Reinforced Masonry Engineering

Handbook

Clay and Concrete Masonry

6th Edition

Masonry Institute of America

ICC International Code Council®

Reinforced Masonry Engineering Handbook

Clay and Concrete Masonry

Sixth Edition

James E. Amrhein, S.E.
Consulting Structural Engineer
Original Author

Max L. Porter, P.E., Ph.D.
Iowa State University

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PREFACE

In 1970, James Amrhein recognized that a comprehensive reinforced engineering design handbook was needed which would encompass the coefficients, tables, charts and design data required for the design of reinforced masonry structures. Mr. Amrhein tried to fulfill these requirements with the first edition of this publication. Since then, subsequent editions have been improved and expanded to comply with applicable editions of the Uniform Building Code and International Building Code keeping pace with the growth of reinforced masonry engineering.

The authors would like this book to be as useful as possible to designers of reinforced masonry in eliminating repetitious and routine calculations. This publication will increase the understanding and reduce the time required for masonry design.

The detail and design requirements included in this book are based upon the 2006 edition of the International Building Code published by the International Code Council, and ASCE/SEI 7-05, Minimum Loads for Buildings and Other Structures published by the American Society of Civil Engineers. Also included in this edition is information and design tables based on the code reference document, ACI 530/ASCE 5/TMS 402 Building Code Requirements for Masonry Structures.

In addition to the code requirements, sound engineering practice has been included in this publication to serve as a guide to the engineer and designer using it.

There may be several design and analysis methods and the results for the design can be somewhat different. Techniques included in this publication have been reviewed by competent engineers who have found the results to be satisfactory and safe. The authors welcome recommendations for the extension and improvement of the material and any new design techniques for future editions.

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AUTHORS

James E. Amrhein

James E. Amrhein, who served as Executive Director of the Masonry Institute of America until his retirement, has more than 50 years experience in construction, engineering, technical promotion, teaching, structural design and earthquake engineering. He was a project engineer with Stone & Webster Engineering Corporation in Boston, Massachusetts, Supervising Structural Engineer for the Portland Cement Association in Los Angeles, and has been active in seismic design and research, including the investigation and reporting of structural performance of buildings subjected to earthquakes throughout the world. His B.C.E. was earned at Manhattan College followed by an M.S.C.E. from Columbia University in New York City. He was elected to the Tau Beta Pi and Chi Epsilon honorary engineering societies.

In 1983, Mr. Amrhein received the Outstanding Engineering Merit Award from the Institute for the Advancement of Engineering and the Steven B. Barnes Award from the Structural Engineers Association of Southern California for his contributions in the field of masonry research and education. He also received the Distinguished Service Award from the Western States Clay Products Association. His research, along with other members of SEAOSC, eliminated the h/t limitations from the code and introduced strength design provisions for masonry tall slender walls.

Mr. Amrhein is a Registered Civil, Structural and Quality Engineer in California and a Licensed Professional Engineer in New York. He is a Fellow in the American Society of Civil Engineers and the American Concrete Institute, and an Honorary Member of The Masonry Society and the Structural Engineers Association of Southern California. He is also a Fellow in the SEAOC College of Fellows and a member of numerous other professional organizations including the International Code Council and the Earthquake Engineering Research Institute. He is a founding member and past president of The Masonry Society.

Mr. Amrhein is a Navy veteran who served overseas in World War II and the Korean incident with the Seabees. From 1961 to 1980 he served on the evening Civil Engineering faculty at California State University, Long Beach, as an adjunct (full) professor. He has presented masonry design seminars for the American Society of Civil Engineers in their continuing education program and has lectured at many universities throughout the United States and around the world. He has written many technical publications on masonry and concrete.

Mr. Amrhein continues to work as a consultant on masonry and concrete issues. He was married to his wife, Laurette, for 56 years. They have four children (three engineers and one scientist) and seven grandchildren.

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Max L. Porter

Max L. Porter is a professor of civil engineering at Iowa State University, Ames, Iowa. He has served multiple officer positions of several organizations and president of five organizations, including President of The Masonry Society and the Structural Engineering Institute. He has chaired several national code committees in the areas of masonry, reinforced concrete, and FRP. He has published over 400 papers, books or chapters, and reports and given over 350 technical presentations. He has taught over 30 different courses with most of the courses being in the areas of reinforced concrete, masonry, timber, and structural engineering. He has received many honors, and received the award as Distinguished (Honorary) Member of ASCE (the organization's highest honor) and the Scalzi Research Prize President's Award from TMS.

He has and continues to serve on the Masonry Standards Joint Committee (MSJC) since its inception, including six years chairing the Committee. He is also active with ASCE and American Concrete Institute. He has taught several of the national design and code seminars or workshops on masonry design, since the inception of the MSJC Code in 1977. He has also contributed a large number of technical presentations and papers on various masonry topics.

Dr. Porter attended Iowa State University where he received his Bachelor Degree in 1965, Masters Degree in 1968 and Ph.D. in 1974. As a young engineer, his experience includes employment with the County of Los Angeles, Iowa State Highway Commission and the American Bridge Division of the U.S. Steel Corporation. Previously, Dr. Porter has served as a professional consultant for over 30 firms and has performed disaster investigations on a regular basis, as well as serving as a consultant for over 200 clients dealing with failed masonry structures over a 42-year period.

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The Masonry Institute of America

The Masonry Institute of America, founded in 1957 under the name of Masonry Research, is a promotional and technical research organization established to improve and extend the use of masonry. The Masonry Institute of America is supported by the California mason contractors through labor management contracts between the unions and contractors.

The Masonry Institute of America is active in California promoting new ideas and masonry work, improving national and local building codes, conducting research projects, presenting design, construction and inspection seminars and publishing technical and non-technical papers, all for the purpose of improving the masonry industry.

The Masonry Institute of America does not engage in the practice of architectural or engineering design or construction nor does it sell masonry materials.

International Code Council®

Since the early 1900's, the United States had been served by three sets of building codes developed by three separate model code groups: Building Officials and Code Administrators International, Inc. (BOCA), International Conference of Building Officials (ICBO), and Southern Building Code Congress International, Inc. (SBCCI). These codes were extremely effective and responsive to regional needs. But, in 1994, recognizing the urgent need for a single set of codes that would serve national needs, the three groups united to form the International Code Council® (ICC®) with the express purpose of creating and developing one master set of comprehensive, coordinated, design and construction codes.

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- Publication of proposed code changes
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MASONRY STANDARDS JOINT COMMITTEE

The Masonry Standards Joint Committee (MSJC) is an organization comprised of volunteers who through background, use, and education have established experience in the manufacturing of masonry units and materials and the design and construction of masonry structures.

Working under its three sponsoring organizations, The Masonry Society (TMS), the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE) the Committee has been charged with developing and maintaining consensus standards suitable for adoption into model building codes. Since The Masonry Society has received ANSI accreditation, TMS has become the lead sponsor in the production of the MSJC Code and Specification.

In the pursuit of its goals, committee activities include:

1. Evaluate and ballot proposed changes to existing standards of the Committee.
2. Develop and ballot new standards for masonry.
3. Resolve negative votes from ballot items.
4. Identify areas of needed research.
5. Monitor international standards.

In this publication the term ‘MSJC Code’ refers to Building Code Requirements for Masonry Structures (ACI 530/ASCE 5/TMS 402) and the term ‘MSJC Specification’ refers to Specification for Masonry Structures (ACI 530.1/ASCE 6/TMS 602).

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THE MASONRY SOCIETY

The Masonry Society (TMS) founded in 1977, is an international gathering of people interested in masonry. It is a professional, technical, and educational association dedicated to the advancement of knowledge of masonry. TMS members are design engineers, architects, builders, researchers, educators, building officials, material suppliers, manufacturers, and others who want to contribute to and benefit from the global pool of knowledge on masonry.

AMERICAN CONCRETE INSTITUTE

The American Concrete Institute (ACI) is a technical and educational society founded in 1904 with 30,000 members and 93 chapters in 30 countries.

As ACI moves into its second century of progress through knowledge, it has retained the same basic mission: develop, share, and disseminate the knowledge and information needed to utilized concrete to its fullest potential.

AMERICAN SOCIETY OF CIVIL ENGINEERS

The American Society of Civil Engineers (ASCE) was founded in 1852 and currently represents 125,000 members of the civil engineering profession worldwide. ASCE's vision is to position engineers as industry leaders building a better quality of life.

To provide essential value to members, their careers, partners and the public, ASCE develops leadership, advances technology, advocates lifelong learning, and promotes the profession.

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SYMBOLS AND NOTATIONS

a = depth of an equivalent compression zone at nominal strength, in.

a_b = depth of stress block of member for strength design.

a_u = ϕf_y (1 – 0.59q). Coefficient for computing steel area A_s.

A = area of floor or roof supported by a member.

= cross sectional area of a member.

A_1 = bearing area, in.^{2}.

A_2 = effective bearing area, in.^{2}.

A_b = cross-sectional area of an anchor bolt, in.^{2}.

A_e = effective area of masonry, in.^{2}.

A_f = area of flange of intersecting wall.

A_g = gross cross-sectional area of masonry, in.^{2}.

A_{jh} = total area of special horizontal shear reinforcement in a masonry frame equal to 0.5 V_{jh}/f_{yh}.

A_{mv} = net area of masonry section bounded by wall thickness and length of section in the direction of shear force considered, in.^{2}.

A_n = net cross-sectional area of masonry, in.^{2}.

A_p = projected area on the masonry surface of a right circular cone for anchor bolt allowable shear and tension calculations, in.^{2}.

A_{ps} = area of prestressing steel, in.^{2}.

A_{pt} = projected area on masonry surface of a right circular cone for calculating tensile breakout capacity of anchor bolts, in.^{2}.

A_{pv} = projected area on masonry surface of one-half of a right circular cone for calculating shear breakout capacity of anchor bolts, in.^{2}.

A_s = effective cross-sectional area of reinforcement, in.^{2}.

A'_s = effective cross-sectional area of compression reinforcement in a flexural member, in.^{2}.

A_{se} = effective area of steel for slender wall design, in.^{2}.

A_{st} = total area of laterally tied longitudinal reinforcing steel in a reinforced masonry column or pilaster, in.^{2}.

A_{tr} = total cross-sectional area of transverse reinforcement (stirrup or tie) within a spacing s and perpendicular to plane of bars being spliced or developed, in.^{2}.

A_v = cross-sectional area of shear reinforcement, in.^{2}.

A_x = the torsional amplification factor at Level x.

ACI = American Concrete Institute.

ANSI = American National Standards Institute.

ASCE = American Society of Civil Engineers.

ASD = Allowable Stress Design.

ASTM = American Society for Testing and Materials.

avg. = average.

b = effective width of rectangular member or width of flange for T and I sections, in.

= column dimension, in.

b' = width of web in T and I members.

b_a = total applied design axial force on an anchor bolt, lb.

b_{af} = factored axial force in an anchor bolt, in.

b_t = computed tension force on anchor bolts, lb.

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b_v = total applied design shear force on an anchor bolt, in.

b_{vf} = factored shear force in an anchor bolt, lb.

b_w = width of wall beam, in.

B_a = allowable axial force on an anchor bolt, lb.

B_{an} = nominal axial strength of an anchor bolt, lb.

B_t = allowable tension force on anchor bolts, lb.

B_v = allowable shear force on an anchor bolt, lb.

B_{vn} = nominal shear strength of an anchor bolt lb.

BTU = British Thermal Units.

c = distance from the fiber of maximum compressive strain to the neutral axis, in.

= coefficient that determines the distance to the neutral axis in a beam in strength design.

= total compression force, lb.

= numerical coefficient.

cm = Centimetre.

cu. = cubic.

C_d = deflection amplification factor.

C_e = combined height, exposure and gust factor.

= snow exposure factor.

C_f = compression on the flange.

Ch. = Chapter.

C_n = nominal bearing strength, lb.

C_p = numerical coefficient.

C_q = pressure coefficient for the structure or portion of the structure under consideration.

C_s = slope reduction factor.

C_t = numerical coefficient.

C_w = compression on the web.

CM = center of mass.

CMU = Concrete Masonry Unit.

CR = center of rigidity.

Comp. = compressive.

d = distance from extreme compression fiber to centroid of tension reinforcement, in.

d_b = diameter of reinforcement, in.

d_{dd} = diameter of largest beam longitudinal reinforcing bar passing through or anchored in the joint, in.

d_{bp} = diameter of largest pier longitudinal reinforcing bar passing through the joint, in.

d_1 *or* d' = distance from compression face of a flexural member to the centroid of longitudinal compressive reinforcement.

d_v = actual depth of masonry in direction of shear considered, in.

d_x = distance in x direction from center of rigidity to shear wall.

d_y = distance in *y* direction from center of rigidity to shear wall.

D = dead load or related internal moments and forces.

= nominal diameter of reinforcing bar, in.

= dimension of a building in direction parallel to the applied force.

D_i = inside diameter, in.

D_o = outside diameter, in.

D_s = the plan dimension of the building of the vertical lateral force resisting system.

DL = dead load.

e = eccentricity of axial load, in.

= eccentricity measured from the vertical axis of a section to the load.

e' = eccentricity measured from tensile steel axis to the load.

e_b = projected leg extension of bent-bar anchor, measured from inside edge of anchor at bend to farthest point of anchor in the plane of the hook, in.

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e_k = eccentricity to kern point.

e_m = strain in masonry.

e_{mu} = maximum useable compressive strain of masonry.

e_s = strain in steel.

e_x = eccentricity in *x* direction of center of mass to center of rigidity.

e_y = eccentricity in y direction of center of mass to center of rigidity.

e_u = eccentricity of P_{uf}, in.

E = load effects of earthquake or related internal moments and forces.

E' = eccentricity measured from tensile steel axis to the load, ft.

E_{AAC} = modulus of elasticity of AAC masonry in compression, psi.

E_c = modulus of elasticity of concrete in compression, 33 $w^{1.5}f'_c$ psi.

E_g = modulus of elasticity of grout in compression.

E_m = modulus of elasticity of masonry in compression, psi.

E_q = equation.

E_s = modulus of elasticity of steel = 29,000,000, psi.

E_v = modulus of rigidity (shear modulus) of masonry, psi.

E.F.P. = equivalent fluid pressure of lateral earth loads.

EST = Equivalent Solid Thickness.

f_a = calculated compressive stress in masonry due to axial load only, psi.

f'_{AAC} = specified compressive strength of AAC, the minimum compressive strength for a class of AAC as specified in ASTM C1386, psi.

f_b = calculated compressive stress in masonry due to flexure only, psi.

f_c = concrete compressive stress in extreme fiber in flexure, psi.

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi.

f'_c = specified compressive strength of grout, psi.

f_g = compressive strength of grout, psi.

f'_g = specified compressive strength of grout, psi.

f_m = actual compressive masonry stress from combined flexural and axial loading, $f_m = f_a + f_b$, psi.

f'_m = specified compressive strength of masonry at age 28 days, psi.

f_{md} = computed compressive strength in masonry due to dead load only.

f'_{mi} = specified compressive strength of masonry at the time of prestress transfer, psi.

f'_{mu} = ultimate compressive strength of the masonry, psi.

f_{ps} = stress in prestressing tendon at nominal strength, psi.

f_{pu} = specified tensile strength of prestressing tendon, psi.

f_{py} = specified yield strength of prestressing tendon, psi.

f_r = modulus of rupture, psi.

f_{rAAC} = modulus of rupture of AAC, psi.

f_s = calculated tensile or compressive stress in reinforcement, psi.

f'_s = stress in compressive reinforcement in flexural members, psi.

f_{sb} = soil bearing pressure, psf.

f_{se} = effective stress in prestressing tendon after all prestress losses have occurred, psi.

f_t = flexural tensile stress in masonry, psi.

f_{tAAC} = splitting tensile strength of AAC as determined in accordance with ASTM C1006, psi.

ft = feet.

ft kips = foot kips, moment.

ft lbs = foot pounds, moment.

f_v = calculated shear stress in masonry, psi.

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f_y = specified yield strength of steel for reinforcement and anchors, psi.

f_{yh} = tensile yield stress of horizontal reinforcement, psi.

F = lateral pressure of liquids or related internal moments and forces.

= dimensional coefficient equal to M/K or $bd^2/1200$ and used in the determination of resisting moment of masonry section.

F_a = allowable compressive stress due to axial load only, psi.

F_b = allowable compressive stress due to flexure only, psi.

F_{br} = allowable bearing stress, psi.

F_i, F_n, F_x = lateral force applied to level i, n or x respectively.

F_p = lateral forces on the part of the structure.

F_s = allowable tensile or compressive stress in reinforcement, psi.

F_{sc} = allowable compressive stress in column reinforcement, psi.

F_{su} = ultimate tensile stress of steel, psi.

F_t = that portion of the base shear, V, considered concentrated at the top of the structure in addition of F_n.

= allowable flexural tensile stress in masonry.

F_v = allowable shear stress in masonry, psi.

F.R. = frictional sliding resistance.

FST = face shell thickness of hollow masonry units, in.

g = acceleration due to gravity.

= gram.

gal = gallons.

G = shear modulus (modulus of rigidity) of the masonry, $0.4E_m$, psi.

h = effective height of column, wall, or pilaster, in.

= hour.

h' = effective height or length of column or wall, ft, in.

h_b = beam depth in a masonry frame equal to $1800d_{bp}/f'^{0.5}_g$.

h'_c = pier depth in the plane of the frame, in.

= beam depth, in.

h_i, h_n, h_x = height in feet above the base to Level i, n or x respectively.

h_p = pier depth in a masonry frame equal to $4800d_{bb}/f'^{0.5}_g$.

h_w = height of entire wall or of the segment of wall considered, in.

H = lateral pressure of soil or related internal moments and forces.

= height of block or brick using specified dimensions, in.

Hz = Hertz, cycles per second.

i = interval.

i.e. = for example.

in. = inches.

in. lbs = inch pounds, moment.

I = moment of inertia about the neutral axis of the cross-sectional area, in^4.

= importance factor.

= impact loads or related internal moments and forces.

I_{er} = moment of inertia of cracked cross-sectional area of a member, in^4.

I_{eff} = effective moment of inertia, in^4.

I_g = moment of inertia of gross cross-sectional area of a member, in^4.

I_n = moment of inertia of net cross-sectional area of a member, in^4.

IBC = International Building Code.

ICC = International Code Council.

IRA = Initial Rate of Absorption.

j = ratio of distance between centroid of flexural compressive forces and centroid of tensile forces to depth, d.

jd = moment arm.

j_w = moment arm coefficient for web.

k = the ratio of depth of the compressive stress in a flexural member to the depth.

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= kip, 1000 pounds.

= kilo, 1000.

k_c = coefficient of creep of masonry, per psi.

k_e = coefficient of irreversible moisture expansion of clay masonry.

k_h = coefficients for lateral earth pressure of backfill against a cantilever retaining wall.

kg = kilogram.

kip = 1000 pounds.

km = kilometers.

k_m = coefficient of shrinkage of concrete masonry.

kN = kilonewtons.

kPa = kilopascals.

k_v = coefficient for vertical earth pressure of backfill against a cantilever retaining wall.

k_t = coefficient of thermal expansion of masonry per degree Fahrenheit.

K = the lesser of the masonry cover, clear spacing between adjacent reinforcement, or five times d_b, in.

= $^1/_2\ f_b jk$ for flexural computations, psi.

= $f_s pj$ for flexural computations, psi.

K_a = active (Rankine) earth pressure coefficient.

K_{AAC} = the least of the grout cover, the clear spacing between adjacent reinforcement, or 5 times d_b, in.

K_b = flexural coefficient for balanced design conditions.

K_{hr} = coefficient for lateral earth pressure of backfill against a retaining wall supported at top.

K_p = passive earth pressure coefficient.

K_u = flexural coefficient for strength design equal to M_u/bd^2.

l = clear span between supports, in.

l' = length of the compression area.

l, L = length of the wall or segment, feet, inches.

= lap splice length.

= embedment length.

= length of the block or brick using specified dimensions as defined in IBC Chapter 21.

l_b = effective embedment length of plate, headed or bent anchor bolts, in.

l_{be} = anchor bolt edge distance, measured in the direction of load, from edge of masonry to center of the cross section of anchor bolt, in.

lbs = pounds.

l_d = required development length or lap length of reinforcement, in.

l_{db} = basic development length, inches.

l_{de} = embedment length of reinforcement, in.

l_e = equivalent embedment length provided by standard hooks measured from the start of the hook (point of tangency), in.

l_p = clear span of the prestressed member in the direction of the prestressing tendon, in.

l_w = length of entire wall or of the segment of wall considered in direction of shear force, in.

L = live load or related internal moments and forces.

LL = live load.

L_s = distance between supports, in.

L_w = length of wall, in.

level i = level of structure referred to by the subscript i. "i = 1" designates the first level above the base.

level n = that level which is uppermost in the main portion of the structure.

level x = that level which is under design consideration. "x = 1" designates the first level above the base.

lin. = linear.

m = metre.

= milli, one thousandth, 0.001.

max. = maximum.

min. = minimum.

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mm = millimetre.

mph = miles per hour.

M = maximum moment at the section under consideration, in.-lb.

= design moment.

= mass of structure.

= mega, 1,000,000.

M_a = maximum moment in member due to the applied loading for which deflection is computed, in.-lb.

M_B = overturning moment at the base of the building or structure.

M_c = moment capacity of compression steel in a flexural member about the centroid of the tensile force.

M_{cr} = nominal cracking moment strength, in.-lb.

M_m = the moment of the compressive force in the masonry about the centroid of the tensile force in the reinforcement.

M_n = nominal moment strength, in.-lb.

M_{OT} = overturning moment.

MPa = Megapascals.

M_R = resisting moment.

M_s = the moment of the tensile force in the reinforcement about the centroid of the compressive force in masonry.

M_{ser} = service moment at midheight of a member, including P-delta effects, in.-lb.

M_T = torsional moment.

M_u = factored moment, in.-lb.

M_x = the overturning moment at level x.

MG = Megagram.

M.M. = Modified Mercali Intensity Scale.

MSJC = Masonry Standards Joint Committee (Also refers to ACI 530/ASCE 5/TMS 402 or ACI 530.1/ASCE 6/TMS 602 Code).

n = ratio of modulus of elasticity of steel (E_s) to that of masonry (E_m) or concrete (E_c). For masonry the modular ratio, n is equal to E_s/E_m.

N = Newton, force.

= North.

= number of bars in a layer being spliced or developed at a critical section.

No. = number.

N_u = factored compressive force acting normal to shear surface that is associated with the V_u loading combination case under consideration.

N_v = compressive force acting normal to shear surface, lb.

NA = neutral axis.

o.c. = on center.

OTM = overturning moment.

p = ratio of the area of flexural tensile reinforcement, A_s, to the area (bd).

p' = ratio of area of compressive reinforcement to the effective area of masonry (bd).

p_b = reinforcement ratio producing balanced design conditions.

pcf = pounds per cubic foot, unit weight.

p_g = ratio of the area of vertical reinforcement to the gross area, A_g.

plf = pounds per linear foot.

p_n = ratio of the area of shear reinforcement to masonry area, A_{mv}.

= ratio of distributed shear reinforcement on a plane perpendicular to plane or A_{mv}.

psf = pounds per square foot.

psi = pounds per square inch.

P = axial load, lb.

= design wind pressure, pounds per square foot.

P_a = allowable compressive force at time in reinforced masonry due to axial load, lb.

= force from the active soil pressure.

Pa = Pascals.

P_b = nominal balanced design axial strength.

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P_{br} = bearing load.

P_e = Euler buckling load, lb.

P_f = minimum roof snow load, pounds per square foot.

= load from tributary floor or roof area.

P_g = basic ground snow load, pounds per square foot.

P_m = compressive capacity of the masonry only in a tied column, pounds.

P_n = nominal axial strength, lb.

P_o = nominal axial load strength without bending, pounds.

P_p = passive soil pressure.

P_{ps} = prestressing tendon force at time and location relevant for design, lb.

P_s = compressive capacity of the reinforcing steel only in a tied masonry column, pounds.

P_u = factored axial load, lb.

P_{uf} = factored weight of wall area tributary floor or roof areas, lb.

P_{uw} = factored weight of wall area tributary to wall section under consideration, lb.

P_w = weight of wall tributary to section under consideration, lb.

q = ratio coefficient for strength design = $p(f_y/f'_m)$.

q_s = surcharge load.

= wind stagnation pressure, psf.

= wind stagnation pressure at the standard height of 33 feet as set forth in Table 3.11.

Q = first moment about the neutral axis of a section of that portion of the cross section lying between the neutral axis and extreme fiber, in^3.

Q_E = the effect of the horizontal seismic (earthquake-induced) forces.

r = radius of gyration, in.

r_b = ratio of the area of bars cut off to the total area of bars at the section.

R = seismic response modification factor.

= h'/t reduction factor for walls and columns.

= reduction in percent.

= support reaction, pounds, kips.

= the resultant force from the weight of soil and the frictional resistance.

R_C = coefficient or rigidity for cantilever piers or walls.

R_{cx} = rigidity of cantilever wall in x direction.

R_{cy} = rigidity of cantilever wall in y direction.

R_F = coefficient of rigidity for fixed piers or walls.

R_s = snow load in pounds per square foot per degree of pitch over 20 degrees.

R_x = rigidity of wall in x direction.

R_y = rigidity of wall in y direction.

s = spacing of reinforcement, in.

= spacing of stirrups or bent bars in the direction parallel to that of the main reinforcement.

= section modulus, in^3.

= total snow load, pounds per square foot.

s_l = total linear drying shrinkage of concrete masonry units determined in accordance with ASTM C426.

sq in. = square inches.

sq ft = square feet.

S = snow load, psf.

= site coefficient, soils characteristics and site geology.

= South.

S_a = acceleration spectra.

S_n = section modulus of the net cross-sectional area of a member, in^3.

SD = strength design.

SI = International Systems of Measurements as adopted by the General Conference of Weights and Measures.

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STC = sound transmission coefficient.

t = specified wall thickness dimension or the least lateral dimension of a column, inches.

t' = effective thickness of a wythe, wall or column, inches.

t_p = least actual lateral dimension of a prism.

T = forces and moments caused by restrain of temperature, shrinkage, and creep strains or differential movements.

= tension force, pounds.

= fundamental period of vibration, in seconds, of the structure in the direction under consideration.

T_E = equivalent thickness, inches.

T_{eq} = equivalent tension force.

TL = total load.

TMS = The Masonry Society

u = bond stress per unit of surface area of bar.

U = required strength to resist factored loads, or related internal moments and forces.

UBC = Uniform Building Code.

v = shear stress, psi.

v' = shear stress taken by shear reinforcement, psi.

v_c = allowable shear stress for concrete, psi.

v_m = allowable shear stress for masonry, psi.

V = shear force, lb.

= the total design lateral load or shear at the base.

= basic wind speed, miles per hour.

V_{AAC} = shear strength provided by AAC masonry lb.

V_c = nominal shear strength provided by the masonry.

V_{jh} = total horizontal joint shear in a masonry frame.

V_{jv} = vertical force acting on joint core.

V_m = shear strength provided by masonry, lb.

V_n = nominal shear strength, lb.

V_s = shear strength provided b shear reinforcement, lb.

V_u = required shear strength due to factored shear force, lb.

V_x = the design story shear in Story x.

w = uniformly distributed load.

= width of beam, wall, or column, inches.

w_b = width of beam in a masonry frame, inches.

w_i, w_x = that portion of W which is located at or is assigned to level i or x respectively.

w_{px} = the weight of the diaphragm and the elements tributary thereto at Level x.

w_s = unit weight of the soil, pounds per cubic foot.

w_{strut} = horizontal projection of the width of the diagonal strut, in.

w_u = out-of-plane factored uniformly distributed load, lb/in.

W = wind load, or related internal moments in forces.

= weight of soil wedge.

= West.

W_a = actual width of masonry unit, inches.

W_p = the weight of en element or component.

= the weight of a part or a portion of a structure.

Wt = weight, pounds, kips.

WSD = See ASD.

WT = equivalent web thickness of hollow masonry units, inches.

x_{CR} = distance from y axis to center of rigidity.

y_{CR} = distance from x axis to center of rigidity.

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y = distance from centroidal axis of the section to centroid of area considered.

z = ratio of distance (z k d) between extreme fiber and resultant of compressive forces to distance k d.

β = 0.25 for fully grouted masonry or 0.15 for other than fully grouted masonry.

= angle of the backfill slope from a horizontal level plane.

β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at a section.

γ = reinforcement size factor.

γ_i = horizontal displacement at Level i.

γ_s = unit weight of soil, pounds per cubic foot.

Δ = calculated story drift, in.

Δ_a = allowable story drift, in.

Δ_C = coefficient of deflection for cantilever piers or walls.

Δ_F = coefficient of deflection for fixed piers or walls.

ΔL = unrestrained expansion, inches.

= change in length.

Δ_m = deflection due to moment.

Δ_s = the midheight deflection limitation for slender walls under service lateral and vertical loads, inches.

ΔT = change in temperature.

Δ_v = deflection due to shear.

Δ_u = horizontal deflection at midheight under factored load; $P\Delta$ effects must be included in the deflection calculation.

δ = angle of the wall friction to a horizontal level plane.

δ_i δ_n = deflection at levels i and n respectively, relative to the base, due to applied lateral forces.

δ_{ne} = displacements computed using code-prescribed seismic forces and assuming elastic behavior, in.

δ_s = horizontal deflection at midheight under service loads, in.

δ_u = deflection due to factored loads, in.

ε_{es} = drying shrinkage of AAC, defined as the difference in the relative change in length between the moisture contents of 30% and 6%.

ε_{mu} = maximum useable compressive strain of masonry.

μ = coefficient of sliding friction.

μ_{AAC} = coefficient of friction of AAC.

ρ = reinforcement ratio.

ρ_n = ratio of distributed shear reinforcement on plane perpendicular to plane of A_{mv}.

ρ_{max} = maximum reinforcement ratio.

Σ_o = sum of perimeters of all the longitudinal reinforcement.

ϕ = strength reduction factor.

= angle of internal friction; angle of shearing resistance in Coulomb's equation, degrees.

°C = degrees Celcius

°F = degrees Fahrenheit.

