, Ke et al. 2009, Yuan et al. 2010, and Wang et al. 2012). In China, the private sector first entered the traditionally government-controlled infrastructure sector in the late 1980s. The first PPP (BOT) project is Shajiao B power plant in Shenzhen, which come to operation in 1998 (Ke et al. 2009).

Up to 2011, there are total 1018 PPP projects in China and the total capital value is US\$ 116.399 bn PPPs mainly divided into four sectors: Energy, Telecommunication, Transport and Water. Transportation is the most popular sector for PPP implementation in both project number and total projects' capital value. Wang (2004) has classified Chinese PPPs into three distinct types—outsourcing, concession and divestiture and summarises that there are 14 different modes of PPP in China. The most popular model is BOT, where the private sectors could directly collect the tariff from end users (Wang et al. 2012).

Recent years, the central government attempts to remove constraints to private sector investment and promotes the development of Public-Private Partnerships (PPPs) in other sectors. There is a huge financial gap in infrastructure investment. China is increasingly turning to PPPs to bridge a huge financing shortfall in infrastructure. On the other hand, the Central Government intends to temper the economic slowdown driven by the ongoing property downturn, but local governments are facing financing difficulties due to sliding tax and land sale revenues and tighter debt controls.

Chinese Central Government has announced a number of PPP guidelines since December 2014. For example, the State Council released an investment and financing guideline emphasizing the importance of PPP in mobilizing private capital. The Chinese Ministry of Finance (MOF) announced 30 projects with a total value of 180 billion Yuan (\$29.3 billion) as model projects to attract private capital in the form of a Public-Private Partnership (PPP). These projects include water and heating supply, sewer systems, waste disposal, underground network, drainage system, healthcare, and other urban infrastructure systems.

The National Development and Reform Commission (NDRC) also developed a pipeline database of 1,043 proposed projects through Public-Private Partnerships (PPP). The investment

needed for these projects are totals 1.97 trillion Yuan (\$322 billion). PPP project operators are encouraged to directly finance from the financial market; pension funds and insurance premiums are also allowed to invest in PPP projects. Moreover, each province has set up their own PPP centre and announce a list of PPP proposed projects and consulting companies/experts to support the development of PPP project in each region. The legal framework of PPP in China is setting up, which will strongly support nationalization of PPP projects in the near future.

There are three key drivers for Chinese public sectors to adopt PPPs. The first driver is due to high rate of urbanization, which causes the high demand for basic public infrastructure and services. As Chen and Doloi (2008) mentioned that China's demand of infrastructure projects is largely driven by urbanisation. Rapid urbanization during China's reform period has resulted in a very high demand for basic urban infrastructure and the need for sustained mechanisms of financing. The second driver is to relieve government's fiscal pressure for infrastructure development (Wang et al. 2012). The third driver is the high saving rate of households and private finance. Chinese households have very high saving rate. The nation saves half of its GDP and its Marginal Propensity to Save (MPS) approached 60% during the 2000s (Ma & Wang 2010). Private capital in China is totally 60 trillion Yuan (US\$9.49 trillion), including 35.2 trillion Yuan (\$5.57 trillion) of individual savings deposit and 25 trillion Yuan (\$3.96 trillion) of capital from private enterprises and other sources are ready for infrastructure investment (Lan 2012). The Government must guide the private capital investment and build a sound and fair financing platform to improve the investment regulations to support private capital in the infrastructure development.

3 LOW CARBON BUILDING

There is no any single definition of Low Carbon Building (LCB) worldwide. Briefly, a LCB is a building, which reduces energy usage and emits significantly less Greenhouse Gas (GHG) than regular buildings. LCB is purposely designed and constructed to achieve a little or zero carbon during their lifetime. Although LCB is more specifically indicated to low carbon emission and better energy efficiency, it is included or interchangeable with the term of 'Green Building' in many countries, such as USA, Canada and China. According to the US Environmental Protection Agency, green building is 'the practice of creating structures and using processes that are environmentally responsible and resource-efficient through a

building's life cycle (UNEP 2010). The difference between LCB and Green Building is that LCB has a clearer indication of carbon emission reduction level, but green buildings normally address more environmental issues: such as land conservation, water, material and indoor air quality, etc. In this paper, LCB will be used in a broad definition, which covers both 'Green Building' and LCB. In most countries, LCB will be evaluated and certified through an assessment tool that validate its green features (UNEP 2010), and there are many assessment tools available worldwide, for example, LEED (Leadership in Energy and Environmental Design) in USA; BREEAM (BRE Environmental Assessment Method) in the UK and Green Building's star-rating system in China.