% = percent

= number

= pounds

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REINFORCED MASONRY ENGINEERING HANDBOOK

CLAY AND CONCRETE MASONRY

SIXTH EDITION

James E. Amrhein, S.E.
Consulting Structural Engineer
Original Author

Max L. Porter, P.E., Ph.D.
Iowa State University

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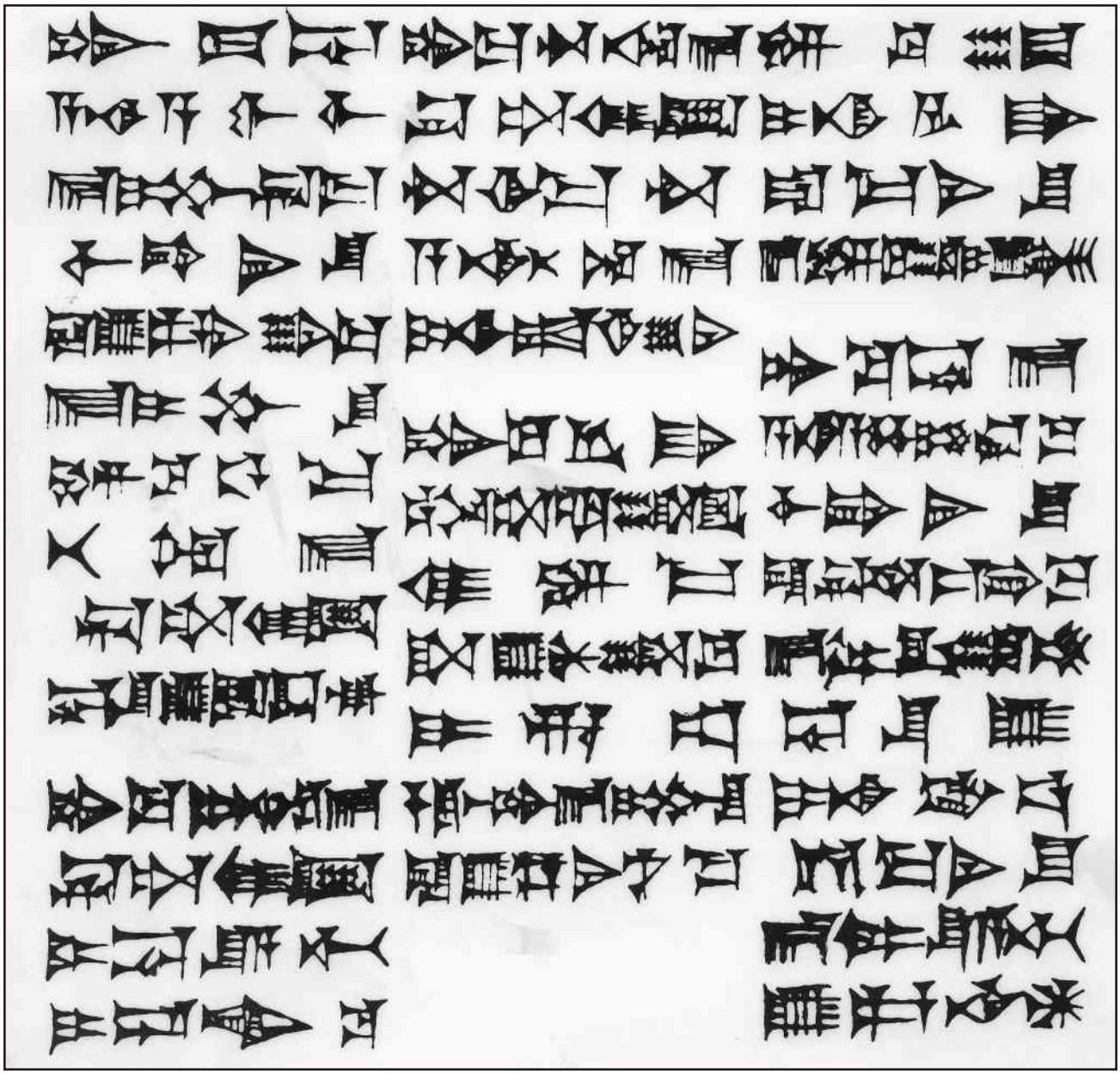

FROM THE CODE OF HAMMURABI (2200 B.C.)

If a builder builds a house for a man and does not make its construction firm and the house collapses and causes the death of the owner of the house — that builder shall be put to death. If it causes the death of a son of that owner — they shall put to death the son of that builder. If it causes the death of a slave of the owner — he shall give to the owner a slave of equal value.

If it destroys property — he shall restore whatever it destroyed and because he did not make the house firm he shall rebuild the house which collapsed at his own expense. If a builder builds a house and does not make its construction meet the requirements and a wall falls in — that builder shall strengthen the wall at his own expense.

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REINFORCED MASONRY ENGINEERING HANDBOOK

"...They said to one another, 'Come, let us make bricks and bake them.' They used bricks for stone and bitumen for mortar. Then they said, 'Let us build ourselves a city and a tower with its top in the heavens.' "

from the Old Testament of the Holy Bible, Book of Genesis, Chapter XI, Versus 3 and 4

INTRODUCTION

INTRODUCTION

Masonry structures have been constructed since the earliest days of mankind, not only for homes but also for works of beauty and grandeur. Stone was the first masonry unit and was used for primitive but breathtaking structures such as the 4000 year old Stonehenge ring on England's Salisbury Plains.

Stonehenge ring on England's Salisbury Plains.

Stone was also used around 2500 B.C. to build the Egyptian pyramids in Giza. Limestone veneer which once clad the pyramids can now be seen only at the top of the great pyramid Cheops, since much of the limestone facing was later removed and reused.

As with the Egyptian Pyramids, numerous other structures such as the 1500 mile long Great Wall of China testify to the durability of masonry.

Egyptian Pyramids located in Giza were constructed around 2500 B.C. Note limestone veneer at the top of the great pyramid, Cheops.

The 1500 mile Great Wall of China was constructed of brick and stone between 200 B.C. and 1640 A.D.

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Additionally, structures such as the stone pyramids of Yucatan and Teotihuacan, Mexico, demonstrate the skill of ancient masons.

The pyramid of El Castillo de Chichén Itzá in Yucatán in Mexico was built between 700 and 900 A.D.

The pyramid of the Sun, built in the 2nd century A.D. dominates the landscape of the ancient city of Teotihuacan in Mexico.

In fact, the stone walls at the Machu Picchu in Peru have masonry unit joints so tight that it is difficult to insert a knife blade between units.

The stone walls at Machu Picchu in Peru were built between 1200 and 1400 A.D.

Masonry has been used worldwide to construct impressive structures such as St. Basil's Cathedral in Moscow.

The outer walls of St. Basil's Cathedral in Moscow, were built in 1492, while the remainder of this impressive cathedral was constructed in the 17th century.

The Taj Mahal in Agra, India, demonstrates unique monumental characteristics of stone.

Built between 1631 and 1653, the Taj Mahal depicts grandeur in symmetry.

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In the United States, masonry is used from Maine to Hawaii and has been the primary material for building construction from the 18th to the 21st centuries.

Built in 1891, the 16 story brick Monadnock Building in Chicago is still in use today.

In the early 1900's concrete block masonry units (CMU) were introduced to the construction industry. Later, between 1930 and 1940, reinforcing steel was introduced into masonry construction to provide increased resistance to lateral dynamic forces from earthquakes.

Prior to the development of reinforced masonry, most masonry structures were designed to support only gravity loads, while the forces from wind and earthquakes were ignored. Massive dead loads from the thick and heavy walls stabilized the unreinforced structures against lateral forces.

The introduction of reinforced masonry allowed wall thickness to be decreased dramatically and provided a rational method to design walls to resist dynamic lateral loads from winds and earthquakes.

An excellent example of the benefits of reinforced masonry is the 13 story Pasadena Hilton Hotel in California, completed in 1971. The load bearing, high strength concrete block walls are 12 in. (305 mm) thick CMU for the bottom three floors and 8 in. (203 mm) thick CMU for the upper 10 floors.

13 Story Pasadena Hilton Hotel, Completed in 1971.

The Pasadena Hilton, like the newer 16 story Queens Surf in Long Beach, California and the 19 story Holiday Inn in Burbank, California is located in one of the most severe seismic areas in the world.

Constructed primarily of concrete masonry units, the Queen's Surf in Long Beach, California rises 16 stories.

Another outstanding example of reinforced load bearing masonry is the 28 story Excalibur Hotel in Las Vegas, Nevada. This large high-rise complex consists of four buildings each containing 1008 hotel rooms. The load bearing walls for the complex required masonry with a specified compressive strength of 4,000 psi at the base of the wall.

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28 Story Excalibur Hotel, Las Vegas, Nevada.

Although taller masonry buildings may someday be constructed, it is of more importance that the benefits of reinforced masonry are appropriate not only for multi-story buildings, but for buildings of every size and type, even single story dwellings.

The Getty Center, Los Angeles, California.

USC – Galen Center and Athletic Pavilion, Los Angeles, California.

BASIS OF DESIGN

The basis of design for masonry structures described in this publication are the requirements found in the International Building Code, (IBC) published by the International Code Council, (ICC) and to a lesser extent, the requirements of the *Building Code Requirements for Masonry Structures* (ACI 530-05/ASCE 5-05/TMS 402-05) and the *Specification for Masonry Structures* (ACI 530.1-05/ASCE 6-05/TMS 602-05). The allowable stresses for masonry and reinforcing steel, dead loads, live loads and lateral forces as prescribed by the IBC are used primarily herein, although ACI/ASCE/TMS allowable stresses equations are given as well, in Chapter 10.

Similar to past editions, numerous tables and diagrams have been provided at the end of this book to facilitate the design of masonry structures. Additional tables have been included to simplify strength design procedures and the ACI/ASCE/TMS design methods, while some of the seldom used old tables were deleted. Note, however, to avoid confusion, the table and diagram numbers were kept the same to be consistent with past editions – thus some gaps exist in the table numbering.

Chapter 14 provides explanations for the tables and diagrams. Additionally, numerous example problems are provided throughout the book, which demonstrates these tables and diagrams. Cross references have also been included at the top of most tables and diagrams to direct the reader to appropriate examples.

Included in this publication is information, tables and some design charts that conform to the requirements of the *Building Code Requirements for Masonry Structures* (ACI 530-05/ASCE 5-05/TMS 402-05) and the *Specification for Masonry Structures* (ACI 530.1-05/ASCE 6-05/TMS 602-05).

As an engineer and designer, one should not get lost in the precision of the numbers listed in the design tables of this handbook, and lose sight of the fact that loads for which the structures are designed are arbitrary and in many cases significantly different than the actual loads.

Judgement in design and detailing which insures both safety and economy is the mark of a professional engineer.

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CHAPTER 1

MATERIALS

1.1 GENERAL

The four principal materials used in reinforced masonry are the masonry units, mortar, grout and reinforcing steel. These materials are assembled into a homogeneous structural system. The primary documents of reference in this publication are the International Building Code (IBC), and the Masonry Standards Joint Committee (MSJC Code) code provisions for masonry and specifications, more precisely designated by the following:

- "International Building Code" (2006 IBC).
- "Building Code Requirements for Masonry Structures" (ACI 530-05/ASCE 5-05/TMS 402-05), and corresponding Commentary referred to as the MSJC Code.
- "Specification for Masonry Structures" (ACI 530.1-05/ASCE 6-05/TMS 602-05), and corresponding Commentary referred to as the MSJC Specification.

The 2006 edition of the IBC is used frequently in this publication, as well as the 2005 edition of MSJC Code. The 2006 IBC references the 2005 MSJC Code, and therefore, many citations in this publication will concentrate on the MSJC Code, but references and differences will also be cited in the IBC. Both the IBC and the MSJC Code make use of other documents. For example, IBC and MSJC Code reference ASTM Standards for material and testing, and reference ASCE 7 for design loads and load-related items.

Since the MSJC Code refers to ASCE 7-02 in several places, the reader is cautioned to check the loads in using ASCE 7-05 versus the ASCE 7-02 version. Also, the 2005 MSJC Code states in Section 1.7 that the ASCE 7-93 version shall be used where service loads are absent in the legally adopted building code, except as noted elsewhere in the MSJC Code. Thus, the reader is cautioned also as to the use of the proper edition of the ASCE 7 document when using the IBC and MSJC Code, and to the citations used in the local legally adopted building code of jurisdiction.

1.2 MASONRY UNITS

Masonry units considered in this publication are clay brick, concrete brick, hollow clay brick and hollow concrete block. Note that the use of the words "brick" and "block" has become colloquial. The proper or more formal terms are "clay and concrete masonry units", respectively, as applied to the corresponding material. Thus, this publication will utilize the newer name forms and refrain from the use of "brick" or "block". However, structural principles given in this publication apply to all types of masonry by using the appropriate allowable stress values.

Examples of the other forms of masonry units are stone, cut stone, prefabricated stone, ashlar, marble, glass, autoclave aerated concrete (AAC), and thin masonry. The units of masonry make up structural components; for example, shear walls, beams, arches, frames, prestressed masonry, veneer, glass walls, infilled walls, and repair and retrofit masonry. This chapter concentrates on the materials; whereas, the structural aspects of the components and complete structures will be covered in later chapters. This publication concentrates on structural uses of masonry, and thus, for example, ceramic wall tile and floor tile units and applications are not addressed.

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Masonry units are available in a variety of sizes, shapes, colors, and textures. Always check with the local manufacturer or supplier for the properties, physical characteristics and availability of the desired units.

1.2.1 CLAY MASONRY

Clay masonry is manufactured to comply with the ASTM International (ASTM) C62; *Specification for Building Brick (Solid Masonry Units Made from Clay or Shale)*, C216; *Specification for Facing Brick (Solid Masonry Units Made from Clay or Shale)* and C652; *Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale)*. Clay masonry is made by firing clay in a kiln for 25 to 35 hours depending upon the type of kiln, size and volume of the units and other variables. The clay is fired at a fusing temperature between 1600°F to 2700°F, depending on the type of clay. For building brick and face brick the temperature is controlled between 1600°F and 2200°F, while the temperature ranges between 2400°F and 2700°F for fire brick.

Ordinary fired clay units have been available in the United States for many years. For example, Independence Hall in Philadelphia, Pennsylvania, was constructed in 1730 and is shown in Figure 1.1.

FIGURE 1.1 Independence Hall in Philadelphia, Pennsylvania, constructed in 1730 of fired brick.

Clays, unlike metals, soften slowly and fuse gradually when subjected to elevated temperatures. This softening property allows clay to harden into a solid and durable unit when properly fired.

Fusing takes place in three stages:

1. Incipient fusion – occurs when the clay particles become sufficiently soft causing the mass to stick together.
2. Vitrification – characterized by extensive fluxing as the mass densifies and solidifies.
3. Viscous fusion – the point at which the clay mass begins to break down and becomes molten.

The key to the firing process is to control the temperature in the kiln so that incipient fusion is complete, and partial vitrification occurs but viscous fusion is avoided.

After the temperature reaches the maximum and is maintained for a prescribed time, the cooling process begins. Usually 48 to 72 hours are required for proper cooling in periodic kilns, and up to 48 hours in tunnel kilns. The rate of cooling has a direct effect on color and the finished quality. Additionally, excessively rapid cooling may cause cracking of the units, and therefore must be controlled closely.

Clays shrink during both drying and firing; therefore, allowances must be made in the size of the finished product. Both drying shrinkage and firing shrinkage vary for different clays, usually falling within the following ranges:

Drying Shrinkage	2 to 8 percent
Firing Shrinkage	2.5 to 10 percent

Firing shrinkage increases with higher temperatures which, in turn, produce darker shades. Consequently, when a wide range of colors is desired, some variation between the final sizes of the dark and light units is inevitable.

To obtain products of uniform size, manufacturers attempt to control factors contributing to shrinkage. However, because of variations in the raw materials and temperature variations within kilns, absolute uniformity is unattainable. Specifications for brick include permissible size variations.

Clay units are manufactured in accordance with the prescribed standards of the ASTM and are classified as either solid units or hollow units. Examples of solid brick are shown in Figure 1.2

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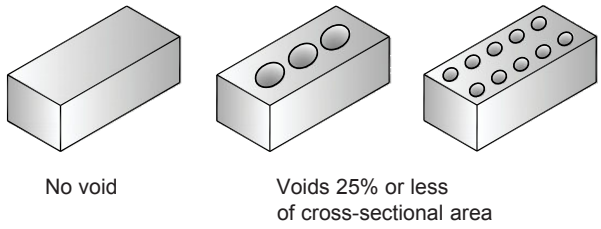

FIGURE 1.2 Solid clay brick.

1.2.1.1 Solid Clay Units

A solid clay masonry unit, as specified in ASTM C62 and C216, is a unit whose net cross-sectional area, in every plane parallel to the bearing surface, is 75% or more of its gross cross-sectional area measured in the same plane. A solid brick may have a maximum coring of 25%.

Solid clay units are referenced in IBC Section 2103.2 and in MSJC Specification Article 2.3 B.

Building bricks are classified as solid masonry units used where appearance is not a consideration. ASTM C62 includes three grades of building brick (SW, MW and NW) which relate the physical requirements to the durability of a brick unit.

Facing bricks are solid masonry units used where the appearance of the units is a consideration. Limits on chippage and cracks, as well as tolerances on the dimensions and distortions of facing brick are included in ASTM C216. This standard covers two grades of facing brick based on their resistance to weathering.

The recommended uses, physical requirements and grade requirements of building brick are the same as for Grades SW and MW under ASTM C62.

1.2.1.1.1 Grades of Building and Facing Bricks

Bricks are graded according to their weathering resistance.

The effect of weathering on a brick is related to the weathering index which, for any locality, is the product of the average annual number of freezing cycle days and the average annual winter rainfall in inches. Grade requirements for face exposures are listed in Table 1.1. Figure 1.3 displays weathering indexes for the United States. The physical requirements for each grade are included in ASTM C62 and C216. Facing brick is classified only as Grades SW and MW.

TABLE 1.1 Grade Requirements for Face Exposures (ASTM C62 Table 2; ASTM C216 Table 2)

Exposure	Weathering Index		
	Less than 50	50 to 500	500 and greater
In vertical surfaces:			
In contact with earth	MW	SW	SW
Not in contact with earth	MW	SW	SW
In other than vertical surfaces:			
In contact with earth	SW	SW	SW
Not in contact with earth	MW	SW	SW

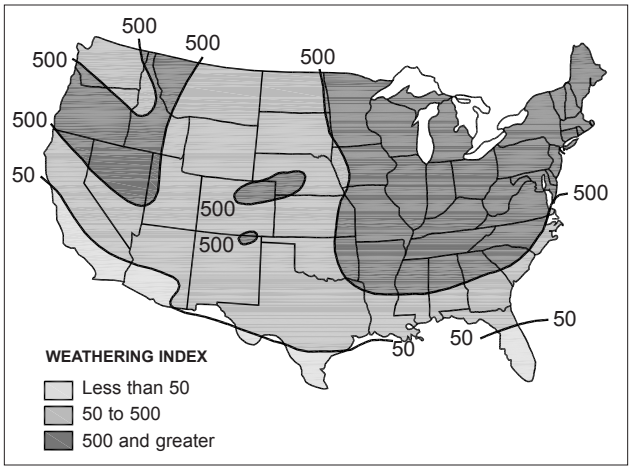

FIGURE 1.3 Weathering index map of the United States.

GRADE SW (Severe Weathering) bricks are intended for use where a high and uniform degree of resistance to frost action and disintegration by weathering is desired and the exposure is such that the brick may freeze when permeated with water.

GRADE MW (Moderate Weathering) bricks are used where they will be exposed to temperatures below freezing, but unlikely to be permeated with water, and where a moderate and somewhat non-uniform degree of resistance to frost action is permissible.

GRADE NW (Negligible Weathering) applies to building brick only and is intended for use in backup or interior masonry.

1.2.1.1.2 Types of Facing Bricks

Included in ASTM C216 are three types of facing, or face brick based upon factors affecting the appearance of the finished wall. These types of face bricks are described as follows:

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TYPE FBS (Face Brick Standard) brick is for general use in exposed masonry construction. Most bricks are manufactured to meet the requirement of Type FBS.

TYPE FBX (Face Brick Extra) brick is for general use in exposed masonry construction where a higher degree of precision and a lower permissible variation in size than that permitted for Type FBS brick is required.

TYPE FBA (Face Brick Architectural) brick is manufactured and selected to produce characteristic architectural effects resulting from non-uniformity in size and texture of the individual units.

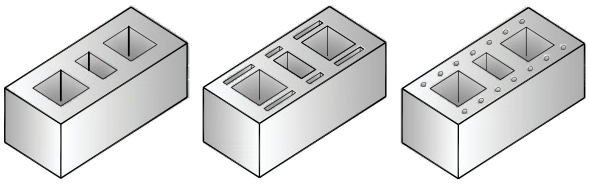
1.2.1.1.3 SOLID CLAY BRICK SIZES

There are no standard solid clay brick sizes and therefore it is always necessary to check with the brick manufacturer or supplier for the actual brick dimensions. As a guide some typical brick sizes are shown below:

	Width	Height	Length
Standard Brick:	$3\frac{3}{4}$"	$2\frac{1}{4}$"	8"
Modular Brick:	$3\frac{5}{8}$"	$2\frac{1}{4}$"	$7\frac{5}{8}$"
Oversize Brick:	3"	$2\frac{5}{8}$"	$9\frac{5}{8}$"
Norman Brick:	$3\frac{1}{2}$"	$2\frac{1}{4}$"	$11\frac{1}{2}$"
Jumbo Brick:	3"	$3\frac{1}{2}$"	$11\frac{1}{2}$"

1.2.1.2 HOLLOW CLAY UNITS

A hollow clay masonry unit as specified in ASTM C652, and, as referenced in IBC Section 2103.2 and MSJC Specification Article 2.3 B, is a unit whose net cross-sectional area in any plane parallel to the bearing surface is less than 75% of its gross cross-sectional area measured in the same plane. Examples are shown in Figure 1.4. Hollow clay units are classified by Grade, Type and Class as outlined below.



Solid shell hollow brick units | Double shell hollow brick units | Cored shell hollow brick units

FIGURE 1.4 Hollow clay brick.

1.2.1.2.1 GRADES OF HOLLOW BRICK

Two grades of hollow brick are covered: Grade SW and Grade MW. These grades are similar to the grades for solid brick.

1.2.1.2.2 TYPES OF HOLLOW BRICK

Four types of hollow brick are covered in ASTM C652.

TYPE HBS (Hollow Brick Standard) is for general use in exposed exterior and interior masonry walls and partitions where a wider color range and a greater variation in size than is permitted for Type HBX hollow brick.

TYPE HBX (Hollow Brick Extra) is for general use in exposed exterior and interior masonry walls and partitions where a high degree of mechanical perfection, a narrow color range, and a minimal variation in size is required.

TYPE HBA (Hollow Brick Architectural) is manufactured and selected to produce characteristic architectural effects resulting from nonuniformity in size, color and texture of the individual units.

TYPE HBB (Hollow Brick Basic) is for general use in masonry walls and partitions where color and texture are not a consideration, and where a greater variation in size is permitted than is required by Type HBX hollow brick.

1.2.1.2.3 CLASSES OF HOLLOW BRICK

Two classes of hollow brick are covered in ASTM C652:

Class H40V – Hollow brick intended for use where void areas or hollow spaces are between 25% to 40% of the gross cross-sectional area of the unit measured in any plane parallel to the bearing surface.

Class H60V – Hollow brick intended for use where larger void areas are desired than allowed for class H40V brick. The sum of the void areas for class H60V must be greater than 40%, but not greater than 60%, of the gross cross-sectional area of the unit measured in any plane parallel to the bearing surface. The void spaces, the web thicknesses, and the shell thicknesses must comply with the minimum requirements contained in Table 1.2.

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TABLE 1.2 Class H60V – Hollow Brick Minimum Thickness of Face Shells and Webs (ASTM C652, Table 1)

Nominal Width of Units (in.)	Face Shell Thicknesses Solid (in.)	Cored or Double Shell (in.)	End Shells or End Webs (in.)
3 and 4	3/4	—	3/4
6	1	1 1/2	1
8	1 1/4	1 1/2	1
10	1 3/8	1 5/8	1 1/8
12	1 1/2	2	1 1/8

1.2.1.2.4 Sizes of Hollow Brick

Hollow clay brick, like solid brick, are available in a variety of sizes but are customarily manufactured in nominal 4, 6 or 8 in. thicknesses. Actual thicknesses, however, are about 1/2 in. less than the nominal thicknesses (i.e., a 6 in. nominal hollow brick is actually about 5 1/2 in. thick.)

1.2.1.3 Physical Requirements of Clay Masonry Units

1.2.1.3.1 General

The physical requirements for each grade of solid and hollow brick are compressive strength, water absorption and the saturation coefficient as shown in Table 1.3. However, note that facing brick is only classified into Grades SW and MW.

TABLE 1.3 Physical Requirements, Solid and Hollow Bricks (ASTM C62, Table 1; ASTM C216, Table 1; ASTM C652, Table 2)

Designation	Minimum Compressive Strength for Brick Flatwise Based on Gross Area (psi)		Maximum Water Absorption by 5 Hour Boiling Percent		Maximum Saturation Coefficient[1]	
	Average of 5 Bricks	Individual	Average of 5 Bricks	Individual	Average of 5 Bricks	Individual
Grade SW	3000	2500	17.0	20.0	0.78	0.80
Grade MW	2500	2200	22.0	25.0	0.88	0.90
Grade NW[2]	1500	1250	no limit	no limit	no limit	no limit

1. The saturation coefficient or C/B ratio, is the ratio of absorption by 24-hour submersion in cold water to that after 5-hour submersion in boiling water.
2. Does not apply for ASTM C216 and C652.

1.2.1.3.2 Water Absorption and Saturation Coefficient

The water absorption rate and saturation coefficient (known as the C/B ratio) are indications of the freeze-thaw resistance of a brick. The values for Grade SW brick and Grade MW brick indicate that there are more voids or pores in Grade SW units which allows water to expand as it transforms into ice.

1.2.1.3.3 Tolerances

Table 1.4 shows the allowable tolerances for face brick and hollow clay brick according to ASTM C216 and ASTM C652, respectively. Dimensional tolerances for building brick conforming to ASTM C62 are the same as for Type FBS. For tolerances on distortion see ASTM C216 and C652.

TABLE 1.4 Dimensional Tolerances (ASTM C216, Table 3; ASTM C652, Table 3)

Specified Dimension (in.)	Maximum Permissible Variation from Specific Dimensions, Plus or Minus (in.)	
	Type FBX; HBX	Type FBS; HBS & HBB
3 and under	1/16	3/32
Over 3 to 4, incl.	3/32	1/8
Over 4 to 6, incl.	1/8	3/16
Over 6 to 8, incl.	5/32	1/4
Over 8 to 12, incl.	7/32	5/16
Over 12 to 16, incl.	9/32	3/8

ASTM C67, *Test Methods for Sampling and Testing Brick and Structural Clay Tile,* includes methods for measuring water absorption and the saturation coefficient.

The saturation coefficient, commonly called the C/B (Cold/Boiling) ratio, is the percent absorption of the twenty-four hour cold water test divided by the percent absorption of the five-hour boiling test.

The C/B ratio is based on the concept that only a portion of the pores will be filled during the cold water test, and that all the pores which can possibly be filled will be filled during the boiling test.

1.2.1.3.4 Initial Rate of Absorption, I.R.A.

The initial rate of absorption (suction) of a brick has an important effect on the bond between the brick and the mortar. It is defined as the amount of water in grams per minute absorbed by 30 square

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inches of brick in one minute. Maximum bond strength occurs when the suction of the brick at the time of placement is between 5 and 20 grams of water per 30 square inches of brick when the surface area is immersed in 1/8 in. of water for one minute.

Note that there is no consistent relationship between total absorption and suction or I.R.A. Some bricks with high absorption have low suction (I.R.A.) and vice versa. Suction of the brick while being laid is of primary importance and suction can be controlled at the jobsite by wetting.

Dry bricks and bricks with high suction rates tend to absorb large quantities of water from mortar which often results in poor bond adhesion. Therefore, wetting the dry bricks a few hours prior to laying is advisable so the cores are moist while the surface is dry. Bricks in this condition, with a dry surface and wet core, are preferred since they tend to bond well with the mortar. Note that very wet or saturated bricks should be avoided since they may not bond well to the mortar. Saturated bricks move easily and do not stay in position (float), thus making bricklaying extremely difficult and slow.

To check the internal moisture condition of a brick, the bricklayer or inspector should occasionally break a brick and observe the interior dampness condition.

Brick properties often vary significantly depending on the clay type and the manufacturer. Consultation with the local brick manufacturer is advisable for specific information on the intended brick for a project.

1.2.2 CONCRETE MASONRY

Concrete masonry units for load bearing systems may be either concrete brick as specified by ASTM C55, *Specification for Concrete Brick* or hollow load bearing concrete masonry units as specified by ASTM C90, *Specification for Loadbearing Concrete Masonry Units*. Likewise, these units are referenced in IBC Section 2103.1 and in MSJC Specification Article 2.3 A.

Concrete brick and hollow units are primarily made from portland cement, water and suitable aggregates with or without the inclusion of other materials.

Concrete brick and hollow units may be made from lightweight or normal weight aggregates or both.

1.2.2.1 CONCRETE BRICK

Concrete brick are typically solid units used for special purposes. Some applications include top or bearing course of load bearing masonry walls, exterior walls of masonry fireplaces and catch basin or manhole construction. ASTM C55 provides the property requirements for concrete brick. Note that component units normally conform to the requirements of ASTM C55.

Unlike masonry units specified under ASTM C90, concrete brick maintain the Grade N and Grade S designation requirements. Concrete brick must also withstand higher compression capacity as outlined in the following sections.

1.2.2.1.1 PHYSICAL PROPERTY REQUIREMENTS

The strength and absorption requirements for concrete brick are given in Table 1.5.

TABLE 1.5 Strength and Absorption Requirements (ASTM C55, Table 1)

Compressive Strength, Min., for Concrete Brick Tested Flatwise (psi)			Water Absorption Max., (Avg. of 3 Brick) with Oven Dry Weight of Concrete (lb/ft^3)		
Average Gross Area			Weight Classification		
Grade	Avg. of 3 Concrete Brick	Individual Concrete Brick	Light-weight Less than 105	Medium Weight Less than 125 to 105	Normal Weight 125 or More
N	3500	3000	15	13	10
S	2500	2000	18	15	13

1.2.2.2 HOLLOW LOADBEARING CONCRETE MASONRY UNITS

As previously noted, the physical and property requirements for concrete masonry units are contained in ASTM C90. The designer must understand that this material standard is very dynamic, that is, it is revised frequently. Often the standard is updated 2 or 3 times a year.

The Grades (S and N) and Types (I and II) have been deleted in favor of the more rigorous requirements. Consequently, it is no longer appropriate to specify a 'Grade N, Type I' unit. Grade designations were deleted in the early 1990's and the type designation was withdrawn in the year 2000.

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1.2.2.2.1 Physical Property Requirements

ASTM C90 requires concrete masonry units to meet the strength and moisture absorption requirements listed in Table 1.6.

TABLE 1.6 Strength and Absorption Requirements (ASTM C90 Table 2)

Compressive Strength[1], Min. (psi)		Water Absorption, Max.[2], (Avg. of 3 Units) with Oven Dry Weight of Concrete (lb/cu. ft)		
Average Net Area		Weight Classification		
Avg. of 3 Units	Individual Unit	Light-weight, Less than 105	Medium Weight, 105 to less than 125	Normal Weight 125 or more
1900	1700	18	15	13

1. Higher compressive strengths may be specified where required by design. Consult with local suppliers to determine availability of units of higher compressive strength.
2. Note: To prevent water penetration, protective coating should be applied on the exterior face of the basement walls and when required on the face of exterior walls above grade.

The water absorption requirements are based on three weight classifications for hollow concrete masonry units:

1. Normal weight units at least 125 pcf when dry.
2. Medium weight units ranging from at least 105 to 125 pcf when dry.
3. Lightweight units weighing less than 105 pcf when dry.

1.2.2.2.2 Categories of Hollow Concrete Units

There are two categories of hollow concrete masonry units:

Standard Units require that no overall dimension (width, height and length) differ by more than 1/8 in. from the specified standard dimensions.

Particular Feature Units have dimensions specified in accordance with the following (local suppliers should be consulted to determine achievable dimensional tolerances):

1. For molded face units, no overall dimension (width, height and length) may vary by more than 1/8 in. from the specified standard dimension. Dimensions of molded features (ribs, scores, hex-shapes, pattern, etc.) must be within 1/16 in. of the specified standard dimensions and must be within 1/16 in. of the specified placement on the unit.
2. For split-faced units, all non-split overall dimensions may differ by no more than 1/8 in. from the specified standard dimensions. On split faces, overall dimensions will vary.
3. For slumped units, no overall height dimension may differ by more than 1/8 in. from the specified standard dimension. On slumped faces, overall dimensions will vary.