There are many perceived benefits to adopt low carbon building development concepts, but the key drivers can be simply concluded to two trends: Government Push and Market Pull. Currently, government intervention has been seen as the biggest driver for low carbon building market, through a range of methods: such as setting restricted building regulations, tax, policies and provide financial incentives (UNEP 2010).

On the other hand, the market demand of low carbon buildings is increasing. A recent survey from McGraw Hill (2013) discovered that now green building is a business imperative around the world. Business drivers such as client and market demand are the key factors influencing the market. There are two main reasons for the increasing LCB market demand, the first one is the price drop of renewable energy technologies, for instance, solar photovoltaic technology has shown significant development in recent years, with ongoing technological improvements and capital cost falling.

Although the market signal positively indicates the increasing interests of low carbon buildings, but the uptakes in most developing countries, such as China, is still extremely slow. Green buildings are commonly perceived to be a lot more expensive than conventional buildings and often not worth the extra cost (Kats 2003, UNEP 2010). According to Statistics (CBRE 2014), it need increase 161 Yuan/m² for public building and 121 yuan for residential building to achieve three star green building standard in China (Fig. 1). These extra costs mainly include outdoor pavement, enclosure structure optimization, renewable energy technologies (such as PV, wind power, air source heating, and geothermal heating systems), thermal storage technology, rainwater harvest system, indoor environmental control and intelligent building system. If, in accordance with the National Bureau of statistics released average new-built house average price 2,643 Yuan/m² in 2013, Green Building increase 0.8%~6.1%. However there are some

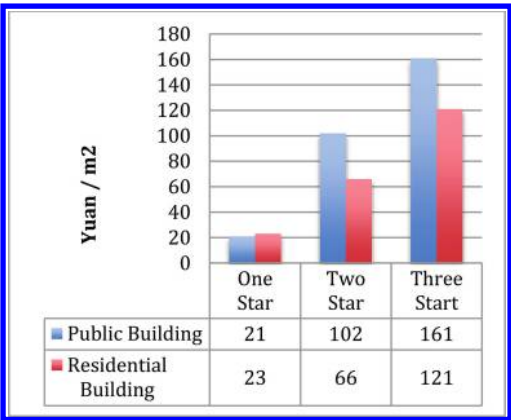


Figure 1. Extra costs of Green Building in China.

hidden costs for developers such as landscaping, green roof and other maintenance costs, etc., which the total extra cost of three-star level may rise up to 300 Yuan/m². It is too high for developers to pay.

Another big barrier is that the incentive for building owners and building tenants to improve energy efficiency, make green improvements and seek or maintain third party certification are often misaligned (UNEP 2010). In China there is still little motivation for developers and house owners to take action on low carbon building investment and less engagement from low carbon products provider on building operation. The main reasons are that developers cannot see direct benefits from extra cost on low carbon buildings, and low carbon facilities providers cannot easily access property market under the conventional development model. It is clear that the real estate sector was slow uptake LCBs, regardless of other driving forcing from the government and social sector, the investment problem remains as one of the biggest barriers.

4 PPP AND LOW CARBON BUILDING INDUSTRY

PPP has been used to modernize the public services and achieve the best value of public spending in the UK since 1992. The public sector contracts to purchase quality services on a long-term basis so as to take advantage of private sector management skills and by having private finance at risk. The government has used PPP to achieve value for money and public service excellence and it has been arguably being used to lead a global public procurement revolution. PPP can offer real scope to promote sustainable construction (Addis &

Talbort 2001). Hill and Collins (2004) found that the PPP mechanism can be used as a lever to move the construction sector towards greater energy efficiency of its products and practices. It incorporates whole-life costing, as opposed to lowest initial price and should encourage a more sustainable approach. The transfer of risks such as energy consumption to the private sector may provide an incentive for investment in more efficient energy usage. The critical factor is to ensure that sustainability gains are assessed against value not cost. PFI clients generally specify outputs rather than input. Clients can use this opportunity to specify a required low carbon performance (e.g. energy usage per year) rather than specifying the use of low energy equipment or facades. It is then the contractor's responsibility to find the most cost-effective way of delivering the performance level demand. Furthermore, the long-term and integrated nature of PPP services has incentivized the contractors to consider the synergies between the design of an asset and its Ultimate Operating Cost (OGC 2002).

As China is still in early stage of the promotion of PPP projects, it can gain benefits to learn lessons from other mature market, such as the UK, which has integrated sustainability objectives into the PPP procurement system successfully over 900 projects (Zhou et al. 2013, Zhou & Smith 2013).

Case study One – St Barts and London Hospital

St Barts and the London Hospital PFI Scheme is the Britain's largest hospital scheme and one of the top ten PFI projects in the UK. It combined two large and old hospitals to become one massive redevelopment project including transforming one old hospital into a Cancer and Cardiac Centre of Excellence. This project is worth £1.1 billion. The facilities will provide, benefit to the people of Tower Hamlets, the City of London and the wider health economy, the space for 1,248 beds in a modern purpose-built environment that is suitable for the provision of healthcare in the 21st century.

Because this project is the largest hospital scheme in the UK, it will be a flagship of sustainable building and will be the best example in the PFI industry and the Department of Health. Its lessons could be learned and provide a good resource or source of reference for other PFI projects. Sustainable regeneration has been integrated at a very early stage and throughout the whole life of the project, from the planning stage to the construction and operation, and a specific regeneration manager has been employed who will provide more expert experience to help the PFI process in going green.

During the negotiation stage, the contractor proposed a sustainable development strategy programme with its client to build up common understanding of sustainable development under this project context. Furthermore, in order to maximize the contribution of the PFI scheme to regeneration, the Trust (2006) developed a sustainable regeneration strategy focused on enhancing employment opportunities for local people. The Trust also looked at how it can use the project to embed sustainability in the way it manages its day-to-day operations. The aim of this strategy is to maximize the return on the investment in the new hospital in terms of local regeneration benefits and sustainability outcomes. This will link the investment to positive community health outcomes and contribute to the Trust's corporate citizenship agenda.

The PFI consortium had set sustainability targets during the construction stage

- Achieving a rating of 'excellent' for all new buildings, using the NHS Environmental Assessment Tool (NEAT)
- Recycling 65% of their waste, and
- Sourcing 20% of their energy from renewable sources.

In this project there are two unique characteristics to show how stakeholders have involvement in the PFI sustainable development. The first one is that the client has employed an environmental manager whose responsibility is to develop the sustainable PFI development strategy. Furthermore the contractor's environmental team will support to provide more sustainable technical solutions during the construction phase, for instance to use acoustic screens to reduce the noise on site and reusable packaging to reduce the waste on site.

Case Study Two: Newport Southern Distributor Road

Newport is the third largest city in Wales, a traditional small city on the banks of the River Usk. Following the UK central government's sustainable development strategy, Newport City Council created its own 21 agenda action plan and devised a new sustainable urban regeneration strategy for its residents for next decade (NCBC 2000).

In order to relieve transport congestion and improve the local transport network, the Newport County Borough Council (NCBC) decided to adopt the PFI model to develop a new road on the south side of the city. The proposed plan is to build a dual carriageway link from Junction 24 of the M4 (Coldra roundabout) to the east Junction 28 (Tredegar roundabout) on the southern edge

of Newport, which includes a major crossing of the River Usk. Morgan Est, one of the largest civil engineering contractors in the UK, together with Vinci, an international contracting company, formed a joint venture 'Morgan Vinci Ltd' which won the £55 million contract. This comprises Design, Build, Finance and Operation (DBFO) of the project over a 40-year life span. This project is the biggest local authority PFI project in Wales and is also recognized as a pathfinder project by the Welsh Assembly Government.