1.2.2.2.3 Sizes of Hollow Concrete Masonry Units

Concrete blocks have customarily been manufactured in modular nominal dimensions which are multiples of 8 in. (i.e., standard block are nominally 8 in. high by 16 in. long), as shown by the examples in Figure 1.5.

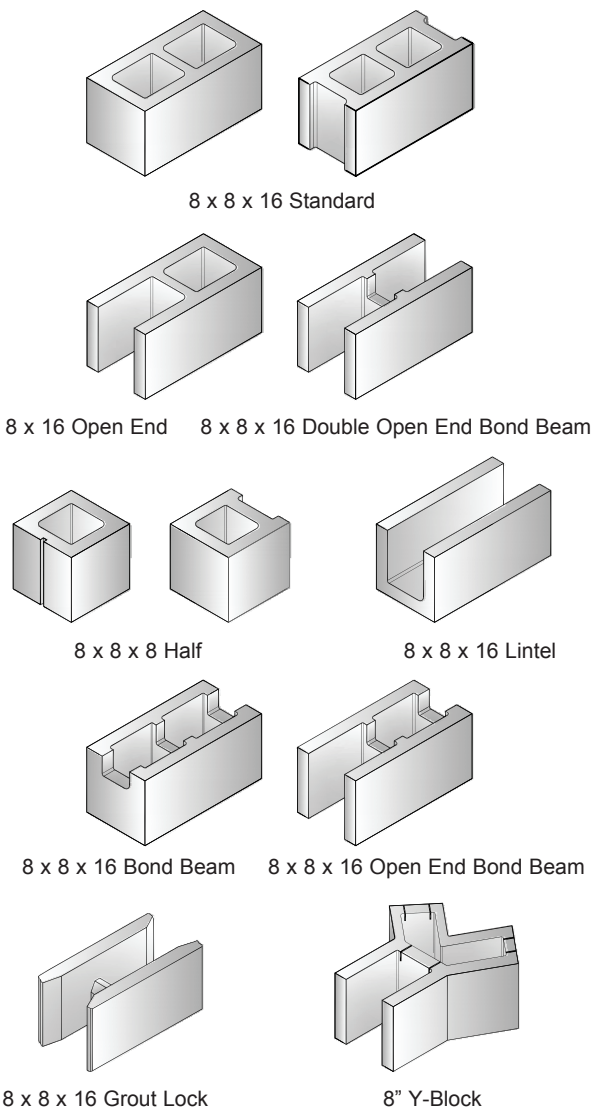

FIGURE 1.5 Typical nominal 8 in. concrete masonry units.

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The actual block dimensions, however, are typically 3/8 in. less than the nominal dimensions to account for a standard thickness mortar joint. Accordingly, an 8 x 8 x 16 in. nominal block is actually 7 5/8 x 7 5/8 x 15 5/8 inches.

Slumped block units are equal to the standard manufacturer's dimensions plus 1/2 in. to account for the thicker mortar joints used with these irregular units. Note also that the nominal dimensions of non-modular size units usually exceed the standard dimensions by 1/8 to 1/4 inch.

Face-shell thicknesses and web thicknesses of concrete masonry units are required to conform to the values listed in Table 1.7.

TABLE 1.7 Minimum Thickness of Face-Shells and Webs (ASTM C90, Table 1)

Nominal Width, (in.)	Actual Width, (in.)	Face-Shell[1] Thickness Min. (in.)	Web Thickness	
			Webs[1] Min., (in.)	Equivalent Web Thickness, (Min. in./Lin Ft)[1,2]
4	3 5/8	3/4	3/4	1 5/8
6	5 5/8	1[4]	1	2 1/4
8	7 5/8	1 1/4[4]	1	2 1/4
10	9 5/8	1 3/8[4] 1 1/4[3,4]	1 1/8	2 1/2
12	11 5/8	1 1/2 1 1/4[3,4]	1 1/8	2 1/2

1. Average of measurements on three units taken at the thinnest point, as prescribed in Test Methods ASTM C140
2. Sum of the measured thickness of all webs in the unit, multiplied by 12, and divided by the length of the unit. In the case of open-ended units where the open-ended portion is solid grouted, the length of that open-ended portion shall be deducted from the overall length of the unit.
3. This face-shell thickness is applicable where allowable design load is reduced in proportion to the reduction in thicknesses shown, except that allowable design loads on solid-grouted units shall not be reduced.
4. For solid grouted masonry construction, minimum face shell thickness shall be not less than 5/8 inches.

Special unit designs (often called face shell units or expandable units, see Figure 1.6) requiring corrosion-resistant metal ties between face shells may be used for appropriate applications. This system adds significant labor cost, but allows the designer to specify an unusual wall thickness and allows for different texture and color differences on opposite sides of the wall.

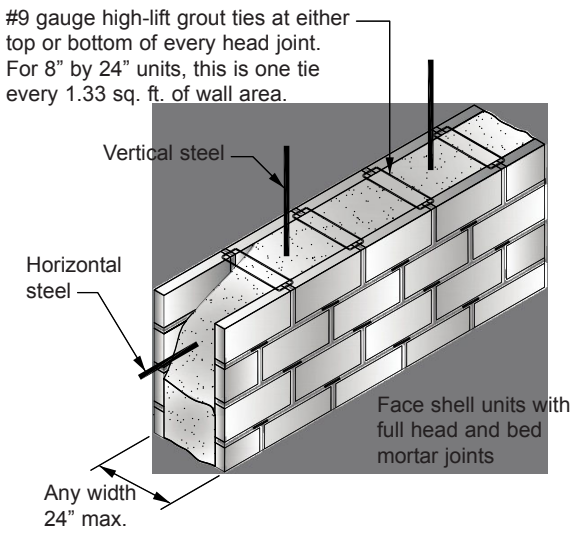

FIGURE 1.6 Expandable component masonry system.

1.2.2.3 MOISTURE CONTENT FOR CONCRETE BRICK AND HOLLOW MASONRY UNITS

The primary purpose of moisture-controlled units was to limit shrinkage of concrete block and concrete brick due to moisture loss. This limitation was based on a table considering moisture content and region of humidity to determine the maximum linear shrinkage for moisture controlled units only.

The requirement was simplified to require a maximum 0.065% maximum linear shrinkage regardless of the unit type (moisture-controlled or nonmoisture-controlled), region of humidity or moisture content.

When considering the significance of moisture content, the application of use of the masonry units should be evaluated. For fences, enclosures and retaining walls, minor cracking in walls may be acceptable since these applications typically do not require moisture resistance from one side of the wall to the other.

Determining linear shrinkage should be based on the moisture content of units when delivered to the jobsite. This implies that the masonry units might have to be protected from the weather after manufacture and during storage. Masonry units manufactured in a moist, rainy area should be stored under cover after they have sufficiently cured. Masonry units manufactured in a dry area could be stored outside and the dry weather will continue the curing process.

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Concrete block, if stored for a period of time, can achieve climatic balance and perform satisfactorily with a minimum of shrinkage. Thus, concrete block units should be protected from the weather even during storage at the jobsite. Units not covered and exposed to rain or snow at the jobsite may not meet moisture requirements until they dry. Concrete masonry units should be aged a sufficient period of time to achieve a climatic moisture balance condition. This period of time is dependent on the materials, the moisture content, the density or permeability of the block and the humidity of the area.

Construction methods have a significant influence on the performance of concrete masonry units. As the wall is constructed, the units are restrained by the mortar head joint and the adjacent units. When fluid, high slump grout is pumped into the cells, excess water is absorbed into the block, increasing its moisture content. The block may expand and, upon drying out, subsequently shrink. This condition is difficult to avoid since a highly fluid slump grout is necessary in reinforced masonry walls.

Multi-story load-bearing masonry buildings have been constructed throughout the United States. They have been built in high seismic areas and one example is the Queen's Surf in Long Beach, California, shown in Figure 1.7. This 16-story structure is built of primarily concrete masonry units.

FIGURE 1.7 Queen's Surf in Long Beach.

1.3 MORTAR

1.3.1 GENERAL

Mortar is a plastic mixture of materials used to bind masonry units into a structural mass. Mortar has the following purposes:

1. Serves as bedding or seating material for the masonry units.
2. Allows the units to be leveled and properly placed.
3. Bonds the units together.
4. Provides compressive strength.
5. Provides shear strength, particularly parallel to the wall.
6. Allows some movement and elasticity between units.
7. Seals irregularities of the masonry units.
8. Can provide color to the wall by using color additives.
9. Can provide an architectural appearance by using various types of joints, as shown in Figure 1.14.

Historically, mortar has been made from a variety of materials. Plain mud, clay, earth with ashes, and sand with lime mortars have all been used. Modern mortar consists of cementitious materials and well graded sand.

1.3.2 TYPES OF MORTAR

The requirements for mortar are provided in ASTM C270, *Specification for Mortar for Unit Masonry*, also referenced in IBC Section 2103.8 and in MSJC Specification Article 2.1 A.

There were originally five types of mortar which were designated as M, S, N, O, and K. The types can be identified by every other letter of the word MaSoNwOrK. Type K is no longer referenced in the code or material standards.

1.3.2.1 SELECTION OF MORTAR TYPES

The performance of masonry is influenced by various mortar properties such as workability, water retentivity, bond strength, durability, extensibility, and compressive strength. Since these properties vary with mortar type, selection of the proper mortar type is important for each particular application. Tables 1.8 and 1.9 are general guides for the selection of mortar type. Selection of mortar type should also consider all applicable building codes and engineering practice standards.

In Seismic Design Category (SDC) D and higher, both the IBC and MSJC Code require that mortar used in the lateral force-resisting system be Type S or M. This requirement provides additional strength and bond in structures located in high seismic risk areas.

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TABLE 1.8 Mortar Types for Classes of Construction

ASTM Mortar Type Designation	Construction Suitability
M	Masonry subjected to high compressive loads, severe frost action, or high lateral loads from earth pressures, hurricane winds, or earthquakes. Structures below or against grade such as retaining walls, etc.
S	Structures requiring high flexural bond strength, and subject to compressive and lateral loads.
N	General use in above grade masonry. Residential basement construction, interior walls and partitions. Masonry veneer and non-structural masonry partitions.
O	Non-load-bearing walls and partitions. Solid load bearing masonry with an actual compressive strength not exceeding 100 psi and not subject to weathering.

TABLE 1.9 Guide for the Selection of Masonry Mortars[1] (ASTM C270, Table X1.1)

Location	Building Segment	Mortar Type	
		Rec.	Alt.
Exterior, above grade	Load-bearing wall	N	S or M
	Non-load bearing wall	O[2]	N or S
	Parapet wall	N	S
Exterior, at or below grade	Foundation wall, retaining wall, manholes, sewers, pavements, walks and patios	S[3]	M or N[3]
Interior	Load-bearing wall	N	S or M
	Non-bearing partitions	O	N

1. This table does not provide for many specialized mortar uses, such as chimney, reinforced masonry, and acid-resistant mortars.
2. Type O mortar is recommended for use where the masonry is unlikely to be frozen when saturated or unlikely to be subjected to high winds or other significant lateral loads. Type N or S mortar should be used in other cases.
3. Masonry exposed to weather in a nominally horizontal surface is extremely vulnerable to weathering. Mortar for such masonry should be selected with due caution.

Masonry cement is also restricted in SDC D and higher. MSJC Code Section 1.14.6.6 gives this SDC exclusion as shown;

MSJC Code Section 1.14.6.6 (SDC D)

1.14.6.6 *Material requirements* — Neither Type N mortar nor masonry cement shall be used as part of the lateral force-resisting system.

1.3.2.2 SPECIFYING MORTAR

Mortar may be specified by either property or proportion specifications. Compliance verification requirements (submittals) for the specified mortar are stated in MSJC Specification Article 1.5 B.1.a:

MSJC Specification Article 1.5 B.1.a

1.5 B. Submit the following:
1. Mix designs and test results
   a. One of the following for each mortar mix, excluding thin-bed mortar for AAC:
      1) Mix designs indicating type and proportions of ingredients in compliance with the proportion specification of ASTM C270, or
      2) Mix designs and mortar tests performed in accordance with the property specification of ASTM C270.

1.3.2.2.1 PROPERTY SPECIFICATIONS

Property specifications are those in which the acceptability of the mortar is based on the properties of the ingredients and the properties of samples of the mortar (water retention, air content, and compressive strength) mixed and tested in the laboratory.

Property specifications as listed in Table 1.10 are used for research so that the physical characteristics of a mortar can be determined and reproduced in subsequent tests. Note that ASTM C780 should only be used for quality control for field tested mortar.

Compressive strength is usually the only property or characteristic which a specifier who is not a researcher would require. Most design situations can accomplish the compressive strength determination for conformance the specified compressive strength, f'_m, by the proportion procedure in ASTM C270. However, the property procedure in C270 provides for compressive strength determination. Two methods are used to determine the compressive strength of mortar. The first method tests 2 in. cubes of mortar in compression after curing for 28 days. The second method, based on ASTM C780, provides for 2 in. cubes or cylinders to be tested as a comparative field determination of the compressive strength. Overall, any testing that is done for field properties is to be done in accordance with ASTM C780, whereas

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any testing to determine the mix properties for laboratory or research purposes is done in accordance with ASTM C270.

TABLE 1.10 Property Specifications for Mortar[1] (ASTM C270, Table 2)

Mortar	Type	Avg. Comp. Strength at 28 Days min. (psi)	Water Retention min. %	Air Content max. %	Aggregate Ratio (Measured in Damp, Loose Conditions)
Cement-Lime	M	2500	75	12	Not less than 2 1/4 and not more than 3 1/2 times the sum of the separate volume of cementitious materials
	S	1800	75	12	
	N	750	75	14[2]	
	O	350	75	14[2]	
Mortar Cement	M	2500	75	12	
	S	1800	75	12	
	N	750	75	14[2]	
	O	350	75	14[2]	
Masonry Cement	M	2500	75	18	
	S	1800	75	18	
	N	750	75	20[3]	
	O	350	75	20[3]	

Note: The property requirements of this table cannot be used as a requirement quality control of field prepared mortar, instead ASTM C780 should be used for this quality control.

1. Laboratory-prepared mortar only.
2. When structural reinforcement is incorporated in cement-lime or mortar cement mortar, the maximum air content shall be 12 percent.
3. When structural reinforcement is incorporated in masonry cement mortar, the maximum air content shall be 18 percent.

Table 1.11 provides a comparison of the equivalent strength between cylinders and cube specimens for three types of mortar.

TABLE 1.11 Compressive Strength of Mortar[1] (psi)

Mortar Type	2-in. Diameter x 4-in. Height Cylinder Specimen	2-in. Cube Specimen
M	2100	2500
S	1500	1800
N	625	750

1. Lesser periods of time for testing may be used provided the relation between early tested strength and the 28-day strength of the mortar is established.

The field strength of mortar should be used only as a quality control test, rather than a quantification evaluation. The in-place mortar strength can be much higher than the test values. Higher in-place strength is a result of a lower cement-water ratio since the units draw excess moisture from the mortar and lower height to thickness aspect ratio (1/4 to 5/8 in. high by 1 1/4 to 4 in. wide) mortar joints. Additionally, the masonry units above and below the mortar joint, as well as the grout, confine the mortar so that the in-place mortar strength is much higher than the strengths of the test specimens.

National Concrete Masonry Association's (NCMA) TEK 18-5 explains that mortar compressive strength is often misinterpreted for several reasons. First, mortar compressive strength in the laboratory is not indicative of the mortar in the masonry wall. Second, there are several different test methods for determining mortar compressive strength and when mortar is correctly proportioned in accordance with ASTM C270, compressive strength values are not given. Additionally, the water-cement ratio of mortar in the wall is more favorable than mortar cast in test cylinders and the aspect ratio of mortar in a test cylinder or mortar cube is greater than mortar in a joint.

Figure 1.8 depicts compressive strength implications of laboratory mortar test specimens compared to the mortar in a masonry wall. This information is contained in NCMA TEK 107 published in 1979 and shows that mortar in a 3/8 in. joint has significantly greater compressive strength than mortar in a 1 in. cube or 2 in. test cylinder.

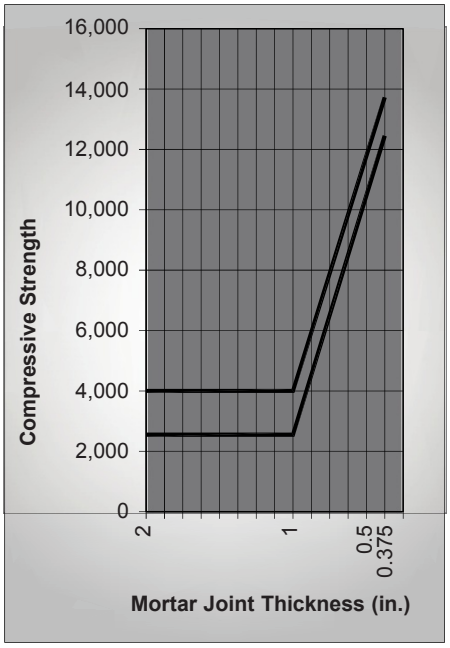

FIGURE 1.8 Effect of specimen thickness on compressive strength.

Because the in-place mortar strength exceeds the cube and cylinder test strengths, mortar will perform well even when tests on mortar are less than the specified strength of the mortar specimens. Additionally, because the in-place strength is quite

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high, mortar performs well even when the specified compressive strength of the entire masonry assemblage, f'_m, is higher than the cylinder and cube strengths.

1.3.2.2.2 PROPORTION SPECIFICATIONS

Proportion specifications limit the amount of the constituent parts by volume. Water content, however, may be adjusted by the mason to provide proper workability under various field conditions. When the proportions of ingredients are not specified, the proportions by mortar type must be used as given in Table 1.12. Mortars other than those approved in Table 1.12 may be used when laboratory or field tests demonstrate that the mortar, when combined with the masonry units, will satisfy the specified compressive strength, f'_m. However, if field tests are used for quality control, then ASTM C780 must be used, not ASTM C270.

Common cement-lime mortar proportions by volume are:

Type M mortar; 1 portland cement: 1/4 lime: 3 1/2 sand
Type S mortar; 1 portland cement: 1/2 lime: 4 1/2 sand
Type N mortar; 1 portland cement: 1 lime: 6 sand
Type O mortar; 1 portland cement: 2 lime: 9 sand

1.3.3 MORTAR MATERIALS

The principal mortar constituents are cement, lime, sand and water – each making a unique contribution to a mortar's performance. Cement contributes to mortar durability, high early strength and high compressive strength. Lime contributes to workability, water retentivity and elasticity. Both contribute to bond strength. Sand acts as a filler and contributes to the strength. Water is the ingredient which creates a plastic, workable mortar and is required for the hydration of the cement.

1.3.3.1 CEMENTS

Three types of cement are now permitted to be used in mortar by the IBC and the MSJC Code: portland cement, masonry cement and mortar cement. Plastic cement, or plasterer's cement is not acknowledged as an acceptable material and therefore must not be used in mortar.

Masonry cement and mortar cement are designated as Types M, S and N, which is not the same as the mortar type (M, S, N and O). In Table 1.12, the M, S and N designation for masonry and mortar cements in the third row represents gray, pure cement added to other materials to make mortar, whereas the M, S, N, and O designations in column 2 signify the mortar type (already mixed) the mason uses to lay the unit.

1.3.3.1.1 PORTLAND CEMENT

The basic cementitious ingredient in most mortar is portland cement. This material must meet the requirements of ASTM C150 for *Portland Cement*. In mortar, the type of portland cement is limited to Type I, II, III or V. The use of air-entraining portland cement (Type IA, IIA or IIIA) is not recommended for masonry mortar because air entrainment can reduce the bond between mortar and the masonry units.

TABLE 1.12 Mortar Proportions for Unit Masonry (IBC Table 2103.8(1), ASTM C270, Table 1)

Mortar	Type	PROPORTIONS BY VOLUME (Cementitious Materials)								Aggregate Measured in a Damp, Loose Condition
		Portland Cement or Blended Cement	Masonry Cement			Mortar Cement			Hydrated Lime or Lime Putty	
			M	S	N	M	S	N		
Cement-lime	M	1	–	–	–	–	–	–	1/4	Not less than 2 1/4 and not more than 3 times the sum of the separate volumes of cementitious materials
	S	1	–	–	–	–	–	–	over 1/4 to 1/2	
	N	1	–	–	–	–	–	–	over 1/2 to 1 1/4	
	O	1	–	–	–	–	–	–	over 1 1/4 to 2 1/2	
Mortar cement	M	1	–	–	–	–	–	1	–	
	M	–	–	–	–	1	–	–	–	
	S	1/2	–	–	–	–	–	1	–	
	S	–	–	–	–	–	1	–	–	
	N	–	–	–	–	–	–	1	–	
	O	–	–	–	–	–	–	1	–	
Masonry cement	M	1	–	–	1	–	–	–	–	
	M	–	1	–	–	–	–	–	–	
	S	1/2	–	–	1	–	–	–	–	
	S	–	–	1	–	–	–	–	–	
	N	–	–	–	1	–	–	–	–	
	O	–	–	–	1	–	–	–	–	

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Portland cement is the primary adhesive material and, based on the water cement ratio, can produce high strength mortar. Hydrated lime is used in conjunction with portland cement to provide the desired strength, workability and board life (board life is defined as the period of time during which mortar is still plastic and workable).

1.3.3.1.2 Masonry Cement

Masonry cement is a proprietary blend of portland cement and plasticizers such as ground inert fillers and other additives for workability. Masonry cement must meet the requirements of ASTM C91 *Masonry Cement* and is available for Types M, S, N and O mortar.

There are three types of masonry cement:

1. **Type N** contains the cementitious materials used in the proportions called for in ASTM C270. Type N masonry cement may also be used in combination with portland cement or blended hydraulic cement to prepare Type S or Type M mortar.
2. **Type S** contains the cementitious materials used in the proportions called for in ASTM C270.
3. **Type M** contains the cementitious materials used in the proportions called for in ASTM C270.

The use of masonry cement for mortar for the lateral force-resisting system is prohibited in Seismic Design Categories D and higher.

1.3.3.1.3 Mortar Cement

Mortar cement is also a portland cement based material which meets the requirements of ASTM C1329, *Mortar Cement*. Mortar cement may be used for mortar in all seismic design categories.

There are three types of mortar cement:

1. **Type N** contains the cementitious materials used in the proportions called for in ASTM C270. Type N mortar cement may also be used in combination with portland cement or blended hydraulic cement to prepare Type S or Type M mortars.
2. **Type S** contains the cementitious materials used in the proportions called for in ASTM C270.
3. **Type M** contains the cementitious materials used in the proportions called for in ASTM C270.

Unlike masonry cement, mortar cement can be used in high seismic applications. Mortar cement has historically had more uniform properties than masonry cement, and ASTM C1329 also requires a lower air content for mortar cement as well as testing of the flexural bond strength of the mortar. These differences give building officials the confidence to permit the use of masonry cement for significant lateral load-resisting systems.

Flexural Bond Strength of Mortar and Masonry Assemblage

The flexural bond strength of mortar cement is based on a laboratory evaluation of a standardized test apparatus, as prescribed in ASTM C1072. The test apparatus consists of a metal frame used to support a specimen as shown in Figure 1.9. The support system must be adjustable to support prisms of various heights (See ASTM C1072 for additional information on this test).

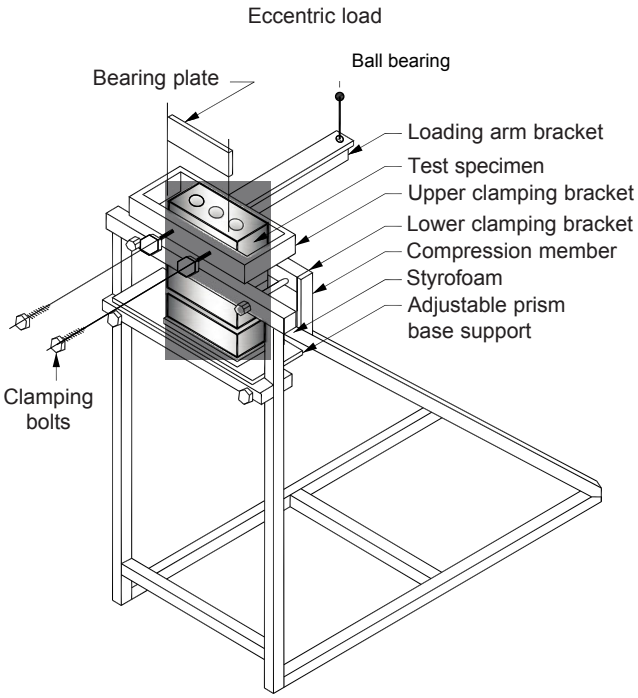

FIGURE 1.9 Bond wrench test apparatus.

1.3.3.2 Hydrated Lime

Hydrated lime is manufactured from calcining limestone (calcium carbonate with the water of crystallization, $CaCO_3H_20$). The high heat generated in the kiln drives off the water of crystallization, H_20, and the carbon dioxide, CO_2, resulting in quicklime, CaO.

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The quicklime can then be slaked by placing it in water thus making hydrated lime, lime putty or slaked lime $Ca(OH)_2$. The hydrated lime is then dried and ground, producing a white pulverized hydrated lime which is sacked and used in mortar.

Hydrated lime can be used without delay making it more convenient to use than quicklime.

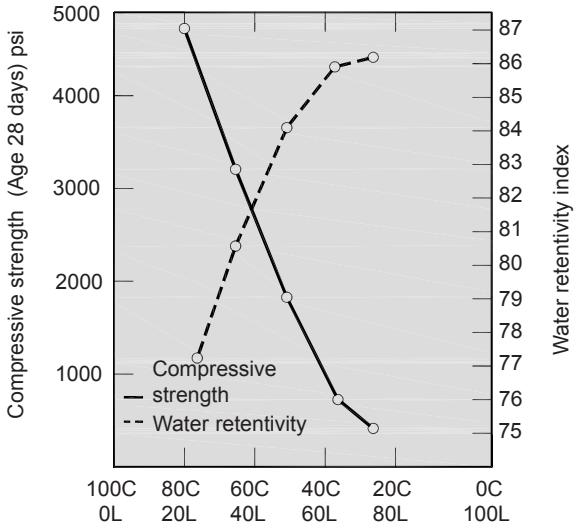
Hydrated lime is required to meet ASTM C207, *Specification for Hydrated Lime for Masonry Purposes*, and is available in the following four Types, S, SA, N and NA. Of these, only Type S hydrated lime is suitable for masonry mortar. Type S and N hydrated limes contain no air entraining admixtures. However, Types NA and SA limes may provide more entrained air in the mortar than allowed. Additionally, unhydrated oxides are not controlled in Type N or NA limes thus making only Type S hydrated lime suitable for masonry mortar.

When used in mortar, lime in mortar provides cementitious properties and is not considered to be an admixture.

Used in mortar lime:

1. Improves the plasticity or workability of the mortar.
2. Improves the water tightness of the wall.
3. Improves the water retentivity or board life of the mortar.

Figure 1.10 shows the relationship between various proportions of cement and lime versus mortar strength and water retentivity.



Proportion of Cement (%) (C): Lime (L) in mortar (C + L): Sand: 1:3 by volume

FIGURE 1.10 Relation between mortar composition, compressive strength, and water retentivity.

1.3.3.3 MORTAR SAND

For masonry mortar, sand aggregate is required to meet ASTM C144, *Specification for Aggregate for Masonry Mortar*.

Sand gradation is most often specified or defined by referring to a standard sieve analysis. For mortar, sand is graded within the limits given in Table 1.13.

TABLE 1.13 Sand for Masonry Mortar (ASTM C144, Section 4.1)

Sieve Size	Percent Passing	
	Natural Sand	Manufactured Sand
No. 4	100	100
No. 8	95 to 100	95 to 100
No. 16	70 to 100	70 to 100
No. 30	40 to 75	40 to 75
No. 50	10 to 35	20 to 40
No. 100	2 to 15	10 to 25
No. 200	0 to 5	0 to 10

Sand should be free of significant amounts of deleterious substances and organic impurities. ASTM C144 provides guidelines on determining if an aggregate has excessive impurities.

Concrete sand should not be used in mortar because the maximum grain size is too large. Additionally, the fine particles which are needed in masonry sand have often been washed out of concrete sand thus creating a harsh, coarse sand unsuitable for mortar. Mortar sand needs at least 5% fines which pass the 200 sieve to aid plasticity, workability and water retention of mortar.

Sand used in preparing mortar can be either natural or manufactured. Manufactured sand is obtained by crushing stone, gravel or air-cooled blast-furnace slag. It is characterized by sharp and angular particles producing mortars with workability properties different than mortars made with natural sand which generally have round, smooth particles.

Mortar sand, like all mortar ingredients, should be stored in a level, dry, clean place. Ideally, it should be located near the mixer so it can be measured and added with minimum handling and can be kept from contamination by harmful substances. Pre-blended mortar shipped in sacks or bulk silos circumvents the need for jobsite protection of exposed materials.

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1.3.3.4 WATER

Water must be clean and free of deleterious amounts of acids, alkalies or organic materials. Water containing soluble salts such as potassium and sodium sulfates should be avoided since these salts can contribute to efflorescence. Also, water should not be high in chloride ion content since that high content can contribute to potential rusting of reinforcement. A practical guide is to limit the chloride ion content in mortar or grout to the prescribed limits given for concrete in Table 4.4.1 of ACI 318. Alternately, epoxy- or zinc-coated reinforcement may be used for corrosion protection.

1.3.3.5 ADMIXTURES

There are numerous admixtures which may be added to mortar to affect its properties. One of these, called a retarding set admixture, delays the set and stiffening of mortar. In fact, the set may be delayed for 36 hours or more if desired.

There are also admixtures used to replace lime. These may be an air entraining chemical or a pulverized fire clay or bentonite clay to provide workability. Care should be taken with these admixtures since the bond between the mortar and the masonry units may be reduced. Use of a lime substitute may be considered when hydrated lime is not available.

The use of any admixtures must be approved by the architect or engineer and should be acceptable to the building official. Admixtures high in chloride ion contribution should be avoided, unless epoxy- or zinc-coated reinforcement is used.

1.3.3.6 COLOR

Mortar colors are generally mineral oxides or carbon black. Iron oxide is used for red, yellow, and brown colors; chromium oxide for green, and cobalt oxide for blue colors. Commercially prepared colors for mortars also offer a wide variety of colors and shades.

The amount of color additive depends on the color and intensity desired. Typically the amount of color additive ranges from 0.5% to 7.0% for the mineral oxides with a maximum of 2% for carbon black when using portland cement. MSJC Specification Article 2.6 A.2 further limits the amount of color additive that can be used with masonry or mortar cement. These percentages are based on the weight of cement content and the maximum percentages are far greater than the normal amounts of color additives generally required.

Mixing time of the mortar should be long enough for a uniform, even color to be obtained and should be the same length of time for every mortar batch. Additionally the mixing sequence should be the same for each batch.

Retempering of colored mortar must be kept to a minimum to reduce the variations in color of the mortar. For best results, mortar should not be retempered at all.

Finally, the source, manufacturer and amount of each ingredient should remain the same for all colored mortar on a project to obtain the same color throughout. Prepackaged mineral color additives that can be added to the mix based on full sacks of portland cement generally provide a consistent mortar color. Pre-blended mortars are extremely precise with material proportioning.

1.3.4 MIXING

1.3.4.1 MSJC SPECIFICATION FOR MIXING

Article 2.6 A provides the mortar mixing requirements as shown:

MSJC Specification Article 2.6

2.6 — Mixing

2.6 A. *Mortar*

1. Mix cementitious materials and aggregates between 3 and 5 min. in a mechanical batch mixer with a sufficient amount of water to produce a workable consistency. Unless acceptable, do not hand mix mortar. Maintain workability of mortar by remixing or retempering. Discard mortar which has begun to stiffen or is not used within 2 1/2 hr after initial mixing.
2. Limit the maximum percentage of mineral oxide or carbon black job-site pigments by weight of cement as follows:
   a. Pigmented portland cement-lime mortar
      1) Mineral oxide pigment 10 percent
      2) Carbon black pigment 2 percent
   b. Pigmented mortar cement mortar
      1) Mineral oxide pigment 5 percent
      2) Carbon black pigment 1 percent
   c. Pigmented masonry cement mortar
      1) Mineral oxide pigment 5 percent
      2) Carbon black pigment 1 percent
3. Do not use admixtures containing more than 0.2 percent chloride ions.
4. *Glass unit masonry* — Reduce the amount of water to account for the lack of absorption. Do not retemper mortar after initial set. Discard unused mortar within 1 1/2 hr after initial mixing.

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For thin-bed mortar used with AAC, the MSJC Code specifies the following:

MSJC Specification Article 2.6 C

2.6 C. *Thin-bed mortar for AAC* — Mix thin-bed mortar for AAC masonry as specified by the thin-bed mortar manufacturer.

1.3.4.2 MEASUREMENT OF MORTAR MATERIALS

The method of measuring materials for mortar must be such that the specified proportions of the mortar materials are controlled and accurately maintained. A reasonable method to control the mortar proportions is to use full sacks of cement in each batch and to use measuring boxes for the proper amounts of lime and sand. Dry preblended mixes are also available.

1.3.4.3 JOBSITE MIXED MORTAR

Mortar mixing is best accomplished in a paddle type mixer. About one-half of the water and one quarter of the sand are put into the operating mixer first, then the cement, lime, color (if any), and the remaining water and sand. All materials should mix for three to five minutes in a mechanical mixer with the amount of water required to provide the desired workability. Dry mixes for mortar which are blended in a factory should be mixed at the jobsite in a mechanical mixer until workable, but not more than five minutes.

Figure 1.11, shows a paddle mixer with a stationary drum. The blades rotate through the mortar materials for thorough mixing.

FIGURE 1.11 Plaster or paddle mortar mixer.

A drum or barrel mixer, shown in Figure 1.12 rotates the drum in which the materials are placed. The materials are carried to the top of the rotation and then the material drops down to achieve mixing.

FIGURE 1.12 Drum or barrel concrete mixer.

1.3.4.4 PRE-BLENDED MORTAR

Mortar can also be factory preblended and stored at the jobsite in sacks or silos. Some silo systems introduce water to the dry mortar mix in an auger screw at the base of the silo, while other silo systems discharge the dry mortar mix directly into a conventional mixer.

FIGURE 1.13 Silo mixing system.

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Pre-blended dry mortar is also available in sacks, which may be beneficial in keeping project debris at a minimum. This packaging method can be especially useful in limited working areas, such as parking garages.

When factory blended mortar is used, manufacturers certification of the type of mortar is recommended.

1.3.4.5 Extended Life Mortar

ASTM C1142, *Specification for Extended Life Mortar for Unit Masonry,* covers the requirements for this material. Extended life mortar consists of cementitious materials, aggregate, water and an admixture for set-control which are measured and mixed at a central location, using weight or volume control equipment. This mortar is delivered to a construction site and is usable for a period in excess of 2 1/2 hours.

There are four types of extended life mortar, RM, RS, RN, and RO. These types of mortar can be manufactured with one of the four mortar formulations: portland cement, portland cement-lime, masonry cement, or masonry cement with portland cement. Table 1.14 shows these property specification requirements.

TABLE 1.14 Property Specification Requirements (ASTM C1142, Table 1)

Mortar Type	Avg[1] Compressive Strength at 28 days, min. (psi)	Water Retention min. (%)	Air Content[2], max. (%)
	Cubes		
RM	2500	75	18
RS	1800	75	18
RN	750	75	18
RO	350	75	18

1. Twenty-eight days old from date of casting. The strength values as shown are the standard values. Intermediate values may be specified in accordance with project requirements.
2. When structural reinforcement is incorporated in mortar, the maximum air content shall be 12%, or bond strength test data shall be provided to justify higher air content.

Extended life mortar is selected by type and the length of workable time required. The consistency based on the mason's use should be specified. Otherwise, the extended life mortar is required to have a cone penetration consistency of 55 ± 5 mm as measured by ASTM C780, *Test Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry*.

Pre-blended mortars that meet the above criteria are popular for many jobs. These pre-blended mortars are especially popular on smaller jobs where economy of control is not available.

1.3.4.6 Retempering

Mortar may be retempered, preferably limited to one time, with water when needed to maintain workability. This should be done on wet mortar boards by forming a basin or hollow in the mortar, adding water, and then reworking the mortar into the water. Splashing water over the top of the mortar is not permissible.

Harsh mortar that has begun to stiffen or harden due to hydration, should be discarded. Mortar should be used within two-and-one-half hours after the initial water has been added to the dry ingredients at the jobsite. Retempering color mortar should be avoided to limit color variations.

1.3.5 Types of Mortar Joints

Nine examples of commonly used mortar joints are illustrated in Figure 1.14. Each joint provides a different architectural appearance to the wall. However, because some joints provide poor weather resistance, care must be taken in the selection of the proper type of mortar joint. Joints with ledges such as weather, squeezed, raked and struck joints tend to perform poorly in exterior applications and allow moisture penetration. Concave tooled joints are recommended for exterior applications since the tooling compacts the mortar tightly preventing moisture penetration.

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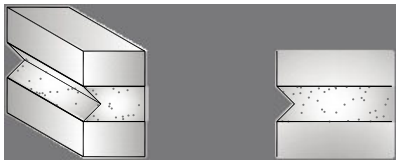
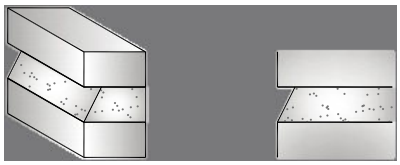
a) Concave Joint – It is the most common joint used. The tooling works the mortar tight into the joint, compressing the mortar producing a weather joint. The joint emphasizes the masonry unit pattern and conceals small irregularities in laying the unit.

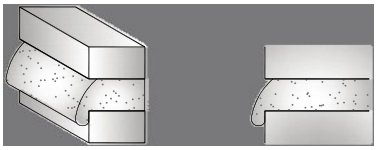


b) "V" Joint – Tooling works the mortar tight and provides a weather joint. However, the “notch” of the “V” can be a point of discontinuity and cracks may develop which allow water migration. This joint emphasizes the masonry unit pattern and conceals small irregularities in laying, while providing a line in center of mortar joint.

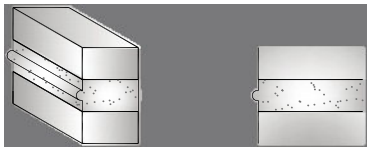


c) Weather Joint – The purpose is to emphasize horizontal joints. This type of joint is a marginally acceptable weather-type joint. The reason for this is the top ledge of the joint acts as drip ledge. If the joint is not properly tooled, the surface tension of water will allow water to pool at the drip ledge and the water can migrate back into the mortar.

d) Flush Joint – This joint is used where the wall is to be plastered. Special care is required to make the joint weatherproof. Mortar joint must be compressed to assure intimate contact with the masonry unit. Not recommended for exposed exterior use.

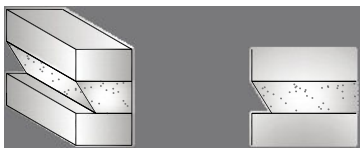


e) Squeezed Joint – This type of joint provides for a rustic, high texture appearance. Satisfactory for interior use and exterior fences. Not recommended for exterior building walls, since no weather resistance is created because the mortar is not compressed back into the joint. Also the top ledge allows for pooling of the water.



f) Beaded Joint – This is a special effect, poor exterior weather joint due to exposed ledge and is not recommended.

g) Raked Joint – This joint type strongly emphasizes the units. Poor weather joint and not recommended if exposed to weather unless tooled at bottom of mortar joint. Pooling of water can occur at the top ledge (surface tension properties of water) and the bottom ledge.



h) Struck Joint – This joint type is used to emphasize horizontal lines. Poor weather joint, therefore not recommended as water will penetrate on lower ledge.

i) Grapevine Joint – This joint shows a horizontal indentation. Same limitations as flush joint.

FIGURE 1.14 Mortar joint types.

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1.4 GROUT

1.4.1 GENERAL

Grout is a mixture of portland cement, sand, pea gravel and water mixed to fluid consistency so that it will have a slump of 8 to 11 inches. Grout is placed in the cells of hollow masonry units or between the wythes of solid units to bind the reinforcing steel and the masonry into a structural system. Additionally, grout provides:

1. More cross-sectional area allowing a grouted wall to support greater vertical and lateral shear forces than a non-grouted wall.
2. Added sound transmission resistance thus reducing the sound passing through the wall.
3. Increased fire resistance and an improved fire rating of the wall.
4. Improved energy storage capabilities of a wall.
5. Greater weight thus improving the overturning resistance of retaining walls.

Requirements for grout are given in ASTM C476, *Specification for Grout for Masonry*. An example of grouting a hollow unit wall is shown in Figure 1.15.

FIGURE 1.15 Grouting of hollow unit block wall.

1.4.2 TYPES OF GROUT

The IBC and MSJC Code identify two types of grout for masonry construction: fine grout and coarse grout. As their names imply, these two types of grouts differ primarily in the maximum allowable size of aggregates. The fineness or coarseness of the grout is selected based on the size of grout space and the height of the grout pour. Table 1.15, Grout Space Requirements, provides the maximum grout pour height based on cell or cavity size and grout type.

TABLE 1.15 Grout Space Requirements (MSJC Code Table 1.16.1, MSJC Specification Table 7)

Grout Type[1]	Maximum Grout Pour Height, (ft)	Minimum Width of Grout Space[2,3] (in.)	Minimum Grout Space Dimensions for Grouting Cells of Hollow Units,[3,4,5] (in. x in.)
Fine	1	3/4	1 1/2 x 2
Fine	5	2	2 x 3
Fine	12	2 1/2	2 1/2 x 3
Fine	24	3	3 x 3
Coarse	1	1 1/2	1 1/2 x 3
Coarse	5	2	2 1/2 x 3
Coarse	12	2 1/2	3 x 3
Coarse	24	3	3 x 4

1. Fine and course grouts are defined in ASTM C476.
2. For grouting between masonry wythes.
3. Grout space dimension is the clear dimension between any masonry protrusions and shall be increased by the diameters of the horizontal bars within the cross section of the grout space.
4. Area of vertical reinforcement shall not exceed 6 percent of the area of the grout space
5. Minimum grout space dimension for AAC masonry units shall be 3-in. x 3-in. or a 3-in. diameter cell.

1.4.2.1 FINE GROUT

Fine grout can be used where the grout space is small, narrow, or congested with reinforcing steel. When fine grout is used, there must be a clearance of 1/4 in. or more between the reinforcing steel and the masonry unit.

Typical proportions by volume for fine grout are:

- 1 part portland cement
- 2 1/4 to 3 parts sand
- Water for a slump of 8 to 11 in.
- Also, up to 1/10 part of hydrated lime or lime putty can be used

1.4.2.2 COARSE GROUT

Coarse grout may be used where the grout space for the grouted cavity of a double-wythe masonry construction is at least 1 1/2 inches in width horizontally, or where the minimum block cell dimension is 1 1/2 x 3 inches.

Although approved aggregates for grout (sand and pea gravel) are limited to a maximum size of 3/8 in., a coarse grout using 3/4 in. aggregate may be used if the grout space is especially wide, (8 in. or

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more horizontally). Larger size aggregates take up more volume, thus requiring less cement for an equivalent strength mix that used smaller aggregates. Larger aggregates also reduce the shrinkage of the grout and allow the slump of grout to be reduced to 7 or 8 in. for easier placement. Placing grout with 3/4 in. aggregate typically requires a concrete pump.

When coarse grout is made with pea gravel, there must be a minimum clearance of 1/2 in. between the reinforcing steel and the masonry unit. Accordingly, if coarse grout is made using larger sized aggregates, the clearance between the reinforcement and the masonry unit must be increased to approximately 1/4 in. more than the largest size aggregate.

The typical proportions by volume for coarse grout are:

- 1 part portland cement
- 2 1/4 to 3 parts sand
- 1 to 2 parts pea gravel
- Water for a slump of 8 to 11 in.
- Also, up to 1/10 part of hydrated lime may be used

Submittal requirements for grout are given in MSJC Specification Article 1.5 B.1.b as shown below:

MSJC Specification Article 1.5 B.1.b

1.5 B. Submit the following:

b. One of the following for each grout mix:

1) Mix designs indicating type and proportions of the ingredients according to the proportion requirements of ASTM C476, or
2) Mix designs and grout strength test performed in accordance with ASTM C476.

Grout space requirements are given in MSJC Code Table 1.16.1 and MSJC Specification Table 7. The table is one of the duplicated items between the Code and Specification as the requirements apply to both the designer and contractor.

Smaller grout spaces and higher grout lifts are possible provided the contractor provides a grout demonstration panel to show that an alternate system can effectively place grout in the wall and conform to code requirements.

1.4.3 SLUMP OF GROUT

Water content of grout is adjusted to provide fluidity (slump) allowing proper grout placement for various job conditions. The high slump allows grout to flow into openings and around steel reinforcement. Excess water in the grout is absorbed by the masonry units, reducing the apparently high water/cement ratio. Additionally the moist masonry aids in curing the grout.

Fluidity is measured by a slump cone test, as shown in Figure 1.16. The test consists of a 12 in. cone with openings on both ends. The grout sample is taken from the middle of a transit mixed load, not the initial 10% discharge and not the last 10% discharge. The cone is placed on a flat horizontal surface and is filled with grout, by placing the grout in the top of the cone and "rodding" to consolidate. The cone is then lifted straight up, and the grout is free to flow into a resting state. The difference in height between the top of the cone and the top of the grout, with the cone removed, is the slump. Both types of grout, fine and coarse, must contain enough water to provide a slump of 8 to 11 inches.

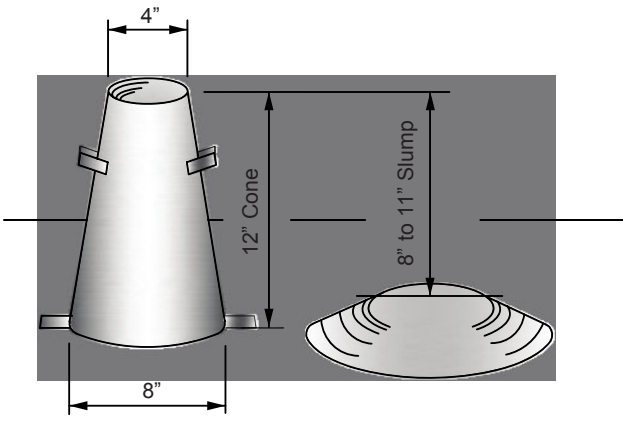

FIGURE 1.16 Slump cone and slump of grout.

1.4.4 PROPORTIONS

Grout ingredient proportions may be selected from Table 1.16, Grout Proportions by Volume. Proportions of the grout ingredients may also be determined by laboratory testing or field experience, if a satisfactory history of the grout's performance is available. Note that any grout performance history must be based on grout, mortar, and masonry units, which are similar to those intended for use on the

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new project. The use of 70% sand and 30% pea gravel requires six sacks of portland cement per cubic yard and results in a pumpable grout that provides the minimum strength of 2000 psi required by ASTM C476. Grout must have adequate strength to satisfy f'_m requirements and for sufficient bonding to the reinforcing steel and the masonry units. Without adequate bonding, stresses cannot properly transfer between the various materials. Adequate strength is also needed to assure that embedded anchor bolts will be anchored securely.

Experience has shown that grout proportions based on Table 1.16 are successful for normal load-bearing concrete masonry construction.

TABLE 1.16 Grout Proportions by Volume (IBC Table 2103.12; ASTM C476, Table 1)

Type	Parts by Volume of Portland Cement or Blended Cement	Parts by Volume of Hydrated Lime or Lime Putty	Aggregate Measured in a Damp, Loose Condition	
			Fine	Coarse
Fine Grout	1	0–1/10	2 1/4–3 times the sum of the volumes of the cementitious materials	–
Coarse Grout	1	0–1/10	2 1/4–3 times the sum of the volumes of the cementitious materials	1–2 times the sum of the volumes of the cementitious materials

1.4.4.1 Aggregates for Grout

Aggregates for grout should meet the requirements of ASTM C404, *Specification for Aggregates for Masonry Grout*. Grading of the aggregate should be in accordance with Table 1.17, Grading Requirements.

1.4.5 Mixing

Grout prepared at the jobsite should be mixed for a minimum of five minutes in order to assure thorough blending of all ingredients. Enough water must be used in the mixing process to achieve a high slump of 8 to 11 inches. Dry grout mixes which are blended at a factory should be mixed at the jobsite in a mechanical mixer for at least 5 minutes in order to obtain the desired consistency.

TABLE 1.17 Grading Requirements (ASTM C404, Table 1)

Amounts Finer than Each Laboratory Sieve (Square Openings), Percent by Weight					
Sieve Size	Fine Aggregate			Coarse Aggregate	
	Size No. 1	Size No. 2		Size No. 8	Size No. 89
		Natural	Manufactured		
1/2 – in.	–	–	–	100	100
3/8 – in.	100	–	–	85 to 100	90 to 100
No. 4	95 to 100	100	100	10 to 30	20 to 55
No. 8	80 to 100	95 to 100	95 to 100	0 to 10	5 to 30
No. 16	50 to 85	70 to 100	70 to 100	0 to 5	0 to 10
No. 30	25 to 60	40 to 75	40 to 75	–	0 to 5
No. 50	10 to 30	10 to 35	20 to 40	–	–
No. 100	2 to 10	2 to 15	10 to 25	–	–
No. 200	0 to 5	0 to 5	0 to 10	–	–

The MSJC Specification requires the following in Article 2.6 B:

MSJC Specification Article 2.6 B

2.6 B. *Grout*

1. Unless otherwise required, proportion and mix grout in accordance with the requirements of ASTM C476.
2. Unless otherwise required, mix grout to a consistency that has a slump between 8 and 11 in. (203 and 279 mm).

1.4.6 Grout Admixtures

Admixtures are any materials other than water, cement and aggregate which are added to the grout, either before or during mixing, in order to improve the properties of the fresh or hardened grout or to decrease its cost.

The four most common types of grout admixtures are:

1. Shrinkage Compensating Admixtures – Used to counteract the loss of water and the shrinkage of the cement by creating expansive gases in the grout.
2. Plasticizer Admixtures – Used to obtain the high slump required for grout without the use of excess water. By adding a plasticizer to a 4 in. slump grout mix, an 8 to 11 in. slump can be achieved.

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3. Cement Replacement Admixtures – Used to decrease the amount of cement in the grout without adversely affecting the compressive and bond strengths of the grout. Types C and F fly ash are by far the most common cement replacement admixtures. Current practice allows 15 to 20% of the portland cement by weight to be replaced with fly ash as long as the strength characteristics are maintained.

4. Accelerator admixtures – Used in cold weather construction to reduce the time that the wall must be protected from freezing. Accelerators decrease the setting time of the grout and speeds strength gain. Accelerators also increase the heat of hydration preventing the grout from freezing under most circumstances.

Careful consideration must be given prior to the use of all admixtures, since an admixture may adversely affect certain grout properties while improving the intended properties. Admixtures containing chloride and antifreeze liquids may not be used per ASTM C476 despite their benefits, since chlorides cause corrosion of the reinforcing steel. Some admixtures can reduce the compressive and bond strengths of the grout.

Similarly, care should be taken when using two or more admixtures in a grout batch since the combination of admixtures often produces unexpected results. Under all circumstances, information regarding laboratory and field performance of an admixture should be obtained from the manufacturer prior to use in grout. Additionally, MSJC Specification Article 2.2 requires approval of all grout admixtures prior to use.

1.4.7 GROUT STRENGTH REQUIREMENTS

According to ASTM C476, the grout can be specified either by proportions (shown in Table 1.16) or by compressive strength. When compressive strength is specified, the slump is to be 8 to 11 in., as determined by ASTM C143, and the compressive strength shall be a minimum of 2000 psi at 28 days when sampled and tested in accordance with ASTM C1019.

The required minimum compressive strength of 2000 psi is needed in order to achieve adequate bond of grout to the reinforcing steel, and to the masonry unit. This minimum value is satisfactory for masonry construction in which the specified design strength, f'_m, equals 1500 psi, and the masonry unit has a compressive strength of at least 1900 psi. The recommended compressive strength of the grout in concrete masonry construction is often taken as 1.25 to 1.40 times the design strength of the masonry assemblage, f'_m. An example is that 2000 psi grout is required for a masonry assemblage with a specified strength, f'_m, of 1500 psi; or a grout that is 1.33 times the specified strength. MSJC Specification Article 1.4 B.2, however, requires that the grout compressive strength equals or exceeds the specified compressive strength, f'_m, of masonry and that the grout compressive strength be not less than 2000 psi. This applies to both clay and concrete masonry.

For Strength Design procedures, MSJC Code Section 3.1.8.1.2 limits the specified strength of grout to 5,000 psi for concrete masonry and 6,000 psi for clay masonry. Actual grout strength should always equal or exceed the design strength, and may be higher than these prescribed design limits.

Normally, grout is specified at 2,000 psi minimum. When grout is delivered to the wall by means of a mechanical grout pump, there is sufficient cement content to achieve this minimum strength. The grout hose would plug if there was insufficient cement in the mix. For higher grout strength requirements, the designer may require testing to verify the grout strength.

If grout tests are required, the following schedule is suggested.

1. At the start of grouting operations, take one test per day for the first three days. The tests should consist of three specimens which are made in accordance with ASTM C1019, *Test Method for Sampling and Testing Grout*.

2. After the initial three tests, specimens for continuing quality control should be taken at least once each week. Additionally, specimens may be taken more frequently for every 25 cubic yards of grout, or for every 2500 square feet of wall, whichever comes first.

1.4.8 TESTING GROUT STRENGTH

In order to determine the compressive strength of grout, specimens, as defined in ASTM C1019, are made that will represent the hardened grout in the wall. The specimen is made in a mold consisting of masonry units identical to those being used in construction and at the same moisture condition as

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those units being laid. The units are arranged to form a space approximately 3 to 4 in. square and twice as high as it is wide (Figures 1.17 and 1.18).

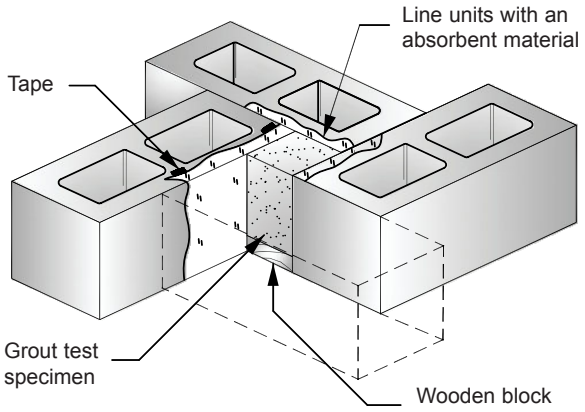

FIGURE 1.17 Typical arrangement for making a grout specimen with block.

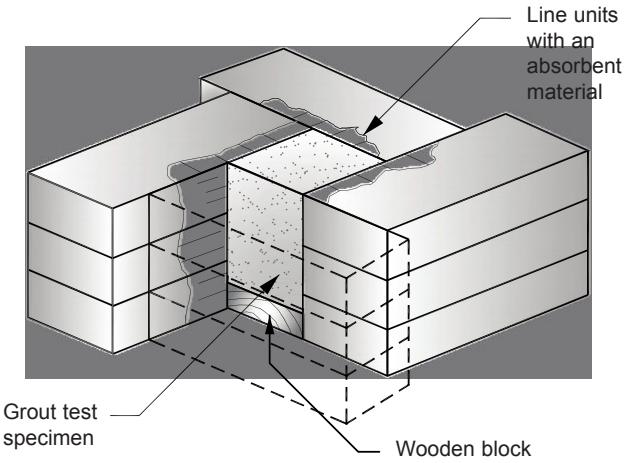

FIGURE 1.18 Typical arrangement for making a grout specimen with brick.

To prevent the grout from bonding to the masonry units, the space is lined with a permeable paper or porous separator, which still allows any excess water to be absorbed into the units. A paper towel does an excellent job.

The representative samples of the grout are placed in the molds, puddled and kept damp, and undisturbed for at least 24 hours. After the grout specimens have cured between 24 and 48 hours, the specimens are taken to a laboratory where they are placed in a fog room until tested.

1.4.9 METHODS OF GROUTING MASONRY WALLS

There are several methods of grouting masonry walls that will result in strong, homogeneous and satisfactory walls. The method selected is influenced by the type of masonry, the area and length of wall, the equipment available, and the experience of the contractor.

1.4.9.1 GROUT POUR AND LIFT

The total height of masonry to be grouted prior to the erection of additional masonry is called a grout pour. Grout is placed in increments called lifts. A grout lift is the height of grout placed in a single continuous operation prior to consolidation.

Though lifts may not exceed 5 ft in height, a grout pour may consist of several lifts. For example, if the wall is built 15 ft high, the total grout pour would be the entire 15 ft. For this situation, the contractor would place the grout in 3 lifts of 5 ft each. Alternately, a grout demonstration panel may be constructed to show grouting procedures, including higher lifts, which deviate from the code prescribed requirements. This provision is contained in MSJC Specification Articles 1.6 E and 3.5 F.

MSJC Specification Articles 1.6 E and 3.5 F

1.6 E. *Grout demonstration panel* — Prior to masonry construction, construct a grout demonstration panel if proposed grouting procedures, construction techniques, and grout space geometry do not conform to the requirements of Articles 3.5 C, 3.5 D, and 3.5 E.

3.5 F. *Alternate grout placement* — Place masonry units and grout using construction procedures employed in the accepted grout demonstration panel.

Currently MSJC Code limits a grout pour to a maximum height of 24 ft. For those cases where grout demonstration panels are constructed, the architect/engineer (A/E) should establish criteria for the panel to assure that the important elements of the masonry construction are represented in the demonstration panel. The A/E should also establish inspection procedures to verify grout placement procedures throughout the construction of the project. These procedures may include either non-destructive or destructive evaluation to confirm that adequate consolidation has been achieved.

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1.4.9.2 LOW LIFT AND HIGH LIFT GROUTING

Although the terms low lift and high lift grouting were deleted from the recent code editions, these terms are still commonly used when referring to grouting methods.

In general, low lift grouting may be used when the height of the grout pour is 5 ft or less. High lift grouting may be used only when cleanout holes are provided, and the height of the masonry wall prior to grouting exceeds 5 ft.

1.4.9.2.1 LOW LIFT GROUTING PROCEDURE

When the low lift grouting procedure is used, masonry walls may be built to a height of 5 feet. Because of this limited pour height which also allows for easy inspection of the walls, cleanout openings are not required.

For multi-wythe masonry walls, the wythes need to be tied together with wire ties or joint reinforcement whenever the grout pour height is more than 12 in. to prevent the wythes from bulging or blowing out (Figure 1.19). These ties should be spaced no more than 24 in. on center horizontally and 16 in. maximum vertically for running bond. For stacked bond construction ties must be spaced no more than 12 in. on center vertically.

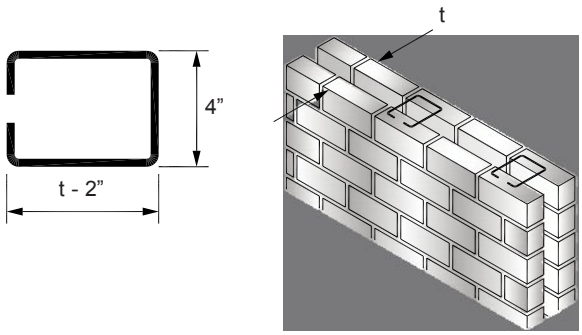

FIGURE 1.19 Ties for two wythe walls.

A single wythe wall consisting of hollow unit masonry does not require ties since cross-webs and end shells connect the face shells and resist bulging and blowouts.

Grout may not be placed until all the masonry units, ties, reinforcing steel and embedded anchor bolts are in place up to the top of the grout pour. Once these are in place the wall may be fully grouted. For grout pours 12 in. high or less, the grout may be consolidated by puddling with a stick such as a 1 x 2 in. piece of wood. However, grout pours in excess of 12 inches in height must be consolidated by means of a mechanical vibrator. The grout must also be reconsolidated after the excess water is absorbed by the units (usually after 3 to 5 minutes) to close any voids due to the water lost.

Masonry units, ties, reinforcing steel, and anchor bolts for the next pour may be placed once the grout has been thoroughly reconsolidated.

Horizontal construction joints should be formed between grout pours by stopping the grout pour 1 1/2 in. below the top of the masonry. Where bond beams occur, these joints may be reduced to 1/2 in. deep to allow sufficient grout above the horizontal reinforcing steel.

At the top of the wall, the grout should be placed flush with the masonry units.

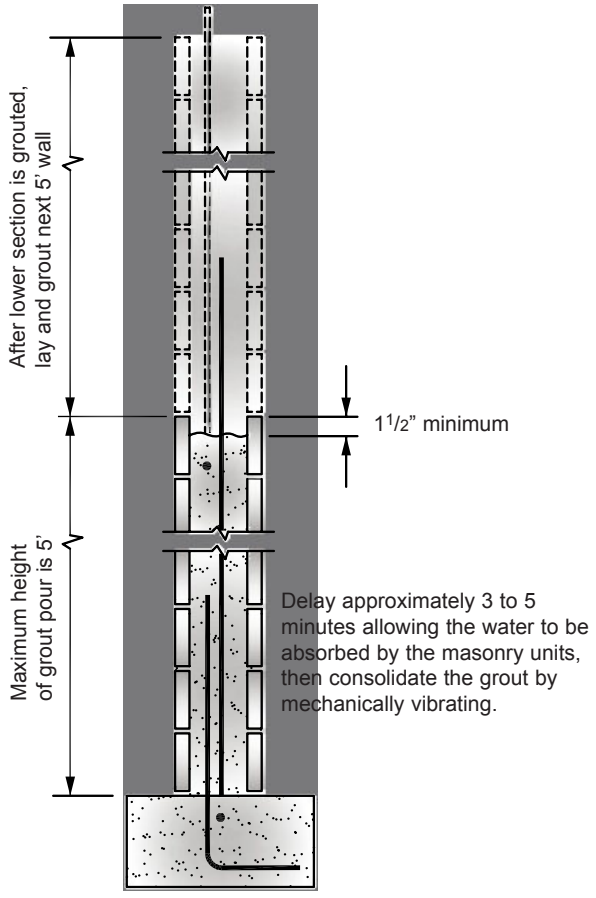

FIGURE 1.20 Low lift grouting, cleanouts not required.