At the project level, approximately £1 million initial cost was saved through the use of around 450,000 tons of recycled and secondary aggregates instead of purchasing primary materials. A variety of secondary aggregates is used as granular fill materials and unbound sub-base. In the recycling programme, specific cost savings included:

- The avoidance of waste disposal charges and Landfill Tax
- The avoidance of Aggregates levy payments, from which recycled and secondary aggregates are exempt
- The lower costs and improved performance of maintenance techniques such as 'crack and seat', Rhinopatch and cold-lay foamed asphalt mix
- Reduced costs of transporting aggregates when recovered materials are available locally.

At the local level the Newport SDR helps to release the congestion of Newport Transport Network and reduces traffic in the city centre. These outcomes provide massive benefits to promote new business investment in Newport.

At the national level, this project is the first pathfinder PFI project in Wales. It constitutes a successful model for future PFI development, reducing government borrowing and increasing investment in public infrastructure. It has observed the green standards by using recycled aggregates and winning a number of national and international awards. For instance, the PFI project has been selected by the Department of Trade and Industry as a flagship case study. Newport recently won a Green Apple award—presented for environmental best practice around the world. In addition to its economic success, the project therefore achieved a high standard of social and environmental performance. It achieved all proposed project objectives and reduced the traffic in the city centre. Other social benefits include providing a safe road for its end users and proper cycle and walk paths for local residents and school students. More interestingly, after comparing the capital cost saving, whole life cost saving and predicted new business investment, the capital cost saving (£1m) matches the long-term saving (£1m), but the social-economic value (£1bn) appears at the

highest level. The total project cost saving becomes relatively tiny at the local outcome level.

5 CURRENT CHALLENGES AND FUTURE TRENDS

- a. **Lack of a support system:** Most PPP projects in China are managed at provincial or municipal government by following sector's departmental guide. However, Wang et al. (2010) argued that there is no organisation at national level in China specifically responsible for PPP projects, such as Partnerships UK or the National Council for Public Private Partnerships in the US. The establishment of the specialized agencies for PPPs is urgently needed to prepare the national PPP Guide and assist the PPP procurement process. In those guides and standards, sustainability and low carbon development should be integrated as a basic principle and recognized by the governing bodies.
- b. **Unbalanced partnership:** In China, PPPs have been seen as a 'quick fixed' tool to reduce the government fiscal pressure, rather than consider it as an effective tool to improve the productivity. Nearly 30 years practice, the government is still not fully recognized the private capital efficiency and better management skills in infrastructure development, and does not believe that private enterprise will shake the dominant position of the centrally planned economy in the supply of public goods. Private sector is more focusing on short-term return without a spirit of long-term partnership (Ho 2006). It is also importance for private sector to recognize the direct and indirect benefits from low carbon building design and technologies and could take advantages of using them to make profit within PPP projects rather than considering the innovative process or products may obtain potential risks.
- c. **Lack of experience and inadequate knowledge:** Evidences found that the private sectors in China are lack of experience on the commercial, technical, legal and political aspects of PPPs (Ho 2006, Wang et al. 2012). As they are not yet familiar with PPP process, so it might delay the implementation of low carbon technologies in PPP projects.

Although PPPs development in China is facing lots of challenges and but the third waving is coming. As the aforesaid three main drivers is increasing strong in China, the private sectors has over 60 trillion Yuan are waiting to invest. The urbanization requests high demand of urban infrastructure and utilities, the government will

evolve deep fiscal and financial reform to welcome more private sectors entering the dynamic emerging market.

6 CONCLUSION

China is the fastest growing country in the world for last three decades and accounts for nearly one fifth of the world population. However, the conventional budgetary allocation from central or local government is unsustainable and facing fiscal decentralization. Public Private Partnership becomes one of the important investment tools to finance public infrastructures. While PPPs are not new in China, there is still a great potential for their application due to large amount of private capital, the high demand of urbanisation and in-depth fiscal system reform. Implementing low carbon design and technologies into PPPs in China are crucial, as China has alternative high environmental costs, which need to be paid by this generation. Lessons learned from British projects demonstrate that using PPP business model could encourage clients to consider the long-term benefits through the whole life costing calculation. The 30-year or longer contract could guarantee the best value for money and good performance of the development. PPP could provide financial incentives for low carbon technologies development and remove economic barriers for low carbon building projects. It is essential to make sure that both public and private sectors understand the social and economic benefits of low carbon building, and agree to use more innovative products and technologies.

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