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There is a provision in MSJC Specification Article 3.5 D allowing a single grout lift of up to 12 ft 8 in. provided all of the following items are met:

- Masonry wall has cured for at least 4 hours
- At all times during placement the grout slump is maintained between 10 and 11 inches.
- No intermediate bond beams (horizontal reinforcement) are obstructing vertical grout placement

1.4.9.2.2 High Lift Grouting Procedure

Grouting after a wall is constructed to its full height is often quite economical. This method allows the mason to continually lay masonry units without waiting for the walls to be grouted. High lift grouting procedures must be used when grout pours exceed 5 feet. Currently the maximum pour height the MSJC Code and Specification allows is 24 feet.

Cleanout openings must be provided in walls which have a grouted pour height exceeding 5 ft, in accordance to MSJC Specification Article 3.2 F. Cleanouts are usually located in the bottom course at every vertical bar. However, in solid grouted walls, cleanouts must be provided at no more than 32 in. on center, even if the reinforcing steel is spaced at a greater spacing (Figure 1.21).

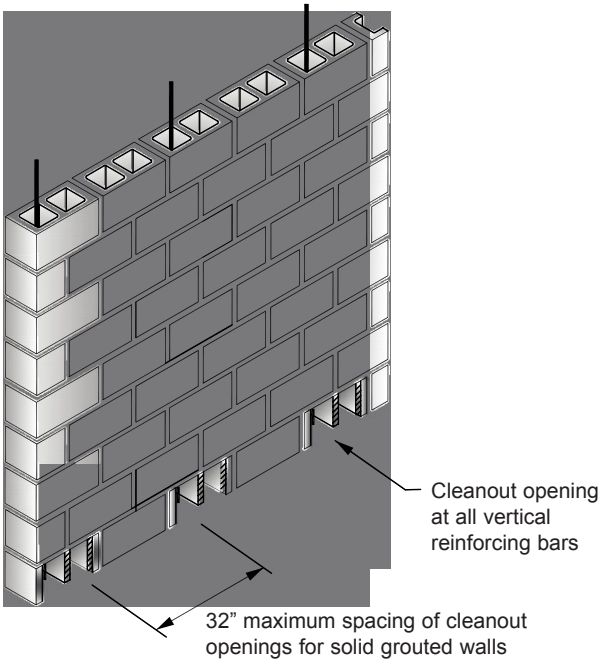

FIGURE 1.21 Maximum spacing of cleanout holes.

The purpose of the cleanouts is to allow the grout space to be cleaned prior to grouting. Cleanouts can also be used to verify reinforcement placement and tying. Cleanouts can be achieved by removing the exposed face shell for units in hollow unit grouted masonry, or removing individual units when grouting between wythes. The MSJC Specification Article 3.2 F requires that the cleanouts have an opening sufficient in size to permit removal of debris, and that the minimum opening dimension shall be 3 inches. After cleaning, the cleanouts are closed with closures braced sufficiently to resist grout pressure.

MSJC Specification Article 3.2 F is shown below:

MSJC Specification Article 3.2 F

3.2 F. *Cleanouts* — Provide cleanouts in the bottom course of masonry for each grout pour when the grout pour height exceeds 5 ft (1.52 m).

1. Construct cleanouts so that the space to be grouted can be cleaned and inspected. In solid grouted masonry, space cleanouts horizontally a maximum of 32 in. (813 mm) on center.
2. Construct cleanouts with an opening of sufficient size to permit removal of debris. The minimum opening dimension shall be 3 in. (76.2 mm).
3. After cleaning, close cleanouts with closures braced to resist grout pressure.

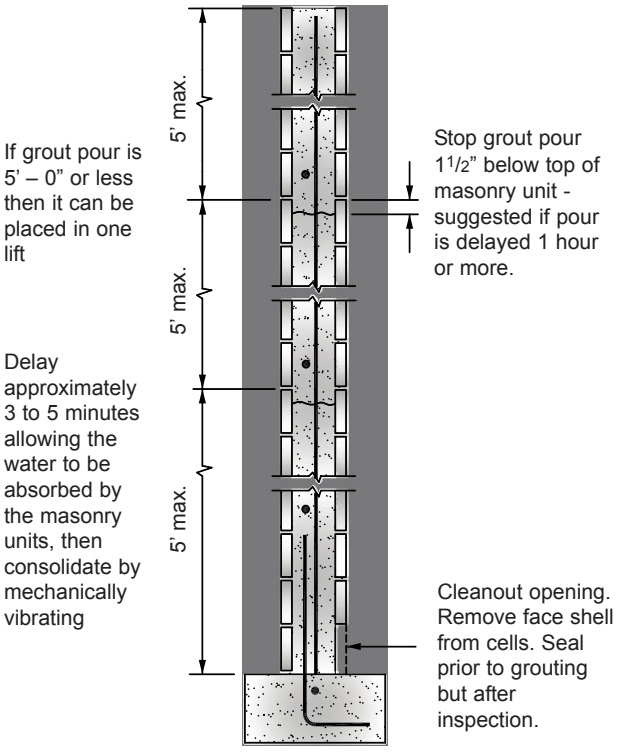

FIGURE 1.22 High lift grouting block wall.

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Two wythe masonry walls must be tied together with wire ties or joint reinforcement, as outlined in the low lift grouting section to prevent blowouts and bulging (Figure 1.23).

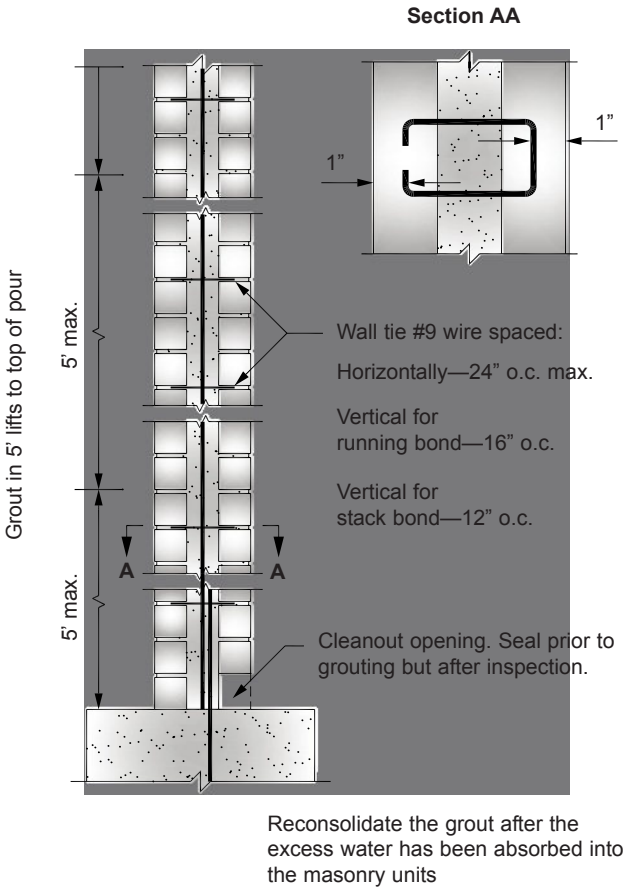

FIGURE 1.23 High lift method of grouting 2 wythe walls, with cleanout openings.

Grout lifts may be up to 5 ft high and must be mechanically consolidated. After a delay of typically 3 to 5 minutes, the grout should be reconsolidated to close any voids due to water loss.

Because of the fluidity of grout and the tendency of the aggregate to segregate, control barriers can be placed in multi-wythe walls to confine the flow of grout. These barriers, which are constructed with masonry units laid in the grout space, must extend the full height of the grout pour. Traditional spacing of these barriers has been no more than 30 ft on center. The full height of the wall between control barriers should be grouted in one day.

At the bottom of the wall the grout space may be covered with a layer of loose sand during construction to prevent mortar droppings from sticking to the foundations. The mortar droppings and sand are then removed from the grout space by blowing it out, washing it out, or cleaning it out by hand.

Once the foundation has been cleaned and inspected, cleanout holes may be sealed with a masonry unit, a face shell, or a form board which is then braced to resist the pressure of the poured grout.

1.4.9.3 CONSOLIDATION OF GROUT

Grout must be consolidated just like concrete. Consolidation eliminates voids and causes grout to flow around the reinforcement and into small openings or voids.

Consolidation may be performed using a puddle stick if the lifts are not higher than 12 inches. Lifts heights greater than 12 in. however, must be consolidated by mechanical vibrators. As there is generally only a small volume of grout to be consolidated in a cell or grout space, the mechanical vibrator need only be used for a few seconds in any location. Excessive vibration increases the possibility of blowing out face shells or dislodging masonry units. Additionally, the grout must be reconsolidated before plasticity of grout is lost.

1.4.10 SELF-CONSOLIDATING GROUT

A new product currently under development and limited use is Self–Consolidating Grout. Self-consolidating grout has properties that can eliminate the need to mechanically vibrate the grout, creating a savings in time, labor, and equipment. Also self-consolidating grout may allow higher lifts during the grout pour. The efficiency of not consolidating and reconsolidating grout without compromising structural integrity makes masonry more economical. The fluidity of self-consolidating grout relies on plasticizing admixtures, but must be stable. This material is not measured in slump, but in spread as depicted in Figure 1.24.

FIGURE 1.24 Self-consolidating grout spread.

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1.4.11 GROUT DEMONSTRATION PANELS

MSJC Specification Article 1.6 E now provides for a "grout demonstration panel" which allows the contractor to build a panel to show that a higher grout pour height can be obtained and still yet provide for proper consolidation of the grout. With approval, some alternate methods may be possible.

1.4.12 GROUT FOR AAC MASONRY

Grout used for AAC masonry construction is provided in the MSJC Specification Article 3.5 G, as follows:

MSJC Specification Article 3.5 G

3.5 G. *Grout for AAC masonry* — Use grout conforming to ASTM C476. Wet AAC masonry thoroughly before grouting to ensure that the grout flows to completely fill the space to be grouted. Grout slump shall be between 8 in. and 11 in. (203 and 279 mm) when determined in accordance with ASTM C143.

1.5 REINFORCING STEEL

1.5.1 GENERAL

Reinforcing steel in masonry has been used extensively in the West Coast since the 1930's, revitalizing the masonry industry in earthquake prone areas. Reinforcing steel extends the characteristics of ductility, toughness and energy absorption that is necessary in structures subjected to the dynamic forces of earthquakes.

Reinforced masonry performs well because the materials; steel, masonry, grout, and mortar, work together as a single structural unit. The temperature coefficient for steel, mortar, grout, and the masonry units are very similar. This similarity of thermal coefficients allows the different component materials to act together through normal temperature ranges. Disruptive stresses, which would destroy the bond between these materials and prevent force transfer, are not created at the interface between the steel and the grout.

Structures subjected to severe lateral dynamic loads such as earthquakes must be capable of providing the necessary strength or energy absorbing capacity and ductility to withstand these forces. Reinforcing steel serves to resist shear and tensile forces generated by the dynamic loads. It can also provide sufficient ductility to the masonry structure so that the structure can sustain load reversals beyond the capability of plain, unreinforced masonry.

In order for the reinforcing steel to provide adequate ductility and strength, placement of the reinforcing steel is of prime importance in providing a continuous load path throughout the structure. The engineer must pay special attention to reinforcing steel details to ensure continuity. The following items must be provided:

1. The proper size and amount of reinforcement which complies with the limited minimum and maximum percentages of reinforcement and other code requirements.
2. The minimum required reinforcement protection (cover).
3. The proper spacing of longitudinal and transversal reinforcement.
4. Sufficient anchorage of flexural and shear reinforcing bars.
5. Adequate lapping of the reinforcing bars.
6. Sufficient stirrups, ties, metal plates, spirals, etc., in order to provide confinement.

1.5.2 TYPES OF REINFORCEMENT

1.5.2.1 GENERAL REINFORCEMENT

MSJC Code Section 1.13.2 provides reinforcement that is used in design of masonry structural systems.

MSJC Code Section 1.13.2

1.13.2 *Size of reinforcement*

1.13.2.1 The maximum size of reinforcement used in masonry shall be No. 11 (M #36).

1.13.2.2 The diameter of reinforcement shall not exceed one-half the least clear dimension of the cell, bond beam, or collar joint in which it is placed. (See Section 1.16.1).

1.13.2.3 Longitudinal and cross wires of joint reinforcement shall have a minimum wire size of W1.1 (MW7) and a maximum wire size of one-half the joint thickness.

The Strength Design provisions of MSJC Code contain further limitations on reinforcing steel.

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MSJC Code Section 3.3.3.1

3.3.3.1 *Reinforcing bar size limitations* — Reinforcing bars used in masonry shall not be larger than No. 9 (M#29). The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimension of the cell, course, or collar joint in which the bar is placed. The area of reinforcing bars placed in a cell or in a course of hollow unit construction shall not exceed 4 percent of the cell area.

1.5.2.2 REINFORCING BARS

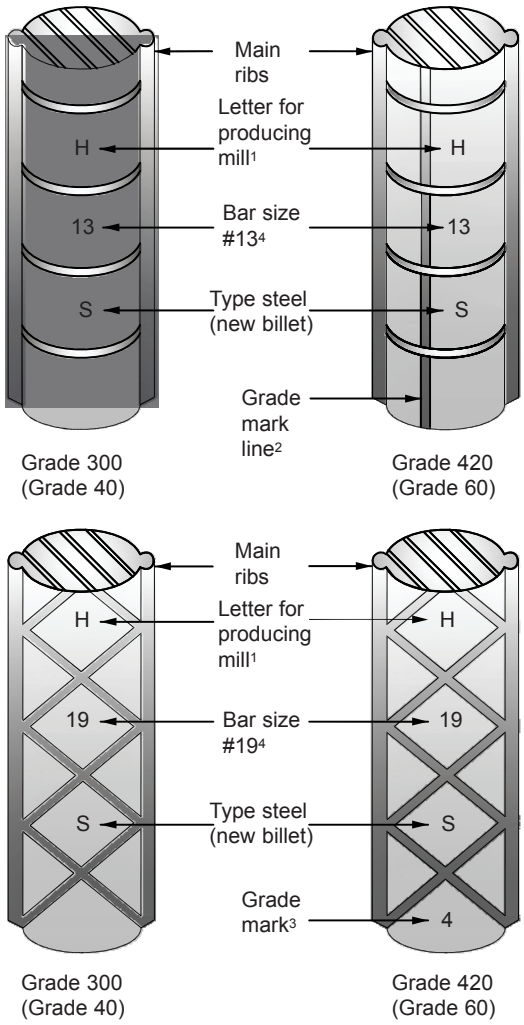
For reinforced masonry construction, deformed bars range in size from a minimum #3 (3/8 in. diameter) to a maximum #11 (1 3/8 in. diameter), however, the upper limit for masonry designed by Strength Design is #9 (1 1/8 in. diameter). Also, the reinforcing steel or reinforcing wire used in masonry must conform to ASTM A82, A185, A496, A497, A580, A615, A706, A767, A775, A951 or A996 which specify applicable physical characteristics.

ASTM A615 and A996 cover reinforcing steel manufactured from billet, rail and axle steel respectively. ASTM A706, A767 and A775 are generally not applicable since they cover low alloy, zinc-coated and epoxy-coated reinforcing steel which are currently seldom used in masonry construction.

Reinforcing steel may be either Grade 40 (Metric Grade 300), with a minimum yield strength of 40,000 psi or Grade 60 (Metric Grade 420) minimum yield strength of 60,000 psi. Grade 60 steel is furnished in all sizes, while Grade 40 steel bars are normally only available in #3, #4, #5 and #6 sizes. If Grade 40 steel is required, special provisions may be required to assure delivery. Good practice consists of determining the grade of steel and sizes available in the area where the project is to be built.

The identification marks are shown (Figure 1.25) in the following order:

1st – Producing Mill (usually an initial).
2nd – Bar Size Number.
3rd – Type of reinforcement (Type S for New Billet, A for Axle, I for Rail, W for Low Alloy).
4th – Grade of reinforcement for Grade 60 steel (grade is shown as a marked 4 (Metric Designation for Grade 420) or one (1) grade mark line. The grade mark line is smaller and between the two main longitudinal ribs which are on opposite sides of all U.S. made bars).



Bar Size # (mm)								
3 (10)	4 (13)	5 (16)	6 (19)	7 (22)	8 (25)	9 (29)	10 (32)	11 (36)

1. Bar identification marks may also be oriented to read horizontally (at 90° to those illustrated above).
2. Grade mark lines must be continued at least five deformation spaces.
3. Grade mark numbers may be placed within separate consecutive deformation spaces to read vertically or horizontally.
4. #13 = 1/2" bar and #19 = 3/4" bar.

Note:
Grade 520 (75) steel also available for masonry.
Bar size markings are given in metric which is indicated on reinforcement supplied for masonry use.

FIGURE 1.25 Identification marks, line system of grade marks.

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1.5.2.3 JOINT REINFORCEMENT

When high strength steel wire fabricated in ladder or truss type configurations is placed in the bed joints to reinforce the wall in the horizontal directions, it is called joint reinforcement.

The most common uses of joint reinforcement are:

1. to control shrinkage cracking in concrete masonry walls.
2. to provide part or all of the minimum steel required.
3. to function as designed reinforcement that resists forces in the masonry, such as tension and shear.
4. to act as a continuous tie system for veneer and cavity walls.

Joint reinforcement must meet the requirements of ASTM A951, *Specification for Masonry Joint Reinforcements*. Examples of joint reinforcement are shown in Figures 1.26 and 1.27. See Chapter 7 of this book for additional information on joint reinforcement.

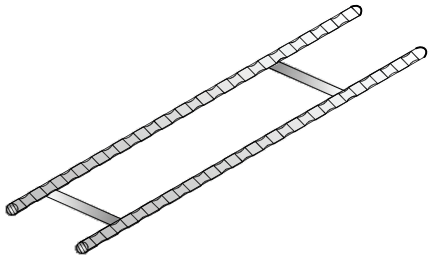

FIGURE 1.26 Ladder type joint reinforcement.

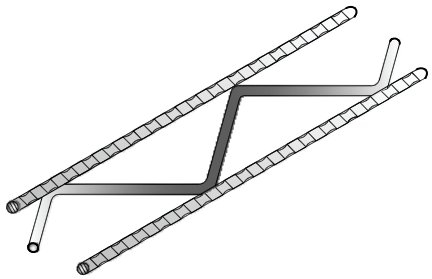

FIGURE 1.27 Truss type joint reinforcement.

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1.6 QUESTIONS AND PROBLEMS

1-1 What three ASTM specifications give the requirements for unit clay masonry?

1-2 What is the range of firing temperatures for building brick and for face brick?

1-3 State the three stages of fusing clay and describe each stage.

1-4 What is the approximate time required for the firing of brick in a kiln?

1-5 What is the difference between a solid clay unit and a hollow clay unit? Can solid units have voids? If so, what is the maximum percentage of voids that is permissible? What are the minimum and maximum percentages of voids in hollow units?

1-6 State the three grades of building brick and describe each grade.

1-7 Describe each type of face brick.

1-8 What are the grades of hollow brick and how are they classified?

1-9 Describe each type of hollow brick.

1-10 What are the three basic physical requirements for clay brick?

1-11 What is the significance of the water absorption rate and the saturation coefficient?

1-12 What is the initial rate of absorption and how does it compare to water absorption?

1-13 Why should clay brick have the proper moisture content before laying? Explain the consequences if it is too wet or too dry.

1-14 Describe each grade of concrete brick. What are the minimum strength requirements for each grade?

1-15 What are the types of concrete brick and what is the difference between them?

1-16 What are the weight ranges for light weight, medium weight and normal weight concrete masonry units?

1-17 A wall is constructed with normal weight hollow concrete masonry units. What is the weight of the wall if it is made of nominal 8 in. units and is grouted at 48 in. o.c.? Compare this to a 12 in. solid grouted wall.

1-18 What is meant by the equivalent solid thickness of a hollow unit?

1-19 A concrete block unit is made from material which weighs 110 pounds per cubic foot. What is its weight classification? If it is made from material which weighs 127 pound per cubic foot, what is its weight classification?

1-20 What is the purpose of mortar? Give six reasons for using mortar.

1-21 Give a classification and description for each type of mortar based upon strength properties.

1-22 What types of mortar are required in Seismic Design Categories D, E, and F for structural masonry?

1-23 What are standard proportions for Type M, S, N mortar using portland cement and lime?

1-24 What types of cement may be used in mortar?

1-25 What are the benefits of using hydrated lime in a mortar mix? What are the disadvantages?

1-26 What is the significance of proper grading of sand for masonry mortar? What ASTM specification gives the requirements for mortar sand?

1-27 Are coloring agents for a mortar considered admixtures?

1-28 How long should mortar generally be mixed? What is the effect of over-mixing mortar? What is retempering and how often may mortar be retempered?

1-29 Name and describe four different mortar joint types.

1-30 What is grout? What are its ingredients?

1-31 Give five reasons for using grout.

1-32 What is fine grout and coarse grout?

1-33 What are the normal proportions for fine grout? For coarse grout?

1-34 What is the average slump for grout to be used in a 6 in. CMU masonry wall? What should its minimum strength be for fine grout or coarse grout?

1-35 What should the range of slumps be for grout? Why is it allowed to be so fluid?

1-36 Name three admixtures for grout and the reasons to use them.

1-37 Describe the method of making a grout test specimen.

1-38 Describe low-lift grouting.

1-39 Describe high-lift grouting.

1-40 Why must grout be consolidated?

1-41 Sketch a reinforcing bar and show its identification marks.

1-42 What are the advantages of using joint reinforcement?

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CHAPTER 2

MASONRY ASSEMBLAGE STRENGTHS AND PROPERTIES

2.1 GENERAL

Masonry assemblies are comprised of the masonry unit, mortar and grout. Grouted masonry has more compressive, flexural, and shear strength than ungrouted masonry; therefore, this characteristic provides for increased structural capacities.

The ultimate compressive strength of the masonry assembly may be given as the symbol, f'_{mu}, to distinguish it from the specified compressive strength, f'_m. For autoclaved aerated concrete, the specified compressive strength is designated as f'_{AAC}.

To obtain the ultimate compressive strength value, f'_{mu}, prisms are constructed and tested in accordance with ASTM C1314 *Standard Test Methods for Compressive Strength of Masonry Prisms*. A prism is a test specimen made up of masonry units, mortar and sometimes grout. The masonry units are laid up in stack bond and tested in compression. From the results of the prism test, a value for f'_m can be confidently satisfied.

2.2 VERIFICATION OF, f'_m, THE SPECIFIED DESIGN STRENGTH

The required or specified value, f'_m, is used as the basis for structural engineering masonry design and must be obtained or verified in accordance with prescribed code requirements.

The 2006 IBC and 2005 MSJC Specification provide the following methods to verify the specified strength of the masonry assembly.

1. *Masonry Prism Testing* – In accordance with IBC Section 2105.2.2.2 or MSJC Specification Article 1.4 B.3
2. *Unit Strength Method* – In accordance with IBC Section 2105.2.2.1 or MSJC Specification Article 1.4 B.2
3. *Testing Prisms from Constructed Masonry* – In accordance with IBC Section 2105.3

The frequency for determination of f'_m is based upon the level of inspection.

IBC Level 1 Quality Assurance requires verification of f'_m prior to the start of construction only; where Level 2 Quality Assurance requires verification of f'_m prior to start of construction and every 5,000 square feet of wall area. More information on levels of inspection is provided at the end of this chapter. The MSJC Code and Specification also contain levels of Quality Assurance, termed A, B and C. MSJC Code Level B and IBC Level 1 are equivalent. Similarly, MSJC Code Level C is equivalent to IBC Level 2. Since MSJC Code Quality Assurance Level A is so minimal, there is no corresponding Quality Assurance Level in the IBC.

2.2.1 VERIFICATION BY PRISM TESTS

2.2.1.1 PRISM TESTING

To verify that the masonry element meets or exceeds the design strength, prisms may be constructed and tested in accordance with ASTM C1314. Additional consideration may be given to the relative strengths of masonry materials making up the wall.

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IBC Section 2105.2.2.2.1

2105.2.2.2.1 General. The compressive strength of clay and concrete masonry shall be determined by the prism test method:

1. Where specified in the construction documents.
2. Where masonry does not meet the requirements for application of the unit strength method in Section 2105.2.2.1

2105.2.2.2.2 Number of prisms per test. A prism test shall consist of three prisms constructed and tested in accordance with ASTM C1314.

Prism testing is primarily used when the specified design strength, f'_m, is required to be higher than 1500 psi for concrete masonry, or 2600 psi for clay masonry. If prism testing is prescribed, then, prior to construction, adequate lead time is required to prepare prisms since retesting could be required. For constructing the prescribed prism samples and subsequent testing, the IBC and the MSJC Specification refer to ASTM C1314. The strength developed depends on many factors, including workmanship and materials. Figure 2.1 shows a typical prism test utilizing a single length, two unit-high assembly, although other arrangements are acceptable, such as cut down units or multi-wythe assemblies.

FIGURE 2.1 Masonry prism test.

Masonry prisms are built one unit or less in length and in a stack bond arrangement. The construction of a prism with running bond would introduce head joints in the specimen forming a vertical plane of weakness, allowing splitting to occur at a much lower value than the actual strength of the wall. In a wall laid up in running bond, the masonry units are confined by the total wall and the effect of the head joints is significantly diminished.

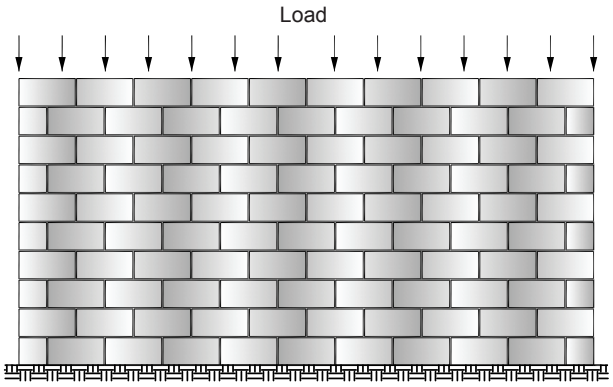

FIGURE 2.2 Masonry units are confined in the wall and cannot move laterally in plane of wall.

Figure 2.3 illustrates various examples of test failures in a running bond specimen. The small size of the specimens do not represent wall loading distribution.

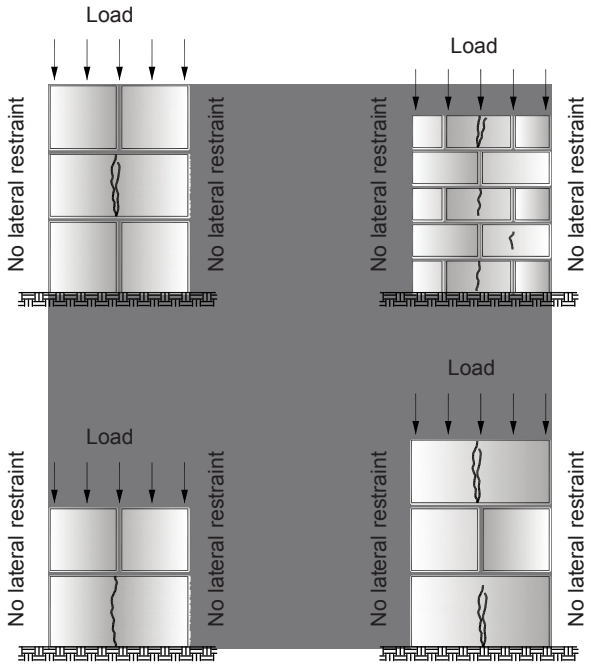

FIGURE 2.3 Unrestrained running bond prisms result in low strength not representative of the strength of the wall.

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When large masonry prisms are tested in compression, the bearing area of the spherical bearing head block of the testing machine may not be large enough to cover the full area of the specimen. In this case, a solid steel plate should be placed between the bearing block and the specimen so that the entire area of the specimen is covered. The solid plate must have a thickness at least equal to the distance from the edge of the spherical bearing to the most distant corner of the specimen. The recommended top plate should be a minimum of 3 1/2 in. thick, however, in some cases, the stiffness of the loading apparatus and the testing machinery must be taken into account, particularly if one is attempting to achieve the complete stress-strain relationships. The Annex to ASTM C1314 provides criteria for determining the plate thickness for compression testing.

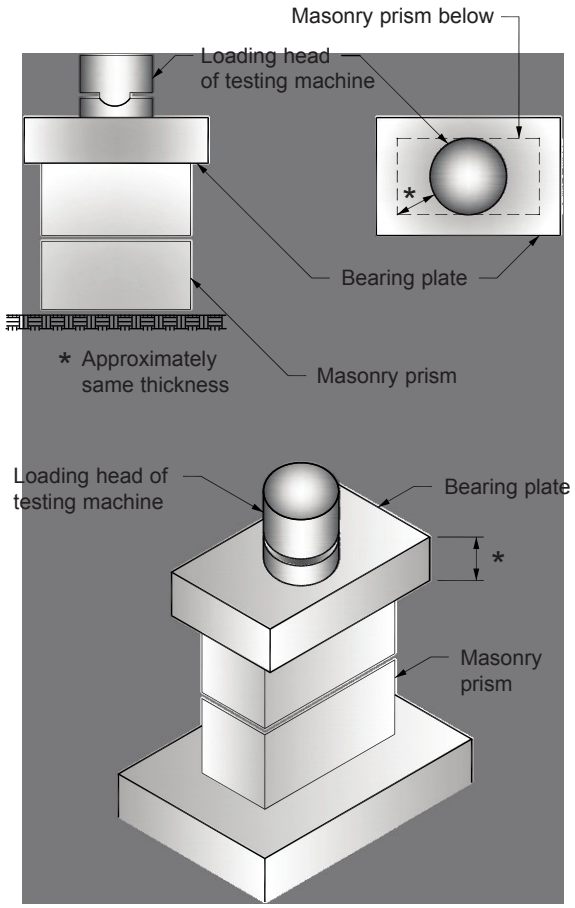

FIGURE 2.4 Masonry prism test plate configuration.

2.2.1.2 CONSTRUCTION OF PRISMS

Prisms are made using the actual materials that will be used in the construction of the wall. The brick or hollow units, sand and cement, mortar mix, and grout (if needed) materials that are to be used in the structure should be used in the construction of the prisms. In the prisms, the mortar bedding, the thickness, the grouting and the condition of the units should be the same as in the structure, except that no reinforcement should be included. Notwithstanding the mortar joint finish specified, masonry prisms are constructed with flush-cut mortar joints. Prisms are built in stack-bond configuration.

Prisms are to be constructed on a level base and in an opened plastic moisture-tight bag, large enough to enclose the completed prism. The prisms need to be constructed in a location where they will remain undisturbed until moved or transported for testing. Where the cross sections of units vary due to architectural surfaces or taper of the cells, the same placement should be used as specified in the project construction.

The length of masonry prisms can be reduced by saw cutting. Prisms composed of regular shaped hollow units should have at least one complete cell with one full-width cross web on each end. Irregular-shaped units for prisms can be cut to obtain as symmetrical a cross section as possible. The minimum allowable length of saw-cut prisms is 4 inches. Prisms should be a minimum of two units in height, and cannot be less than 1.3 times nor more than 5.0 times the least thickness.

When the project construction is solid grouted, the prisms are solid grouted. The grout should be placed between 24 and 48 hours following construction of the prism. Consolidation of grout should be the same as that used in the construction. After reconsolidation and settlement due to water loss, additional grout is placed in the prism to level off the top. When open-ended units are used, masonry units may be used to confine the grout during placement.

When the project construction is partially grouted, two sets of prisms are constructed; one set is grouted solid and the other set remains ungrouted.

Walls of ungrouted multi-wythe masonry having different units or different mortar require construction of separate prisms for each wythe of masonry. ASTM C1314 suggests, by a graphic depiction, that grouted multi-wythe masonry be constructed as a single specimen.

Prisms should be left undisturbed in the plastic bags for at least 48 hours following construction and grouting.

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2.2.1.3 STANDARD PRISM TESTS

The provisions of the IBC and the MSJC Specification are based on ASTM Standard C1314 which requires a prism two-units high with at least one mortar joint, as shown in Figure 2.5 and 2.6.

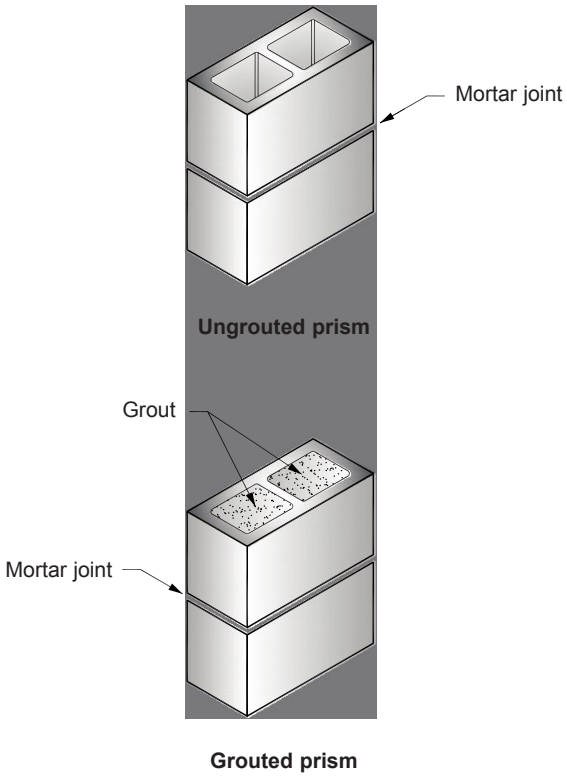
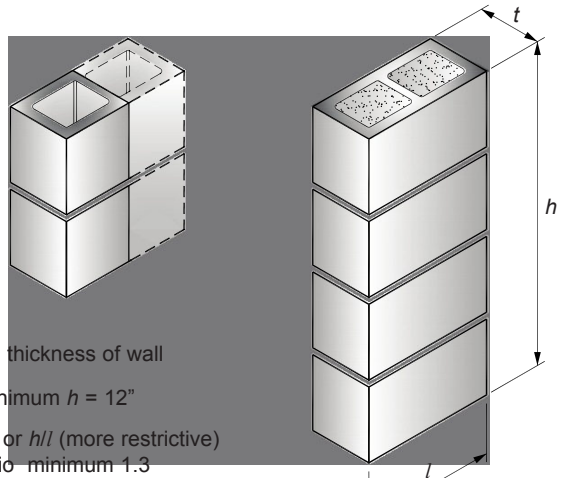


FIGURE 2.5 Masonry prism construction for typical hollow and grouted specimens.



t = thickness of wall

Minimum *h* = 12”

h/*t* or *h*/*l* (more restrictive) ratio minimum 1.3 maximum 5.0

l = length of unit or part of a unit including at least one cell and adjacent web but not less than 4”

FIGURE 2.6 Size of prism specimen.

Reduced length specimens may be needed for testing of higher strength units. The capacity of some compression testing machines may be limited. Also, reduced length prisms are easier to handle and transport. The height-to-thickness ratios can vary from 1.3 to 5.0.

In accordance with ASTM C1314, a set of three masonry prisms should be made and tested prior to the start of construction of the actual wall so that the required f'_m can be verified for the actual materials. The prisms are tested at 28 days and/or at designated test ages. Prisms tested at other (than 28 day) ages require additional specimens for comparison testing.

Capping and testing a full size nominal 12 x 8 x 16 in. masonry unit prism may be difficult, particularly for high strength clay or concrete masonry. For these more difficult cases, approximately half length units may be made into a prism and tested. The half length unit should include the full thickness of the middle cross web, as shown in Figure 2.7.

The "half" specimen, as shown in Figure 2.7, would be approximately 12 in. wide by 9 in. long. It can be made, transported, capped and tested much easier than a full unit. The results may be more consistent with significantly less chance of eccentric loading and uneven capping.

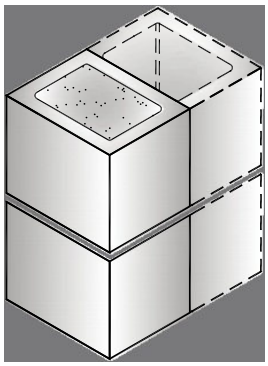


FIGURE 2.7 Prism of half hollow masonry unit.

Additionally, smaller prisms do not require special testing machines while full size high strength masonry unit prisms often require testing equipment with a capacity in excess of 750,000 pounds. Examples of various sizes and configurations of prisms are shown in Figure 2.8.

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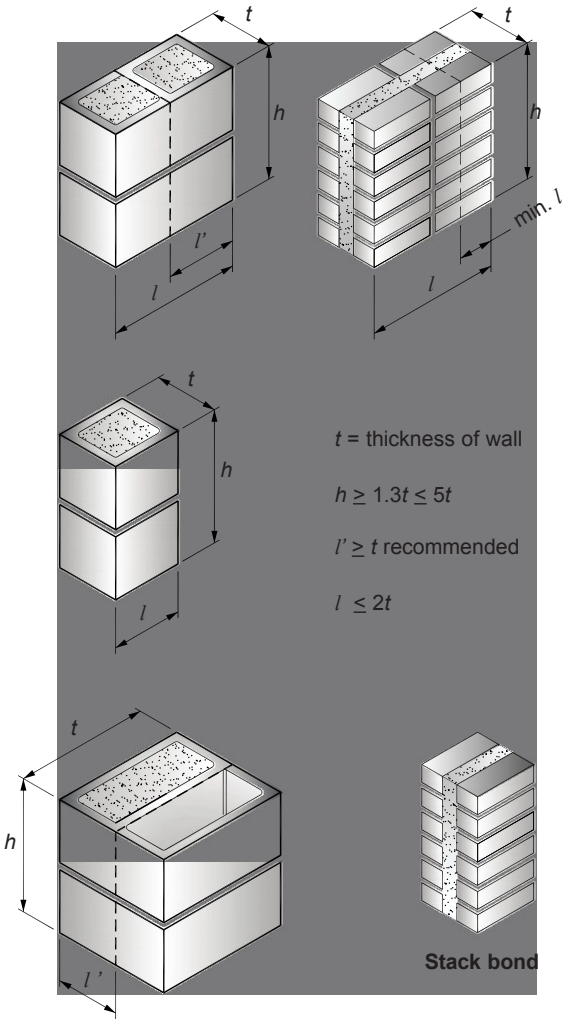

FIGURE 2.8 Sizes of masonry prisms.

For example, brick with a unit strength of 14,000 psi may have an assumed f'_m = 5,300 psi, however, properly constructed prisms should result in greater strengths. A grouted two-wythe prism 9 in. thick, 18 in. high and 11 1/2 in. long (one unit) would require a testing machine with a capacity of at least 550,000 pounds. However, if the prism were only 9 in. in length, a 500,000 pound capacity testing machine could easily verify the required compressive strength.

Seven-day tests have historically been used when a relationship between the seven-day and the 28-day strength has been established. When seven-day tests are made, extrapolation could determine whether projected 28-day tests results will be satisfactory and meet the 28-day strength requirement.

2.2.1.4 TEST RESULTS

The compressive strength of the masonry prisms determined in accordance with ASTM C1314 is the ultimate compressive strength, f'_{mu}, (termed f'_{mt} in ASTM C1314) and the average for each set of prisms must equal or exceed the specified compressive strength, f'_m. The prism test strength is multiplied by the prism height-to-thickness correction factor. This correction factor is based on the ratio of h_p/t_p, where h_p is the measured height of the prism and t_p is the least actual lateral dimension of the prism. Test results are multiplied by the correction factors given in Table 2.1 apply to either concrete or clay masonry prisms.

TABLE 2.1 Prism Correction Factor (ASTM C1314, Table 1)

Prisms h_p/t_p[1]	1.30	1.50	2.00	2.50	3.00	4.00	5.00
Correction Factor	0.75	0.86	1.00	1.04	1.07	1.15	1.22

1. h_p/t_p ratio of prism height to least actual lateral dimension of prism.

The relationships between h and t for the more common typical masonry prisms are shown in Figure 2.9 for clay and concrete masonry construction.

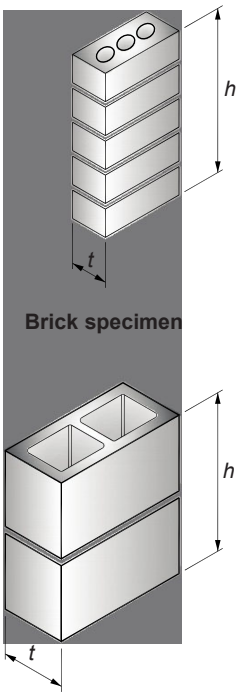

FIGURE 2.9 Typical test specimens.

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2.2.1.5 STRENGTH OF COMPONENT MATERIALS

When the compression strength of the masonry assemblage, f'_m, is specified, the component materials of grout and masonry units should be stronger than the specified strength.

Individual material strengths exceeding the design strength must be specified in order to obtain a satisfactory strength of the wall because there are differences in the Modulus of Elasticity and the Poisson's Ratio between the masonry units and the grout. These differences cause a reduction in the strength of the total masonry assembly and must be compensated for by starting out with higher strength grout and masonry units. In addition, the workmanship in the construction of the prisms and the masonry walls has a significant influence on the strength of the masonry system.

2.2.1.5.1 HOLLOW CONCRETE MASONRY

The specified strength is the minimum strength that must be obtained in the wall. For concrete block systems, a suggested strength of the masonry unit and grout is a minimum of 25 to 40 percent more than the specified strength. This suggestion may be adjusted if the strength relationship has been established between the materials used and the prism strength. Accordingly, for a specified f'_m of 3000 psi, the concrete masonry units and grout should have a strength of at least 3700 to 4200 psi. When the masonry unit and the grout are combined and tested, the strength obtained for the prism, f'_m, should be at least 3000 psi.

When specifying masonry units, specify a minimum strength only, not a range of strengths. This minimum strength would be the average of three units with no single unit less than 20% below the specified minimum unit strength.

2.2.1.5.2 CLAY BRICK AND HOLLOW BRICK MASONRY

Clay brick and hollow brick are generally high strength clays that are fired and fused together to create a strong body or masonry unit. The strength of units depends on the clays or shale used, the firing temperature and the duration of firing.

The strength of clay units is normally at least one-third more than the specified f'_m. Grout should be mixed to the proportions provided in Section 1.4.2 or prisms may be made to determine the required strength of grout to obtain the f'_m strength.

2.2.1.5.3 MORTAR

As specified in the MSJC Code Section 1.14.6.6, the seismic provisions for Seismic Design Categories D, E, or F require that only Type S or Type M mortar should be used in components that are a part of the lateral load-resisting system and also that masonry cement is not allowed in these seismic categories. Because of the relatively thin mortar joints, Type S or M mortar used in masonry will have an in-place strength of 3000 psi or more. The *h/t* ratio of the mortar in the joint is very small, enabling the mortar to exhibit strengths far higher than the strengths obtained from cube tests of mortar. The *h/t* ratio of the mortar is depicted in Figure 2.10.

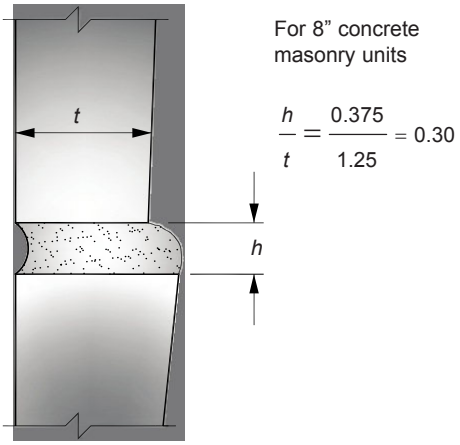

FIGURE 2.10 ***h/t*** **for confined mortar bed joint.**

When the compressive strength of mortar is desired, mortar should be specified by property specifications only. Any testing that is done for field properties is to be done in accordance with ASTM C780, whereas any testing to determine the mix properties for laboratory or research purposes is done in accordance with ASTM C270.

2.2.1.5.4 GROUT

As prescribed in IBC Section 2103.12, the requirements for grout are given in ASTM C476. When required, grout strengths are determined by making grout specimens in accordance with ASTM C1019. The minimum strength must be at least 2000 psi and grout should not be less than the strength of the units for concrete masonry construction. Additional information on grout testing is contained in ASTM C1019.

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2.2.2 VERIFICATION BY UNIT STRENGTH METHOD

2.2.2.1 SELECTION OF f'_m FROM CODE TABLES

The specified compressive strength of masonry, f'_m, may be selected from tables that are based on the strength of the masonry unit and mortar used. These tables are conservative and higher values may be obtained by conducting prism tests. The specified compressive strength of masonry, f'_m, for design is usually taken as the "net area compressive strength" as tabulated in the tables.

IBC Section 2105.2.2.1

2105.2.2.1.1 Clay masonry. The compressive strength of masonry shall be determined based on the strength of the units and the type of mortar specified using Table 2105.2.2.1.1, provided:

1. Units conform to ASTM C62, ASTM C216 or ASTM C652 and are sampled and tested in accordance with ASTM C67.
2. Thickness of bed joints does not exceed 5/8 inch (15.9 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   3.1. Grout conforms to ASTM C476.
   3.2. Minimum grout compressive strength equals or exceeds f'_m but not less than 2,000 psi (13.79 MPa). The compressive strength of grout shall be determined in accordance with ASTM C1019.

2105.2.2.1.2 Concrete masonry. The compressive strength of masonry shall be determined based on the strength of the unit and type of mortar specified using Table 2105.2.2.1.2, provided:

1. Units conform to ASTM C55 or ASTM C90 and are sampled and tested in accordance with ASTM C140.
2. Thickness of bed joints does not exceed 5/8 inch (15.9 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   3.1. Grout conforms to ASTM C476.
   3.2. Minimum grout compressive strength equals or exceeds f'_m but not less than 2,000 psi (13.79 MPa). The compressive strength of grout shall be determined in accordance with ASTM C1019.

Depending on the level of Quality Assurance specified, the compressive strength of masonry, f'_m, and mortar and grout proportions may require verification prior to the start of the project and every 5,000 square feet of wall area. Specific Quality Control requirements are contained in Section 2.7.

Grout strength requirements for clay masonry and concrete masonry are contained in IBC Sections 2105.2.2.1.1 and 2105.2.2.1.2 respectively. The requirement in both cases in that grout compressive strength is at least the masonry design strength, f'_m, and never less than 2,000 psi.

Table 2.2 shows the determined f'_m values based on the strength of the concrete or clay unit and the type of mortar used. Tables 2.2A and 2.2B are based on the requirements for clay and concrete masonry contained in IBC Section 2105.2.2.

TABLE 2.2A Compressive Strength of Masonry Based on the Compressive Strength of Clay Masonry Units and Type of Mortar Used in Construction (IBC Table 2105.2.2.1.1)

Net Area Compressive Strength of Clay Masonry Units, psi		Net Area Compressive Strength of Masonry[1], psi
Type M or S Mortar[2]	Type N Mortar[2]	
1,700	2,100	1,000
3,350	4,150	1,500
4,950	6,200	2,000
6,600	8,250	2,500
8,250	10,300	3,000
9,900	—	3,500
13,200	—	4,000

For SI: 1 pound per square inch = 0.00689 MPa.

1. Values may be interpolated. When hollow clay masonry units are grouted, the grout shall conform to ASTM C476 or the grout compressive strength equals at least f'_m, but not less than 2000 psi. The grout compressive strength is determined in accordance with ASTM C1019.
2. Mortar for unit masonry, proportion specification, as specified in ASTM C270.

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TABLE 2.2B Compressive Strength of Masonry Based on the Compressive Strength of Concrete Masonry Units and Type of Mortar Used in Construction (IBC Table 2105.2.2.1.2)

Net Area Compressive Strength of Concrete Masonry Units, psi		Net Area Compressive Strength of Masonry[1, 2], psi
Type M or S Mortar[3]	Type N Mortar[3]	
1,250	1,300	1,000
1,900	2,150	1,500
2,800	3,050	2,000
3,750	4,050	2,500
4,800	5,250	3,000

For SI: 1 inch = 25.4 mm, 1 pound per square inch = 0.00689 MPa.

1. For units less than 4 inches in height, 85 percent of the values listed.
2. Values may be interpolated. In grouted concrete masonry the grout shall conform to ASTM C476 or the grout compressive strength equals at least f'_m, but not less than 2000 psi. The grout compressive strength is determined in accordance with ASTM C1019.
3. Mortar for unit masonry, proportion specification, as specified in ASTM C270.

2.2.3 TESTING PRISMS FROM CONSTRUCTED MASONRY

In the event that verification of f'_m is not confirmed, the IBC allows for testing prisms from constructed masonry. Inadequate test results can be a result of improper casting, handling, or testing of the original masonry prisms, therefore, this alternate method is a logical step in lieu of rejecting the masonry.

IBC Section 2105.3

2105.3 Testing prisms from constructed masonry. When approved by the building official, acceptance of masonry that does not meet the requirements of Section 2105.2.2.1 or 2105.2.2.2 shall be permitted to be based on tests of prisms cut from the masonry construction in accordance with Sections 2105.3.1, 2105.3.2 and 2105.3.3.

2105.3.1 Prism sampling and removal. A set of three masonry prisms that are at least 28 days old shall be saw cut from the masonry for each 5,000 square feet (465 m^2) of the wall area that is in question but not less than one set of three masonry prisms for the project. The length, width and height dimensions of the prisms shall comply with the requirements of ASTM C1314. Transporting, preparation and testing of prisms shall be in accordance with ASTM C1314.

2105.3.2 Compressive strength calculations. The compressive strength of prisms shall be the value calculated in accordance with ASTM C1314, except that the net cross-sectional area of the prism shall be based on the net mortar bedded area.

2105.3.3 Compliance. Compliance with the requirements for the specified compressive strength of masonry, f'_m, shall be considered satisfied provided the modified compressive strength equals or exceeds the specified f'_m. Additional testing of specimens cut from locations in question shall be permitted.

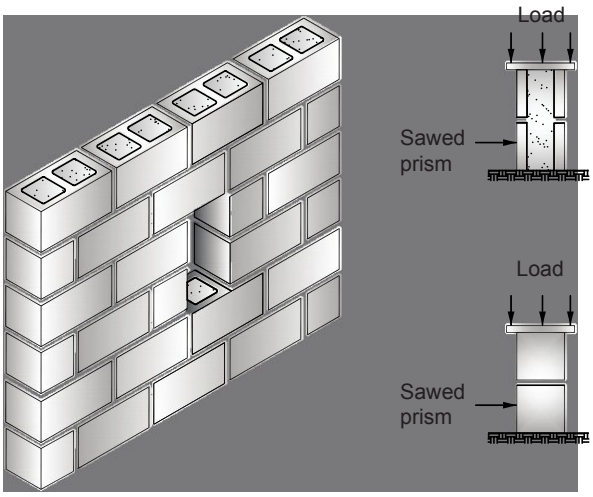

FIGURE 2.11 Test of prism sawed from wall. Load on specimen causes uniform strain, load is shared by all components of specimen.

2.3 PROPERTIES FOR GROUTED MASONRY SYSTEMS

2.3.1 SOLID GROUTED WALLS

The use of solid grouted walls has many advantages including:

1. Increased cross-sectional area provides greater capacity for shear and vertical loads.
2. Increased fire rating. An 8 in. CMU wall not solidly grouted has a fire rating of one hour while a solidly grouted wall has a four hour fire rating. See Table 2.3 which shows the rated fire resistance periods.
3. In retaining walls, the increased weight improves the stability of the wall.

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TABLE 2.3 Rated Fire-Resistance Periods for Various Walls and Partitions[1, 7, 8] (IBC-Table 720.1(2))

Material	Item Number	Construction	Minimum Finished Thickness Face–to–Face[2] (inches)			
			4 hour	3 hour	2 hour	1 hour
1. Brick of clay or shale	1–1.1	Solid brick of clay or shale[3]	6	4.9	3.8	2.7
	1–1.2	Hollow brick, not filled	5.0	4.3	3.4	2.3
	1–1.3	Hollow brick unit wall, grout or filled with perlite vermiculite or expand shale aggregate	6.6	5.5	4.4	3.0
	1–2.1	4” nominal thick units at least 75 percent solid backed with a hat–shaped metal furring channel 3/4” thick formed from 0.021” sheet metal attached to the brick wall on 24” centers with approved fasteners, and 1/2” Type X gypsum wallboard attached to the metal furring strips with 1”–long Type S screws spaced 8” on center.	–	–	5[4]	–
2. Combination of clay brick and load-bearing hollow clay tile	2–1.1	4” solid brick and 4” tile (at least 40 percent solid)	–	8	–	–
	2–1.2	4” solid brick and 8” tile (at least 40 percent solid)	12	–	–	–
3. Concrete masonry units	3–1.1[5,6]	Expanded slag or pumice	4.7	4.0	3.2	2.1
	3–1.2[5,6]	Expanded clay, shale or slate	5.1	4.4	3.6	2.6
	3–1.3[5]	Limestone, cinders or air–cooled slag	5.9	5.0	4.0	2.7
	3–1.4[5,6]	Calcareous or siliceous gravel	6.2	5.3	4.2	2.8

For SI: 1 inch = 25.4 mm, 1 square inch = 645.2 mm^2, 1 cubic foot = 0.0283 m^3

1. Staples with equivalent holding power and penetration shall be permitted to be used as alternate fasteners to nails for attachment to wood framing.
2. Thickness shown for brick and clay tile are nominal thicknesses unless plastered, in which case thicknesses are net. Thickness shown for concrete masonry and clay masonry is equivalent thickness defined in Section 721.3.1 for concrete masonry and Section 721.4.1.1 for clay masonry. Where all cells are solid grouted or filled with silicone-treated perlite loose-fill insulation; vermiculite loose-fill insulation; or expanded clay, shale or slate lightweight aggregate, the equivalent thickness shall be the thickness of the block or brick using specified dimensions as defined in Chapter 21. Equivalent thickness may also include the thickness of applied plaster and lath or gypsum wallboard, where specified.
3. For units in which the net cross-sectional area of cored brick in any plane parallel to the surface containing the cores is at least 75 percent of the gross cross-sectional area measured in the same plane.
4. Shall be used for nonbearing purposes only.
5. The fire-resistance time period for concrete masonry units meeting the equivalent thicknesses required for a 2-hour fire-resistance rating in Item 3, and having a thickness of not less than 7 5/8 in. is 4 hours when cores which are not grouted are filled with silicone-treated perlite loose-fill insulation; vermiculite loose-fill insulation; or expanded clay, shale or slate lightweight aggregate, sand or slag having a maximum particle size of 3/8 inch.
6. The fire-resistance rating of concrete masonry units composed of a combination of aggregate types or where plaster is applied directly to the concrete masonry shall be determined in accordance with ACI 216.1/TMS 0216. Lightweight aggregates shall have a maximum combined density of 65 pounds per cubic foot.
7. Generic fire-resistance ratings (those not designated as PROPRIETARY* in the listing) in the GA 600 shall be accepted as if herein listed.
8. NCMA Tek 5-8A, shall be permitted for the design of fire walls.

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4. Improved Sound Transmission Coefficient, STC. Solid grouted walls do not easily transmit sound. See "Sound Transmission Class Ratings for Concrete Masonry Walls" (NCMA TEK 13-1B).

TABLE 2.4 Calculated STC Ratings for Concrete Masonry Walls (NCMA TEK 13-1B, Excerpts)

Nominal Unit Size	Density (pcf)	STC[1]			
		Hollow Unit	Grout Filled	Sand Filled	Solid Units
4	115	44	47[2]	46	46
6	115	45	51	49	50
8	115	47	55	52	53
10	115	48	58	55	57
12	115	49	62	58	60

Nominal Unit size	Density (pcf)	STC			
		Hollow Unit	Grout Filled	Sand Filled	Solid Units
4	135	45	47[2]	47	47
6	135	46	52	50	51
8	135	48	56	53	55
10	135	50	60	56	59
12	135	51	64	59	63

1. Based on grout density of 140 lb/ft^3; sand density of 90 lb/ft^3; unit percentage solid from mold manufacturer's literature for typical units 4 in. (73.8% solid), 6 in. (55.0% solid); 8 in. (53.0% solid); 10 in. (51.7% solid); 12 in. (48.7% solid). STC values for grout-filled and sand-filled units assume the fill materials completely occupy all voids in and around the units. STC values for solid units are based on all mortar joints solidly filled with mortar.
2. Because of small core size and the resulting difficulty consolidating grout, these units are rarely grouted.

Some disadvantages to solid grouted walls are:

1. More material (grout) is required.
2. Wall is heavier and foundation may have to be bigger.
3. Seismic load on wall is greater because it weighs more.

2.3.2 PARTIALLY GROUTED WALLS

The advantages to partially grouted walls are as follows:

1. Less material (grout) is needed.
2. Wall is lighter and seismic forces are decreased.
3. Allows for insulation fill.

Disadvantages to partially grouted walls are:

1. Decreases cross-sectional area and provides less capacity for shear and vertical loads.
2. Decreased fire rating.
3. In retaining walls, the decreased weight lessens the stability of the wall.
4. Sound transmits more easily through partially grouted walls.
5. Design may be slightly more difficult for a hollow section.

2.4 STRESS DISTRIBUTION IN A WALL

Brick masonry generally has high unit compressive strength and for out of plane forces, the outside brick shells resist the maximum stresses. This strength offers a great advantage in reinforced brick masonry construction and thus an assumed f'_m for brick can easily be 2500 psi (as selected from Table 2.2), which means that the clay masonry strength of the units is a minimum 6600 psi for Type S or M mortars. IBC Section 2103.2 requires that clay masonry units conform to ASTM C62, ASTM C216 or ASTM C652. These three standards refer to ASTM C67 for testing.

ASTM C90, *Standard Specifications for Loadbearing Concrete Masonry Units,* requires the masonry unit strength of 1,900 psi. This value verifies typical masonry compressive design strength, f'_m, of 1,500 psi in the wall system.

If masonry walls or columns are not subjected to flexural stresses and support vertical load only, a deficiency in the strength of the masonry unit may be compensated for by an increase in the strength of the grout. However, this is not a satisfactory solution for stresses perpendicular to the plane of the wall.

Figure 2.12 shows the flexural stress distribution on a cross-section of a wall with maximum flexural compressive stresses on the outside of the wall. The masonry is subjected to compression and the grout may not be stressed due to flexural moment. The strength of grout would not contribute as greatly to the flexural strength of the wall and the strength of the masonry unit is the governing factor that controls the moment capacity of the wall, along with the quantity of reinforcement when moment is perpendicular to the plane of the wall.

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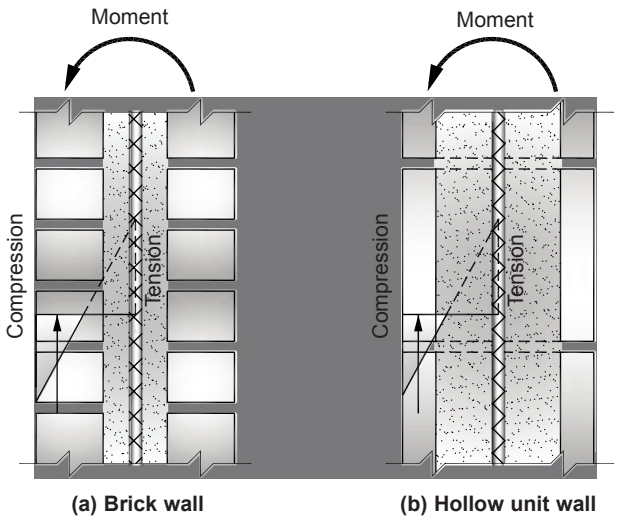

FIGURE 2.12 Bending perpendicular to plane of wall.

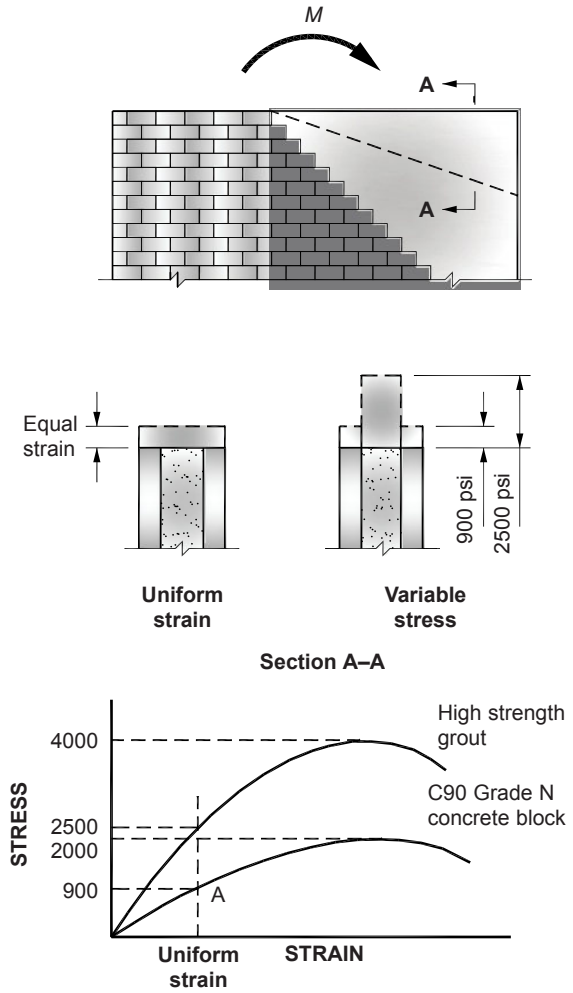

FIGURE 2.13 Moment parallel to wall, stress and strain distribution.

If the masonry wall is subjected to an overturning moment parallel to the wall, as is the case of a shear wall resisting lateral wind and seismic forces (Figure 2.13), the use of high strength grout to compensate for lower strength masonry may be reasonable. However, the recommended strength of the component materials should be as specified in Section 2.2.1.5.

2.5 WALLS OF COMPOSITE MASONRY MATERIALS

Masonry walls may be constructed with a combination of masonry materials of different characteristics and strength. If the individual masonry elements of such a composite wall are not bonded together, they would be considered to act structurally independent. In many cases, one masonry element is considered to be the structural wall and the other to be a veneer, such as the wall section shown in Figure 2.14.

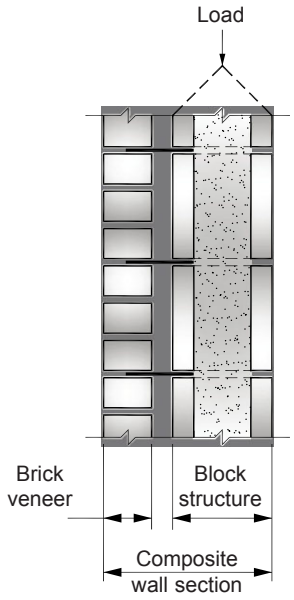

FIGURE 2.14 Structural masonry wall with masonry veneer.

When masonry materials are bonded together, these materials are assumed to act as a total structural system, distributing stresses between the wythes, such as the system shown in Figure 2.15. The thickness would be the total thickness of the wall, and the ultimate strength for axial compression would be limited to the strength of the weakest masonry unit, or handled by calculating a transformed section to an equivalent material as is typically done by using the ratios of the moduli of elasticities of the wythes.

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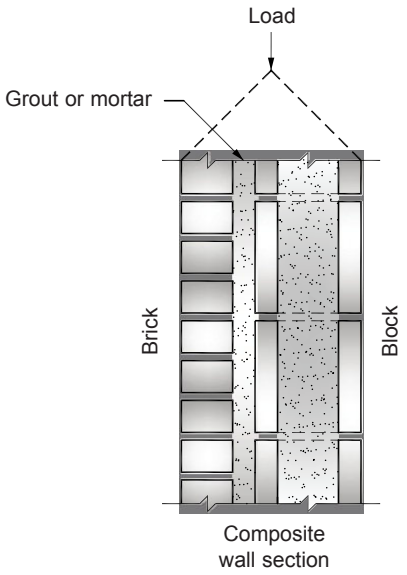

FIGURE 2.15 Composite masonry wall in which all materials act structurally.

When the wall is subjected to flexure, the ultimate compression strength should be governed by the strength of the masonry unit that is resisting the flexural compression stress. The bond between units would be achieved by grout or mortar as the units are laid.

Shear at the interfaces of the composite wythes of the masonry is given in MSJC Code Section 2.1.5.2.2. Usually, this value is not a controlling stress in the design of composite masonry walls. Tests of composite walls have been conducted at Iowa State University (ISU) and other locations resulting in MSJC Code Commentary explanation of the application and implications.

MSJC Code Commentary Section 2.1.5.2

Test results[2.4, 2.5] show that shear bond strength of collar joints could vary from as low as 5 psi (34.5 kPa) to as high as 100 psi (690 kPa), depending on type and condition of the interface, consolidation of the joint, and type of loading. McCarthy et al.[2.4] reported an average value of 52 psi (35.9 kPa) with a coefficient of variation of 21.6 percent. A low bound allowable shear value of 5 psi (34.5 kPa) is considered to account for the expected high variability of the interface bond. With some units, Type S mortar slushed collar joints may have better shear bond characteristics than Type N mortar. Results show that thickness of joints, unit absorption, and reinforcement have a negligible effect on shear bond strength. Grouted collar joints have higher allowable shear bond stress than the mortared collar joints[2.5]. Requirements for masonry headers are empirical and taken from prior codes. The net area of the header should be used in calculating the stress even if a solid unit, which allows up to 25 percent coring, is used. Headers do not provide as much ductility as metal tied wythes with filled collar joints. The influence of differential movement is especially critical when headers are used. The committee does not encourage the use of headers.

A strength analysis has been demonstrated by Porter and Wolde–Tinsae[2.7, 2.8] for composite walls subjected to combined in-plane shear and gravity loads. In addition, these authors have shown adequate behavioral characteristics for both brick-to-brick and brick–to–block composite walls with a grouted collar joint[2.9 - 2.12]. Finite element models for analyzing the interlaminar shearing stresses in collar joints of composite walls have been investigated by Anand et al.[2.13 - 2.16]. They found that the shear stresses were principally transferred in the upper portion of the wall near the point of load application for the in–plane loads. Thus, below a certain distance, the overall strength of the composite is controlled by the global strength of the wall, providing that the wythes are acting compositely.

Eccentric loads and moments on a wall cause higher stresses on one side of the wall. Higher strength masonry could advantageously be used on the side of higher stress.

An example of this would be a cantilever retaining wall using high strength brick on the outside of the wall and lower strength masonry units on the inside.

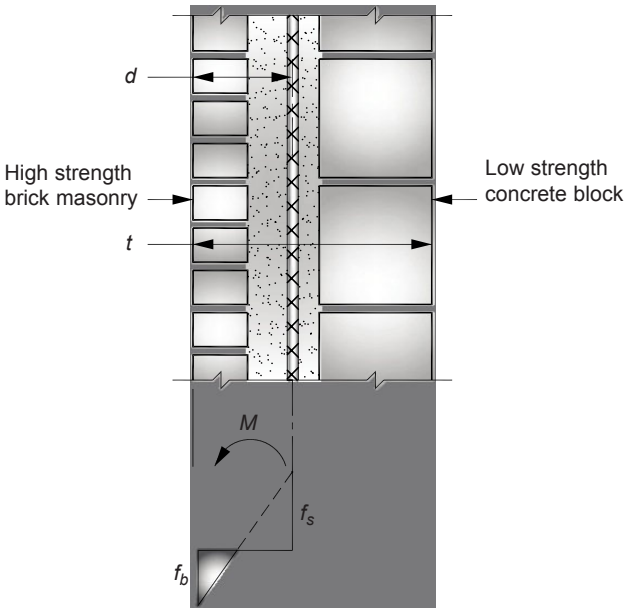

FIGURE 2.16 Cantilever retaining wall with masonry of different strengths.

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2.6 MODULUS OF ELASTICITY, E_m

2.6.1 GENERAL

The physical measure of a material to deform under load is called the modulus of elasticity, E_m. It is the ratio of the stress to the strain of a material or combination of materials as is the case for grouted masonry.

By definition, the modulus of elasticity, E_m, is determined by the secant method (chord modulus) in which the slope of the line is taken from 0.05 f'_m to a point on the curve at 0.33 f'_m. A more detailed explanation is given in MSJC Code Commentary Section 1.8.2.

Originally, E_m for masonry was the same as for concrete, namely 1000 f'_c or for masonry, 1000 f'_m. This value changed for concrete in the 1967 UBC to $33w^{1.5}(f'_c)^{0.5}$ to reflect the influence of the unit weight of concrete and the curvature of the stress strain curve.

The value for masonry assemblies was maintained as E_m = 1000 f'_m until 1988 when it was changed to 750 f'_m. This change recognized that masonry is not as stiff as concrete and has a lower modulus.

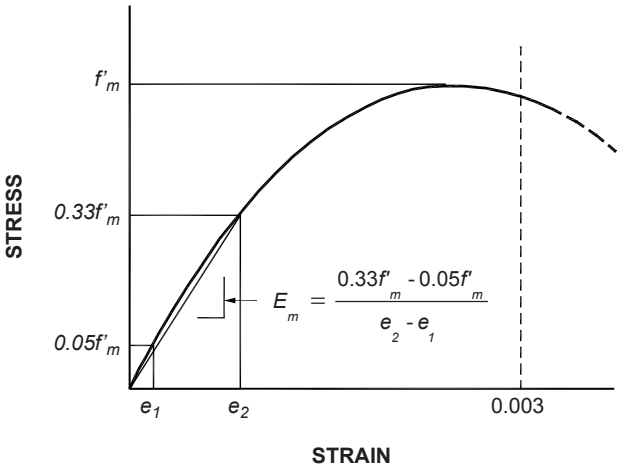

FIGURE 2.17 Stress–strain curve for grouted masonry prism and slope of line for modulus of elasticity.

However, no accommodation was made to further define the E_m based on weight, strength or volume of component materials. Thomas Holm, of the Solite Corporation, suggested the equation, $E_m = 22w^{1.5}(f'_m)^{0.5}$, to reflect the influence of light weight masonry and the strength of the assembly.

The 2005 edition of MSJC Code states the following values for E_m:

a) E_m = 700 f'_m for clay masonry

b) E_m = 900 f'_m for concrete masonry

Since the IBC does not contain values for E_m, but simply references the MSJC Code, the above values should be used. The calculated values for E_m are provided in Appendix Tables ASD-2a and ASD-2b.

2.6.2 PROPOSED EVALUATION OF MODULUS OF ELASTICITY

The modulus of elasticity (E_m) is made up of multiple parameters including the strength of the masonry unit, mortar and grout; the unit weight of the unit, mortar and grout; the volume of each of the components and the material of the masonry unit (clay or concrete).

The influence of grout will be greater on a 10 in. concrete masonry unit (CMU) wall than a 6 in. CMU wall. Also if lightweight units are used versus normal weight units, the modulus will be different. Even varying the type of mortar or the height of the units can affect the modulus of elasticity.

All the above can change the modulus of elasticity but sensitivity evaluations can be made to determine the influence of each parameter. The wide variation in materials, workmanship and quality control may make the detailed determination of the E_m unnecessary or even unrealistic.

2.7 INSPECTION OF MASONRY DURING CONSTRUCTION

Reinforced masonry is normally built in place at the job site. Accordingly, there must be some assurance that the masonry units, mortar, grout, and reinforcing steel, and any other installed material, conform to the material standards and that the construction, steel placement and grouting conform with the plans and specifications and applicable building code. This assurance takes the form of observation by a qualified masonry construction inspector required by IBC Section 1704.1.

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2.7.1 ADVANTAGES OF INSPECTION

Special masonry inspection has a great advantage in providing concerned parties such as the owner, architect, engineer, building official and masonry contractor assurance that all facets of the masonry construction are in accordance with the plans and specifications.

When using masonry inspection, full allowable stresses are used to design the masonry as opposed to past history of using full versus half stresses. This inspection process can result in smaller members, higher, thinner walls and reduced requirements for reinforcing steel. These beneficial factors can often offset the cost of the inspection. The recent editions of the IBC and the MSJC Code presume that structural masonry will be inspected in accordance with the appropriate level of Quality Assurance (QA). This inspection is based on the same philosophy as reinforced concrete and structural steel construction, which presume inspection according to the Code QA procedures. Masonry inspections must be made to assure that the steel is proper size, in the correct location and that the grout is placed and consolidated correctly.

IBC Section 109.1

109.1 General. Construction or work for which a permit is required shall be subject to inspection by the building official and such construction or work shall remain accessible and exposed for inspection purposes until approved. Approval as a result of an inspection shall not be construed to be an approval of a violation of the provisions of this code or of other ordinances of the jurisdiction.

Quality Assurance is provided in both IBC and MSJC Code. Since IBC requirements supercede MSJC Code requirements, IBC provisions are given in Tables 2.5 through 2.7. For information, the Quality Assurance provisions of MSJC Code Section 1.15 are provided.

MSJC Code Section 1.15

1.15 – Quality assurance program

The quality assurance program shall comply with the requirements of this section, depending on the facility function, as defined in the legally adopted building code or ASCE 7-02. The quality assurance program shall itemize the methods used to verify conformance of material composition, quality, storage, handling, preparation, and placement with the requirements of ACI 530.1/ASCE 6/TMS 602.

1.15.1 The minimum quality assurance program for masonry in non-essential facilities and designed in accordance with Chapter 5, 6, or 7 shall comply with Table 1.15.1.

1.15.2 The minimum quality assurance program for masonry in essential facilities and designed in accordance with Chapter 5, 6, or 7 shall comply with Table 1.15.2.

1.15.3 The minimum quality assurance program for masonry in non–essential facilities and designed in accordance with chapters other than Chapter 5, 6 or 7 shall comply with Table 1.15.2.

1.15.4 The minimum quality assurance program for masonry in essential facilities and designed in accordance with chapters other than Chapter 5, 6, or 7 shall comply with Table 1.15.3.

1.15.5 The quality assurance program shall set forth the procedures for reporting and review. The quality assurance program shall also include procedures for resolution of noncompliances.

1.15.6 The quality assurance program shall define the qualifications for testing laboratories and for inspection agencies.

2.7.2 INSPECTION REQUIREMENTS

Various editions of the UBC provided for half allowable stresses to be used in design of masonry structures that were not inspected and full allowable stresses for those cases where the special inspection was performed.

The only condition where half allowable stresses would currently apply is the use of the International Existing Building Code if one were evaluating a building that had been designed and constructed under the criteria of the half stresses for masonry that was not inspected. Current design provisions provide for full allowable stresses based on the level of inspection for the type of structure. Naturally, an emergency (essential) facility requires a higher level of inspection than a small convenience store. The required minimum level of inspection incorporates this concept.

Quality assurance is contained in Chapter 17 of the IBC. The MSJC Code and Specification also contain quality assurance provisions that may be used when design and construction does not implement the IBC. Since this is rarely the case, the IBC provisions are presented.

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IBC Section 1704

1704.1 General. Where application is made for construction as described in this section, the owner or the registered design professional in responsible charge acting as the owner's agent shall employ one or more special inspectors to provide inspections during construction on the types of work listed under Section 1704. The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for inspection of the particular type of construction or operation requiring special inspection. These inspections are in addition to the inspections specified in Section 109.

Exceptions:

1. Special inspections are not required for work of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.
2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in Group R-3 as applicable in Section 101.2 and occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.

IBC Section 1704.5

1704.5 Masonry construction. Masonry construction shall be inspected and evaluated in accordance with the requirements of Sections 1704.5.1 through 1704.5.3, depending on the classification of the building or structure or nature of the occupancy, as defined by this code.

Exception: Special inspections shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of ACI 530/ASCE 5/TMS 402, respectively, when they are part of structures classified as Occupancy Category I, II or III in accordance with Section 1604.5.
2. Masonry foundation walls constructed in accordance with Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

1704.5.1 Empirically designed masonry, glass unit masonry and masonry veneer in Occupancy Category IV. The minimum special inspection program for empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of ACI 530/ASCE 5/TMS 402, respectively, in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

1704.5.2 Engineered masonry in Occupancy Category I, II or III. The minimum special inspection program for masonry designed by Section 2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

1704.5.3 Engineered masonry in Occupancy Category IV. The minimum special inspection program for masonry designed by Section 2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.3.

IBC Section 1708.1

1708.1 Masonry. Testing and verification of masonry materials and assemblies prior to construction shall comply with the requirements of Sections 1708.1.1 through 1708.1.4, depending on the classification of the building or structure or nature of the occupancy, as defined by this code.

1708.1.1 Empirically designed masonry and glass unit masonry in Occupancy Category I, II or III. For masonry designed by Section 2109 or 2110 or by Chapter 5 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, certificates of compliance used in masonry construction shall be verified prior to construction.

1708.1.2 Empirically designed masonry and glass unit masonry in Occupancy Category IV. The minimum testing and verification prior to construction for masonry designed by Section 2109 or 2110 or by Chapter 5 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with the requirements of Table 1708.1.2.

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TABLE 1704.5.1
LEVEL 1 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC Section	ACI 530/ ASCE 5/ TMS 402[a]	ACI 530.1/ ASCE 6/ TMS 602[a]
1. As masonry construction begins, the following shall be verified to ensure compliance:					
a. Proportion of site-prepared mortar.	—	x	—	—	Art. 2.6A
b. Construction of mortar joints.	—	x	—	—	Art. 3.3B
c. Location of reinforcement, connectors, prestressing tendons and anchorages.	—	x	—	—	Art. 3.4, 3.6A
d. Prestressing technique.	—	x	—	—	Art. 3.6B
e. Grade and size of prestressing tendons and anchorages.	—	x	—	—	Art. 2.4B, 2.4H
2. The inspection program shall verify:					
a. Size and location of structural elements.	—	x	—	—	Art. 3.3G
b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	—	x	—	Sec. 1.2.2(e), 2.1.4, 3.1.6	—
c. Specified size, grade and type of reinforcement.	—	x	—	Sec. 1.13	Art. 2.4, 3.4
d. Welding of reinforcing bars.	x	—	—	Sec. 2.1.10.7.2, 3.3.3.4(b)	—
e. Protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).	—	x	Sec. 2104.3, 2104.4	—	Art. 1.8C, 1.8D
f. Application and measurement of prestressing force.	—	x	—	—	Art. 3.6B
3. Prior to grouting, the following shall be verified to ensure compliance:					
a. Grout space is clean.	—	x	—	—	Art. 3.2D
b. Placement of reinforcement and connectors and prestressing tendons and anchorages.	—	x	—	Sec. 1.13	Art. 3.4
c. Proportions of site-prepared grout and prestressing grout for bonded tendons.	—	x	—	—	Art. 2.6B
d. Construction of mortar joints.	—	x	—	—	Art. 3.3B
4. Grout placement shall be verified to ensure compliance with code and construction document provisions.	x	—	—	—	Art. 3.5
a. Grouting of prestressing bonded tendons.	x	—	—	—	Art. 3.6C
5. Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed.	x	—	Sec. 2105.2.2, 2105.3	—	Art. 1.4
6. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified.	—	x	—	—	Art. 1.5

For SI: °C = (°F - 32)/1.8.
a. The specific standards referenced are those listed in Chapter 35.

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TABLE 1704.5.3
LEVEL 2 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC Section	ACI 530/ ASCE 5/ TMS 402[a]	ACI 530.1/ ASCE 6/ TMS 602[a]
1. From the beginning of masonry construction, the following shall be verified to ensure compliance:					
a. Proportions of site-prepared mortar, grout and prestressing grout for bonded tendons.	—	x	—	—	Art. 2.6A
b. Placement of masonry units and construction of mortar joints.	—	x	—	—	Art. 3.3B
c. Placement of reinforcement, connectors and prestressing tendons and anchorages.	—	x	—	Sec. 1.13	Art. 3.4, 3.6A
d. Grout space prior to grouting.	x	—	—	—	Art. 3.2D
e. Placement of grout.	x	—	—	—	Art. 3.5
f. Placement of prestressing grout.	x	—	—	—	Art. 3.6C
2. The inspection program shall verify:					
a. Size and location of structural elements.	—	x	—	—	Art. 3.3G
b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	x	—	—	Sec. 1.2.2(e), 2.1.4, 3.1.6	—
c. Specified size, grade and type of reinforcement.		x	—	Sec. 1.13	Art. 2.4, 3.4
d. Welding of reinforcing bars.	x	—	—	Sec.2.1.10.7.2, 3.3.3.4(b)	—
e. Protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).	—	x	Sec. 2104.3, 2104.4	—	Art. 1.8C, 1.8D
f. Application and measurement of prestressing force.	x	—	—	—	Art. 3.6B
3. Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed	x	—	Sec. 2105.2.2, 2105.3	—	Art. 1.4
4. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified.	—	x	—	—	Art. 1.5

For SI: °C = (°F - 32)/1.8.
a. The specific standards referenced are those listed in Chapter 35.

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TABLE 1708.1.2
LEVEL 1 QUALITY ASSURANCE

MINIMUM TESTS AND SUBMITTALS
Certifications of compliance used in masonry construction.
Verification of f'_m and f'_{AAC} prior to construction, except where specifically exempted by this code.

1708.1.3 Engineered masonry in Occupancy Category I, II or III. The minimum testing and verification prior to construction for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1708.1.2.

1708.1.4 Engineered masonry in Occupancy Category IV. The minimum testing and verification prior to construction for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1708.1.4.

TABLE 1708.1.4
LEVEL 2 QUALITY ASSURANCE

MINIMUM TESTS AND SUBMITTALS
Certifications of compliance used in masonry construction.
Verification of f'_m and f'_{AAC} prior to construction and every 5,000 square feet during construction.
Verification of proportions of materials in mortar and grout as delivered to the site.

For SI: 1 square foot = 0.0929 m^2

Buildings expected to remain operational after a disaster are classified as Occupancy Category IV, or essential facilities. Occupancy Categories I, II and III, defined in IBC Table 1604.5, are not as critical, and are therefore subject to less stringent inspection and testing requirements.

Non-essential facilities classified as empirically designed, or masonry veneer and glass block are not subjected to a minimum tabled level of inspection or a minimum tabled level of tests and submittals.

Essential facilities of empirical design, masonry veneer and glass block and non-essential facilities of engineered masonry (Allowable Stress Design, Strength Design) require minimum inspection of Level 1 (IBC Table 1704.5.1) and minimum tests and submittals of Level 1 (IBC Table 1708.1.2).

Essential facilities of engineered masonry require minimum inspection of Level 2 (IBC Table 1704.5.3) and minimum tests and submittals of Level 2 (IBC Table 1708.1.4).

2.7.3 SUMMARY OF QUALITY ASSURANCE (QA) REQUIREMENTS

Tables 2.5 through 2.7 provide a summary of the inspection requirements for masonry construction. The design type (Empirical, Glass Block, Veneer, Allowable Stress, Strength) must be correlated with the building use (Essential, Non-Essential), then applied to Level 1 or Level 2 Inspection as listed in Tables 2.6 and 2.7. MSJC Code contains similar provisions. Note that the Levels in the MSJC Code are termed A, B, and C; whereas in the IBC they are termed 1 and 2.

Technically, the inspection level is based on occupancy use category (I, II, III or IV) which is closely related to classifying structures as 'Essential' or 'Non-Essential'. However, in general, the following apply:

- Non-Essential Buildings using procedures for Empirical Designed Structures, Veneer, and Glass Block are exempt from inspection.
- Non-Essential Buildings using procedures for Engineered Designed Structures require Level 1 Inspection.
- Essential Buildings using procedures for Empirical Designed Structures, Veneer, and Glass Block require Level 1 Inspection.
- Essential Buildings using procedures for Engineered Designed Structures requires Level 2 Inspection.

The type of facility (Essential, Non-Essential) is defined by ASCE 7 *Minimum Design Loads for Buildings and Other Structures* or by the IBC. The level of the required QA depends on whether the masonry was designed as engineered by IBC Section 2107 or 2108 or as empirical, IBC Section 2109, 2110 or Chapter 14. The most important aspects of this QA are the testing and evaluation that need to be addressed during the masonry construction. The evaluation of the test results and observations during inspection must result in the proper criteria for compliance and provide provisions for nonconformance. Proper record keeping is another important aspect of QA. Laboratories need to comply with the requirements of ASTM C1093.

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2.8 CODEMASTERS

A simple guide showing the basics of masonry inspection is presented in the CodeMaster titled *Special Inspection for Masonry*. This guide shows a 6-step process from establishing responsibilities to examination of the project and documents for final acceptance. Another Codemaster, *Masonry Materials*, has also been developed showing how to properly specify masonry materials. CodeMasters available from the Masonry Institute of America, www.masonryinstitute.org.

TABLE 2.5 Quality Assurance/Inspection Level Required by IBC Section 1704.5

Masonry Type	Building Type/Use	
	Non-Essential Facility	Essential Facility
Empirically Designed Masonry, Glass Block Masonry, Masonry Veneer	Exempt (IBC Section 1704.5.1)	Level 1 (IBC Tables 1704.5.1 & 1708.1.2)
All other Masonry (e.g. Masonry Utilizing Allowable Stress or Strength Design)	Level 1 (IBC Tables 1704.5.1 & 1708.1.2)	Level 2 (IBC Tables 1704.5.3 & 1708.1.4)

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TABLE 2.6 Level 1 Quality Assurance/Inspection

MINIMUM TESTS AND SUBMITTALS	MINIMUM INSPECTION
Certificates of compliance for materials used in masonry construction. * Verification of f'_m, prior to construction, except where specifically exempted by the Code. *	**ACTIVITIES REQUIRING CONTINUOUS INSPECTION TASKS:** • Welding reinforcing bars • Grout placement * • Grouting of prestressing bonded tendons * • Preparation of test specimens * **ITEMS REQUIRING PERIODIC INSPECTION TASKS:** ***Verify the following:*** ***As masonry construction begins, compliance of:*** • Proportions of site prepared mortar * • Construction of mortar joints * • Location of reinforcement/connectors * • Prestressing technique * • Grade/size of prestressing tendons/anchorages * ***During Construction:*** • Size/location of structural elements • Type/size/location of anchors * • Size/grade of reinforcement * • Protection in cold/hot weather conditions • Application/measurement of prestressing force ***Prior to grouting:*** • Clean grout space * • Placement of reinforcement/connectors, prestressing tendons/anchorages * • Proportions of site prepared grout/prestress grout * • Construction of mortar joints * * = corresponding provision in MSJC Code QA Level B

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TABLE 2.7 Level 2 Quality Assurance/Inspection

MINIMUM TESTS AND SUBMITTALS	MINIMUM INSPECTION
Certificates of compliance for materials used in masonry construction ** Verification of f'_m: • Prior to construction ** • Every 5,000 sq ft. (464.5 m^2) during construction ** Verification of proportions of materials in mortar and grout as delivered to the site. **	**ACTIVITIES REQUIRING CONTINUOUS INSPECTION TASKS:** ***Verify the following:*** ***From the beginning of masonry construction:*** • Grout space prior to grouting ** • Placement of grout ** • Placement of prestressing grout ** ***During Construction:*** • Type/size/location of anchors ** • Welding of reinforcement • Application/measurement of prestressing force • Preparation of test specimens ** **ITEMS REQUIRING PERIODIC INSPECTION TASKS:** ***Verify the following:*** ***As masonry construction begins, compliance of:*** • Proportions of site-prepared mortar/ grout/prestress grout ** • Placement of masonry units ** • Construction of mortar joints ** • Placement of reinforcement/connectors/ prestressing tendons/anchors ** ***During Construction:*** • Size/location of structural elements • Size/grade/type of reinforcement ** • Protection in cold/hot weather conditions ** = corresponding provision in MSJC Code QA Level C

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2.9 QUESTIONS AND PROBLEMS

2-1 What three methods are described in the Code for verifying the specified strength in masonry?

2-2 When must prisms be made? How many prisms are required prior to construction? How many prisms for full stress design should be made during construction?

2-3 Is it necessary to make and test prisms for concrete masonry when f'_m = 1500 psi?

2-4 Are prisms required before and during construction for inspected work if f'_m = 2700 psi for clay masonry?

2-5 What can the assumed f'_m be for a wall if you use solid clay units for a structure that has a gross strength of 6000 psi? What should be the strength of the grout? Are prism tests required?

2-6 What are the correction factors based for concrete masonry prisms, (a) 12 in. thick, 18 in. high and 24 in. long, (b) 6 in. thick, 24 in. high and 16 in. long? What are the correction factors based upon ACI requirements for (a) hollow clay units 6 in. wide, 12 in. high and 12 in. long, (b) for solid clay units that are 4 in. wide, 20 in. high and 12 in. long?

2-7 What is the maximum verified f'_m if the results of five compression tests are as follows: 3250 psi, 2700 psi, 2600 psi, 3400 psi, and 3160 psi? If the test results are 4308 psi, 4410 psi, 3560 psi, 3010 psi, 3900 psi, what is the maximum verified f'_m?

2-8 Why must the strength of the masonry unit be greater than the desired f'_m?

2-9 What are the MSJC and ACI equations for the modulus of elasticity?

2-10 What is the influence of the strength of grout and mortar on the modulus of elasticity?

2-11 Explain what is meant by the levels of inspected masonry? What items should be inspected? What are the advantages and disadvantages of inspection during construction?

2-12 Describe the benefits of prism testing?

2-13 Why is the compressive strength of grouted masonry systems not governed by the water-to-cement ratio of the mortar or grout as is concrete? State in words why it is better to let a mason use judgment when adding water to a mortar mix rather than specifying a certain amount that must be used.

2-14 What is a grout demonstration panel and when is it used? What procedure is followed and who decides the acceptable outcome?

2-15 Describe a procedure for determining the compressive strength of an in-place masonry wall. State a section of the IBC that could be used for this determination.

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CHAPTER 3

LOADS

3.1 GENERAL

All structures must be designed to support their own weight along with any superimposed forces, such as the dead loads from other materials, live loads, wind pressures, seismic forces, snow and ice loads, and earth pressures. These vertical and lateral loads may be of short duration such as those from earthquakes, or they may be of longer duration, such as the dead loads of machinery and equipment. Proper design must consider all possible applied forces along with the interaction of these forces on the structure.

3.2 LOAD COMBINATIONS

Because various loads may act on a structure simultaneously, load combinations should be evaluated to determine the most severe conditions for design. These load combinations vary from one document to another, depending upon the jurisdiction. The MSJC Code has common load combinations that have traditionally been applied to structural masonry design. There are a set of combinations for the allowable stress design and another set that incorporates load factors for strength design. Paragraphs below provide these load combinations.

The 2006 IBC has three sets of load combinations. There are two sets of load combinations contained (one each) in Sections 1605.2.1 and 1605.3.1 for "Basic load combinations" and one set in Section 1605.3.2 for "Alternative basic load combinations". This chapter includes the allowable stress load combinations given below, whereas the combinations for strength design will be given in Chapter 6.

The "Basic load combinations" for the allowable stress design (working stress design) are given in 2006 IBC Section 1605.3.1 as follows:

1. Dead load plus lateral fluid pressures, [$D + F$],
2. Dead load plus hydrostatic lateral soil plus lateral fluid pressures plus live load plus temperature, creep and shrinkage or differential movement, [$D + H + F + L + T$],
3. Dead load plus hydrostatic lateral soil plus lateral fluid pressures plus either roof live load, or snow load, or rain load, [$D + H + F +$ (L_r, or S, or R)],
4. Dead load plus hydrostatic lateral soil plus lateral fluid pressures + 0.75 times (Live Load plus temperature, creep and shrinkage or differential movement) plus 0.75 times either roof live load, or snow load, or rain load [$D + H + F + 0.75(L + T) + 0.75(L_r$, or S or R)],
5. Dead load plus hydrostatic lateral soil plus lateral fluid pressures plus (Wind or 0.7 times earthquake load) [$D + H + F +$ (W or $0.7E$)],
6. Dead load plus hydrostatic lateral soil plus lateral fluid pressures plus 0.75 times (Wind or 0.7 times earthquake load) + 0.75 time live load + 0.75 times (roof live load or snow load, or rain load) [$D + H + F + 0.75$(W or $0.7E$) + $0.75L + 0.75$(L_r or S or R)],
7. 0.6 times dead load plus wind plus hydrostatic lateral soil [$06D + W + H$],
8. 0.6 times dead load plus 0.7 times earthquake load plus hydrostatic lateral soil [$0.6D + 0.7E + H$],

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Footnotes to the Basic load combinations:

- Include lateral earth pressures in the design where they result in a more critical combination.
- The IBC does not require crane hook loads to be combined with roof live loads nor with more than three fourths of the snow load or one-half of the wind load.
- For flat roof snow loads exceeding 30 psf, 20 percent of the snow load shall be combined with the seismic loads, but flat roof snow loads of 30 psf or less need not be combined with seismic loads.
- The floor live load should not be included if its inclusion would result in lower stresses for the structure or member being designed.
- Increases in allowable stresses shall not be used with the load combinations given in this section of the IBC.

The following "alternative basic load combinations", as given in IBC Section 1605.3.2, may be used in lieu of the basic load combinations given above for Section 1605.3.1.

1. Dead load plus live load plus either roof live load, snow load, or rain load, [$D + L + (L_r, S,$ or $R)$],
2. Dead load plus live load plus coefficient ω times the wind load, [$D + L + (\omega W)$],
3. Dead load plus live load plus coefficient ω times the wind load plus one-half times the snow load, [$D + L + \omega W + S/2$],
4. Dead load plus live load plus snow load plus one-half coefficient ω times the wind load, [$D + L + S + \omega W/2$],
5. Dead load plus live load plus snow load plus (1/1.4) times the earthquake load, [$D + L + S + E/1.4$],
6. 0.9 times the dead load plus (1/1.4) times the earthquake load, [$0.9D + E/1.4$],

Footnotes to the above combinations:

- Include lateral earth pressures in the design where they result in a more critical combination.
- The IBC does not require crane hook loads to be combined with roof live loads nor with more than three fourths of the snow load or one-half of the wind load.
- For flat roof snow loads exceeding 30 psf, 20 percent of the snow load shall be combined with the seismic loads, but flat roof snow loads of 30 psf or less need not be combined with seismic loads.
- When using these alternate basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced, where permitted by the IBC or by the referenced standard of IBC (which is the MSJC Code).
- Where the wind loads are calculated by Chapter 6 of ASCE 7, the coefficient ω in the above equations shall be taken as 1.3; and, for other wind loads ω shall be taken as 1.0.
- When these combinations are used for foundations for loads including seismic, the vertical seismic effect, E_v, in Equation 12.4-4 of ASCE 7 is permitted to be taken as zero.
- When these combinations are used to evaluate sliding, overturning, and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 of ASCE 7 shall not be used.
- For load combinations that include counteracting effects of dead and wind loads, only two-thirds of the minimum dead load that is likely to be in place during the designed wind event shall be used.

Special seismic load combinations are given in IBC Section 1605.4. These combinations apply to both allowable stress design and strength design methods, where specifically required by IBC Sections 1605.1 or by Chapters 18 through 23 of IBC, which includes the masonry chapter (which is IBC Chapter 21). The following equation applies when the forces from seismic ground motion are additive to the gravity loads:

$$1.2D + f_1L + E_m \qquad \text{(IBC Eq 16-22)}$$

The following equation applies when the forces from seismic ground motion counteracts the gravity loads:

$$0.9D + E_m \qquad \text{(IBC Eq 16-23)}$$

where:

E_m = The maximum effect of horizontal and vertical forces as set forth in Section 12.4.3 of ASCE 7.

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 psf and for parking garage live load, or

f_1 = 0.5 for other live loads.

The maximum earthquake load effect, E_m, in Section 12.4.3 of ASCE 7 includes the effects of the special load combinations where a system "overstrength" needs to be considered in the design. This E_m includes the effects of the horizontal load, E_{mh}, and the vertical component, E_v. The E_{mh} is the product of the overstrength factor, Ω_o, and Q_E, which are the effects of the horizontal earthquake forces. The E_v accounts for the vertical acceleration due to the earthquake ground motion, which is taken as

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$0.2S_{DS}D$. The magnitude of the E_v is not intended to represent a total vertical response, since this component is not likely to occur at the same time as horizontal response, and represents a portion of the dead load, D, that is additive or subtractive in the load combinations. The S_{DS} and other earthquake items are discussed later in the seismic portions of this text. Therefore, the E_m expression reads:

$$E_m = \Omega_o Q_E \pm E_v$$

Therefore, substituting into IBC Equations 16-22 and 16-23 gives:

$$1.2D + f_1 L + E_m = (1.2 + 0.2S_{DS})D + f_1 L + \Omega_o Q_E$$

and

$$0.9D + E_m = (0.9 - 0.2S_{DS})D + \Omega_o Q_E$$

Note, again, that the code requires the use of these strength combinations for both the allowable stress and the strength design procedures.

The basis of many load combinations comes from ASCE 7. In cases where more detailed load combinations are needed or where the details of the loads are needed, the reader is referred to ASCE 7. For example, the details of computing the earthquake and wind loads are contained in ASCE 7. Subsequent sections in this chapter contain additional design information.

The load combinations according to MSJC Code Section 2.1.2.1 are as follows:

1. Dead load (only), [*D*]
2. Dead load plus live load, [*D* + *L*]
3. Dead load plus live load plus either wind or earthquake loads, [*D* + *L* + *(W* or *E*)]
4. Dead load plus wind load, [*D* + *W*]
5. (0.9) times the dead load plus earthquake load, [0.9*D* + *E*]
6. Dead load plus live load plus either hydrostatic lateral soil or lateral fluid pressures, [*D* + *L* + (*H* or *F*)]
7. Dead load plus either hydrostatic lateral soil or lateral fluid pressures, [*D* + (*H* or *F*)]
8. Dead load plus live load plus forces caused by temperature, creep, and shrinkage or differential movements, [*D* + *L* + *T*]
9. Dead load plus forces caused by temperature, creep, and shrinkage or differential movements, [*D* + *T*]

The allowable stresses and allowable loads for Chapters 2 and 4 (i.e. for Allowable Stress Design and Prestressed Masonry Design) of the MSJC Code are permitted to be increased by one-third for the above load combinations 3, 4, and 5.

3.3 DEAD LOADS

Dead loads are long term stationary forces which include the self-weight of the structure and the weights of permanent equipment and machinery. The actual weights of materials and construction can be used. The weight of fixed service equipment, such as plumbing stacks and risers, electrical feeders, heating, ventilating and air-conditioning systems (HVAC) and fire sprinkler system are included.

Since the actual weight cannot be explicitly determined by weighing a structure or a component of the structure, the dead loads are usually obtained by calculating the weights of the structural and non-structural elements, such as the equipment and machinery. Non-structural elements include, as examples, the cladding, movable partitions, floor slab wearing surfaces, ceiling tiles and other non-functional elements attached to the building. Where movable partitions exist, a uniformly distributed load is usually included to account for these partitions to be located at various positions. This amount can vary depending upon the type of movable partitions, but 20 psf is often used for this amount of dead load. Sometimes, the designer may choose a partition live load of at least 15 psf and treat partitions that may be moved as a live load. The decision is between the marketed movable partitions versus the material partitions that may be moved. If the partitions are of known material amounts, such as masonry that is higher in weight than the commercial movable partitions, then the larger weight is used. Codes also recognize the seismic forces on these partitions as well and require that partitions not become a part of the lateral load resisting system. Seismic requirements are covered later in this chapter.

Tables GN-3a and GN-3b provide weights of masonry walls, consistent with other published industry sources.

3.4 LIVE LOADS

Live loads are short duration forces which are variable in magnitude and location. Examples of live load items include people, furniture, planters, non-stationary equipment and pianos, moveable storage materials, wind, earthquakes and snow. For this chapter,

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the wind, earthquake and snow loads are separated into subsections since ASCE 7 contains extensive coverage of those individual items.

Building codes provide live loads based on the use of the structure. For instance, office areas must be designed for 50 psf live loads (*L*), residences for 40 psf *L* and corridors for 100 psf *L*. Table 3.1 (from Table 1607.1 of the IBC) provides a more complete list of design live loads based on use.

TABLE 3.1 Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads[7] (IBC Table 1607.1)

Occupancy or use	Uniform (psf)	Concentrated (lbs)
1. Apartments (see residential)	—	—
2. Access floor systems Office use Computer use	 50 100	 2,000 2,000
3. Armories and drill rooms	150	—
4. Assembly areas and theaters Fixed seats (fastened to floor) Follow spot, projections and control rooms Lobbies Movable seats Stages and platforms	 60 50 100 100 125	 —
5. Balconies On one- and two-family residences only, and not exceeding 100 ft.^{2}	100 60	—
6. Bowling alleys	75	—
7. Catwalks	40	300
8. Dance halls and ballrooms	100	—
9. Decks	Same as occupancy served[8]	—
10. Dining rooms and restaurants	100	—
11. Dwelling (see residential)	—	—
12. Cornices	60	—
13. Corridors, except as otherwise indicated	100	—
14. Elevator machine room grating (on area of 4 in.^{2})	—	300
15. Finish light floor plate construction (on area of 1 in.^{2})	—	200
16. Fire escapes On single-family dwellings only	100 40	—
17. Garages (passenger vehicles only) Trucks and buses	40 See IBC Section 1607.6	Note 1

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TABLE 3.1 Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads[7] (IBC Table 1607.1) (Continued)

Occupancy or use	Uniform (psf)	Concentrated (lbs)
18. Grandstands (see stadium and arena bleachers)	—	—
19. Gymnasiums, main floors and balconies	100	—
20. Handrails, guards and grab bars	See IBC Section 1607.7	
21. Hospitals Corridors above first floor Operating rooms, laboratories Patient rooms	 80 60 40	 1,000 1,000 1,000
22. Hotels (see residential)	—	—
23. Libraries Corridors above first floor Reading Rooms Stack rooms	 80 60 150[2]	 1,000 1,000 1,000
24. Manufacturing Light Heavy	 125 250	 2,000 3,000
25. Marquees	75	—
26. Office buildings Corridors above first floor File and computer rooms shall be designed for heavier loads based on anticipated occupancy Lobbies and first-floor corridors Offices	 80 — 100 50	 2,000 — 2,000 2,000
27. Penal institutions Cell blocks Corridors	 40 100	—
28. Residential One- and two-family dwellings Uninhabitable attics without storage[9] Uninhabitable attics with limited storage[9,10,11] Habitable attics and sleeping areas All other areas except balconies and decks Hotels and multifamily dwellings Private rooms and corridors serving them Public rooms and corridors serving them	 10 20 30 40 40 100	 —
29. Reviewing stands, grandstands and bleachers	Note 3	

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TABLE 3.1 Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads[7] (IBC Table 1607.1) (Continued)

Occupancy or use	Uniform (psf)	Concentrated (lbs)
30. Roofs		
All roofs surfaces subjected to maintenance		300
Workers		
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	5 Nonreduceable	
All other construction	20	
Ordinary flat, pitched, and curved roofs	20	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs:		
Over manufacturing, storage warehouses, and repair garages		2,000
All other occupancies		300
Roofs used for other special purposes	Note 12	Note 12
Roofs used for promenade purposes	60	
Roofs used for roof gardens or assemby purposes	100	
31. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
32. Scuttles, skylight ribs and accessible ceilings	—	200
33. Sidewalks, vehicular driveways and yards, subject to trucking	250[4]	8,000[5]
34. Skating rinks	100	—
35. Stadiums and arenas		
Bleachers	100[3]	—
Fixed seats (fastened to floor)	60[3]	
36. Stairs and exits		Note 6
One- and two-family dwellings	40	
All other	100	
37. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		—
Light	125	
Heavy	250	

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TABLE 3.1 Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads[7] (IBC Table 1607.1) (Continued)

Occupancy or use	Uniform (psf)	Concentrated (lbs
38. Stores		
Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125	1,000
39. Vehicle barriers	See IBC Section 1607.7.3	
40. Walkways and elevated platforms (other than exitways)	60	—
41. Yards and terraces, pedestrians	100	—

For SI: 1 in. = 25.4 mm, 1 sq in. = 645.16 mm^2,
1 sq ft = 0.0929 m^2,
1 lbs per sq ft = 0.0479 kN/m^2,
1 lb = 0.004448 kN,
1 lb per cubic ft = 16 kg/m^3

1. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated loads: (1) for garages restricted to vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 in. by 4.5 in.; (2) for mechanical parking structures without slab or deck which are used for storing passenger vehicles only, 2,250 pounds per wheel.
2. The loading applies to stack room floors that support nonmobile, double-faced library bookstacks subject to the following limitations:
   a. The nominal bookstack unit height shall not exceed 90 in.;
   b. The nominal shelf depth shall not exceed 12 in. for each face; and
   c. Parallel rows of double-faced bookstacks shall be separated by aisles not less than 36 in. wide.
3. Design in accordance with the ICC *Standard on Bleachers, Folding and Telescopic Seating and Grandstands*.
4. Other uniform loads in accordance with an approved method which contains provisions for truck loadings shall also be considered where appropriate.
5. The concentrated wheel load shall be applied on an area of 20 sq in.
6. Minimum concentrated load on stair treads (on area of 4 sq in.) is 300 lbs.
7. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see IBC Section 1608). For special-purpose roofs, see IBC Section 1607.11.2.2.
8. See IBC Section 1604.8.3 for decks attached to exterior walls.
9. Attics without storage are those where the maximum clear height between the joist and rafter is less than 42 in., or where there are not two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 in. high by 2 ft wide, or greater, located within the plane of the truss. For attics without storage, this live load need not be assumed to act concurrently with any other live load requirements.
10. For attics with limited storage and constructed with trusses, this live load need only be applied to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 in. high by 2 ft wide or greater, located within the plane of the truss. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met:
    a. The attic area is accessible by a pull-down stairway or framed opening in accordance with IBC Section 1209.2, and
    b. The truss shall have a bottom chord pitch less than 2:12.
    c. Bottom chords of trusses shall be designed for the greater of actual imposed dead load or 10 psf, uniformly distributed over the entire span.
11. Attic spaces served by a fixed stair shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.
12 Roofs used for other special purposes shall be designed for appropriated loads as approved by the building official.

3.4.1 FLOOR LOADS

Floor live loads are based on the use of a structure as listed in Table 3.1. If expected floor loads exceed the values in Table 3.1, actual loads should be used in the design.

Since the full live load is unlikely to occur over a large floor area, the floor loads listed in Table 3.1 may be reduced in accordance with IBC Section 1607.9 and the following general criteria of IBC Section 1607.9.1 or an alternate floor live load reduction criteria in IBC Section 1607.9.2. Subject to the limitations in IBC Section 1607.9.1, members for which a value of $K_{LL}A_T$ is 400 sq ft or more may be designed for a reduced live load in accordance with the following equation:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

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For SI: $L = L_o\left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right)$

where:

L = reduced design live load per sq ft of area supported by the member.

L_o = unreduced design live load per sq ft of area supported by the member, as per Table 3.1 (IBC Table 1607.1).

K_{LL} = live load element factor, See Table 3.2, (IBC Table 1607.9.1).

A_T = Tributary area in sq ft L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.40L_o$ for members supporting two or more floors.

TABLE 3.2 Live Load Element Factor K_{LL} (IBC Table 1607.9.1)

Element	K_{LL}
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified above, including:	1
Edge beams with cantilever slabs	
Cantilever beams	
Two-way slabs	
Members without provisions for continuous shear transfer normal to their span	

IBC Section 1607.9.1.1

1607.9.1.1 Heavy live loads. Live loads that exceed 100 psf (4.79 kN/m^2) shall not be reduced.

Exceptions:

1. The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 1607.9.1.
2. For uses other than storage, where approved, additional live load reduction shall be permitted where shown by the registered design professional that a rational approach has been used and that such reductions are warranted.

1607.9.1.2 Passenger vehicle garages. The live loads shall not be reduced in passenger vehicle garages except the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 1607.9.1.

1607.9.1.3 Special occupancies. Live loads of 100 psf (4.79 kN/m^2) or less shall not be reduced in public assembly occupancies.

1607.9.1.4 Special structural elements. Live loads shall not be reduced for one-way slabs except as permitted in Section 1607.9.1.1. Live loads of 100 psf (4.79 kN/m^2) or less shall not be reduced for roof members except as specified in Section 1607.11.2.

The principle behind reduced live load over large tributary areas is to account for the probability that a girder supporting a very large area is not as likely to have the full live load over the entire large area as compared, for example, to a beam having a much smaller tributary area to support. To accommodate for some of the provisions from the 1997 UBC, the IBC provided for an alternative floor live load reduction method that may be used instead of the method in IBC Sections 1607.9 through 1607.9.1.4.

The alternate floor live load reduction permitted by IBC Section 1607.9.2 is based on the following procedures. These reductions shall apply to slab systems, beams, girders, columns, piers, walls, and foundations. These alternative provisions apply as follows:

1. A reduction shall not be permitted to the IBC-defined Group A occupancy.
2. A reduction shall not be permitted when the live load exceeds 100 psf except that the design live load for members supporting two or more floors may be reduced by 20 percent.
3. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors may be reduced by a maximum of 20 percent.
4. For live loads not exceeding 100 psf, the design live load for any structural member supporting 150 sq ft or more may be reduced by the formula:

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$R = 0.08 (A - 150)$

For SI: $R = 0.861 (A - 13.94)$

Such reduction shall not exceed the smallest of: 40 percent for horizontal members; 60 percent for vertical members; or R as determined by the following equation:

$$R = 23.1 (1 + D/L_o)$$

Where:

R = Reduction in percent.

A = Area (in sq ft) of floor supported by the member being designed.

D = Dead load per sq ft of area supported by the member.

L_o = Unreduced live load per sq ft of area supported by the member.

Heavy live loads are those exceeding 100 psf, and are not to be reduced, except for members supporting two or more floors, in which case a maximum of 20 percent, but the live load cannot be less than the L as calculated above (IBC Section 1607.9.1.1). This 20 percent reduction for members supporting two or more floors applies to passenger vehicle garages as well, but again L cannot be less than as calculated above (IBC Section 1607.9.1.2). As per IBC Section 1607.9.1.3, live loads of 100 psf (or less) shall not be reduced in public assembly occupancies.

Table 3.1 includes an allowance for impact conditions, except for uses and loads that include unusual vibration and impact. See IBC Section 1607.8.1 for elevators, Section 1607.8.2 for machinery, Section 1607.5 for partition loads subject to change, and Section 1607.7 for loads on handrails, guards, grab bars, and vehicle barriers.

3.4.2 CONCENTRATED LOADS

Concentrated loads are considered uniformly occupying a space 2½ ft x 2½ ft and are located to produce the maximum load effects in structural members. Floors and other similar surfaces shall be designed to support the uniformly distributed live loads or the concentrated loads as shown in Table 3.1. For further details on concentrated loads refer to IBC Section 1607.6 for truck or bus garages, Section 1607.9.1.2 for passenger vehicles, and Section 1607.12 for cranes.

3.4.3 ROOF LOADS

Building codes recognize that roofs carry lower loads than floors since roofs are not occupied or subjected to other high live loads. However, if the roof is used for personnel occupancy, the live load for occupancy must be used in design. Roofs must be designed for not only these occupancy live loads, but also loads due to wind, snow and earthquake. Roof occupancy loads are prescribed in the following paragraphs (IBC Section 1607.11), or in Table 3.1.

For ordinary flat, pitched (sloped), or curved (arches or domes) roofs, the roof live load, L_r is:

$$L_r = 20R_1R_2$$

Where:

L_r is in psf for the vertical component acting on the horizontal projection of the roof and is between 12 and 20 psf,

R_1 = 1 for a tributary area of 200 sq ft or less,

R_1 = 1.2 - $0.001A_t$ for tributary areas, A_t between 200 and 600 sq ft, or

R_1 = 0.6 for tributary areas greater than 600 sq ft

R_2 = 1 for F less than or equal to 4,

R_2 = 1.2 - $0.05F$ for F between 4 and 12, or

R_2 = 0.6 for F greater than 12,

F is the slope of the roof expressed as the number of inches of rise per foot, or for an arch or dome is the rise-to-span ratio multiplied by 32.

For other special roofs, see Table 3.3 (IBC Section 1607.11).

TABLE 3.3 Other Special Roofs

Type of Roof Use	Minimum Roof Live Load, psf
Promenade	60
Roof gardens	100
Roof assembly areas	100
Landscaped areas (landscaping is considered as dead load)	20
Awnings and Canopies	5 (plus wind and snow loads)

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Interior partitions shall be subject to a minimum horizontal load of 5 psf acting on the partition surface area for partitions that exceed 6 ft in height (IBC Section 1607.13).

3.4.3.1 SNOW LOADS

Snow loads are generally based upon local climate conditions and thus often established by the local building official. The weight of snow, depth of snow and depth of snow drifts should be obtained from the local jurisdiction where the structure is to be built. Snow loads should be considered in place of the roof live loads and their effect will result in larger members. In lieu of local jurisdiction-controlled snow loads, the map, shown in Figure 3.1 (IBC Figure 1608.2) serves as a reference source for snow loads. The snow load criteria is based upon Section 7 of ASCE 7. Those areas marked "CS" on the map are site specific areas and require a special determination of the snow load. Figure 3.1 is based upon snow loads that have a 2% annual probability of being exceeded, i.e. a 50-year mean recurrence interval. Special Alaska loads are based on CS areas and are tabulated in Table 3.4 (IBC Table 1608.2).

IBC Section 1608 provides criteria as given in ASCE 7 and utilizes the Exposure Index and Thermal Index for flat roof snow loads, p_f, as called for in Section 7.3 of ASCE 7. Table 3.5 shows the snow exposure factor, C_e, based upon the noted exposure conditions. Table 3.6 shows the thermal factor, C_t, for snow loads. A flat roof snow load, p_f, is for roofs with a slope less than or equal to 5 degrees. The factors in Tables 3.5 and 3.6 are used to calculate the snow loads for the appropriate conditions indicated in the tables (calculated as per Section 7.3 of ASCE 7).

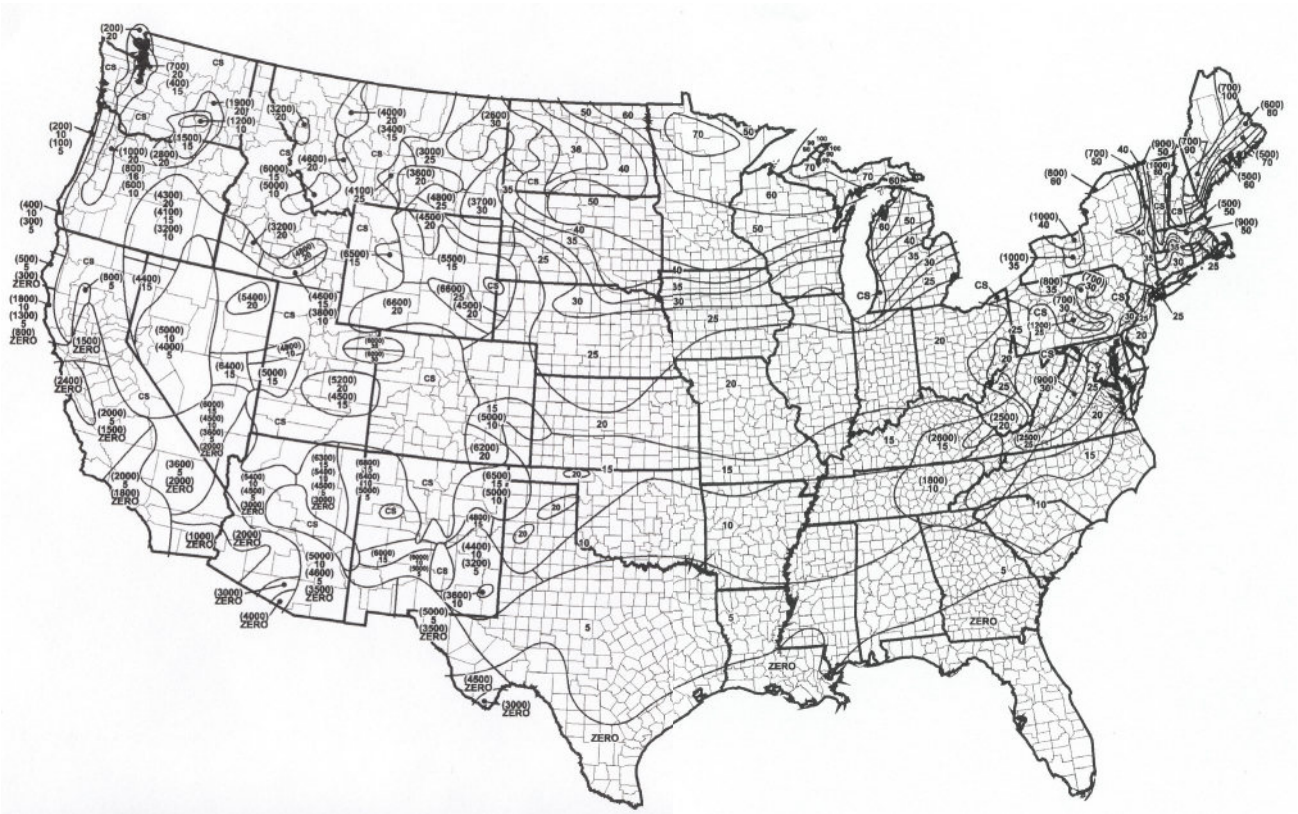

FIGURE 3.1 Ground Snow Loads, p_g, for the United States (psf) (IBC Figure 1608.2).

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TABLE 3.4 Ground Snow Loads, p_g, for Alaskan Locations (IBC Table 1608.2)

Location	Pounds per Square Foot	Location	Pounds per Square Foot	Location	Pounds per Square Foot
Adak	30	Galena	60	Petersburg	150
Anchorage	50	Gulkana	70	St. Paul Islands	40
Angoon	70	Homer	40	Seward	50
Barrow	25	Juneau	60	Shemya	25
Barter Island	35	Kenai	70	Sitka	50
Bethel	40	Kodiak	30	Talkeetna	120
Big Delta	50	Kotzebue	60	Unalakleet	50
Cold Bay	25	McGrath	70	Valdez	160
Cordova	100	Nenana	80	Whittier	300
Fairbanks	60	Nome	70	Wrangell	60
Fort Yukon	60	Palmer	50	Yakutat	150

For SI: 1 lb per sq ft = 0.0479 kN/m^2.

TABLE 3.5 Snow Exposure Factor, C_e (ASCE 7, Table 7-2)

Terrain Category[2]	Exposure of Roof[1,2]		
	Fully Exposed[3]	Partially Exposed	Sheltered
B	0.9	1.0	1.2
C	0.9	1.0	1.1
D	0.8	0.9	1.0
Above the treeline in windswept mountainous area	0.7	0.8	N/A
In Alaska, in areas where trees do not exist within 2 miles radius of the site	0.7	0.8	N/A

For SI: 1 mile = 1609 m

1. The terrain category and roof exposure condition chosen shall be representative of the anticipated conditions during the life of the structure. An exposure factor shall be determined for each roof of a structure. See ASCE Section 6.5.6.
2. Definitions of roof exposure are as follows:
   (a) Fully exposed shall mean roofs exposed on all sides with no shelter afforded by terrain, higher structures, or trees. Roofs that contain several large pieces of mechanical equipment, parapets which extend above the height of the balanced snow load, h_b, or other obstructions are not in this category.
   (b) Partially exposed shall include all roofs except those designated as "fully exposed" or "sheltered".
   (c) Sheltered roofs shall mean those roofs located tight in among conifers that qualify as "obstructions"
3. Obstructions within a distance of $10h_o$ provide "shelter," where h_o is the height of the obstruction above the roof level. If the only obstructions are a few deciduous trees that are leafless in winter, the "fully exposed" category shall be used, except for terrain category "A". Note that these are heights above the roof. Heights used to establish the terrain category (per IBC Section 1609.4) are heights above the ground. See ASCE Section 6.5.3.

TABLE 3.6 Thermal Factor, C_t (ASCE 7, Table 7-3)

Thermal Condition[1]	C_t
All structures except as indicated below:	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (*R*-value) between the ventilated space and the heated space exceeds 25°F x *h* x ft^2/Btu	1.1
Unheated structures and structures intentionally kept below freezing	1.2
Continuously heated greenhouses[2] with a roof having a thermal resistance (*R*-value) less than 2.0°F x h x ft^2/Btu	0.85

1. The thermal condition shall be representative of the anticipated conditions during winters for the life of the structure.
2. A continuously heated greenhouse shall mean a greenhouse with a constantly maintained interior temperature of 50°F or more during winter months. Such greenhouse shall also have a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating system failure.

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The importance factor is an additional snow load consideration in calculating p_f. The occupancy category (Table 3.7) is used to determine snow, wind and earthquake importance factors.

IBC Section 1604.5.1 provides for multi-occupancy categories, as follows:

IBC Section 1604.5.1

1604.5.1 Multiple occupancies. Where a structure is occupied by two or more occupancies not included in the same occupancy category, the structure shall be assigned the classification of the highest occupancy category corresponding to the various occupancies. Where structures have two or more portions that are structurally separated, each portion shall be separately classified. Where a separated portion of a structure provides required access to, required egress from or shares life safety components with another portion having a higher occupancy category, both portions shall be assigned to the higher occupancy category.

TABLE 3.7 Occupancy Category of Buildings and Other Structures (IBC Table 1604.5)

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Occupancy Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Covered structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures with elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures with an occupant load greater than 500 for colleges or adult education facilities. • Health care facilities with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities. • Jails and detention facilities. • Any other occupancy with an occupant load greater than 5,000. • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities. • Fire, rescue and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined by IBC Section 307 where the quantity of the material exceeds the maximum allowable quantities of IBC Table 307.1(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water treatment facilities required to maintain water pressure for fire suppression.

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The roof snow load, p_f, is calculated in accordance with ASCE Section 7:

For flat roofs, the snow load is found from:

$$p_f = 0.7 C_e C_t I p_g$$

Where:

C_e = is the snow exposure coefficient given in Table 3.5
I = is the importance factor based on occupancy given in Tables 3.7 and 3.8
C_t = is the thermal factor as given in Table 3.6

but not less than the following minimum values for low slope roofs as defined in ASCE 7 Section 7.3.4:

where p_g is 20 lb/ft^2 or less,
$p_f = (I)\ p_g$ (Importance Factor times p_g)

where p_g exceeds 20 lb/ft^2,
$p_f = 20(I)$ (20 lb/ft^2 times Importance Factor),

ASCE Section 7.3.4

7.3.4 Minimum Values of p_f for Low-Slope Roofs. Minimum values of p_f shall apply to monoslope roofs with slopes less than 15°, hip and gable roofs with slopes less than the larger of 2.38° (1/2 on 12) and $(70/W) + 0.5$ with W in ft (in SI: $21.3/W + 0.5$, with W in m), and curved roofs where the vertical angle from the eaves to the crown is less than 10°.

TABLE 3.8 Importance Factor, *I* (Snow Loads)

Category[1]	*I*
I	0.8
II	1.0
III	1.1
IV	1.2

1. Table 3.7 and ASCE 7 Section 1.5 and ASCE Table 1-1

For sloped roofs (with a slope greater than five degrees), the snow load, p_s, is calculated by:

$$p_s = C_s p_f$$

Where:

C_s is the roof slope factor

The values for C_s are determined for warm roofs, cold roofs, curved roofs, and multiple roofs in accordance with Sections 7.4.1 through 7.4.4 of ASCE 7. The factor C_t given in Table 3.6 determines if a roof is considered warm or cold.

Roof snow loads are assumed to act vertically on the horizontal projected roof area. When snow loads act on a slope of a roof which is more than 5 degrees, the roof snow load is calculated by Section 7.4 of ASCE 7.

3.4.3.2 RAIN LOADS

IBC Section 1611 gives additional loads due to rain.

IBC Section 1611

RAIN LOADS

1611.1 Design rain loads. Each portion of a roof shall be designed to sustain the load of rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow.

$$R = 5.2\ (d_s + d_h) \qquad \textbf{(Equation 16-36)}$$

For SI: $R = 0.0098\ (d_s + d_h)$

where:

d_h = Additional depth of water on the undeflected roof above the inlet of secondary drainage system at its design flow (i.e., the hydraulic head), in inches (mm).

d_s = Depth of water on the undeflected roof up to the inlet of secondary drainage system when the primary drainage system is blocked (i.e., the static head), in inches (mm).

R = Rain load on the undeflected roof, in psf (kN/m^2). When the phrase "undeflected roof" is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

1611.2 Ponding instability. For roofs with a slope less than 1/4 inch per foot [1.19 degrees (0.0208 rad)], the design calculations shall include verification of adequate stiffness to preclude progressive deflection in accordance with Section 8.4 of ASCE 7.

1611.3 Controlled drainage. Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system at a higher elevation that limits accumulation of water on the roof above that elevation. Such roofs shall be designed to sustain the load of rainwater that will accumulate on them to the elevation of the secondary drainage system plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow determined from Section 1611.1. Such roofs shall also be checked for ponding instability in accordance with Section 1611.2.

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3.4.3.3 FLOOD LOADS

IBC Section 1612 provides provisions for flood loads. In cases where flood conditions are possible, these provisions should be considered. If they apply, they can control the design of masonry structures, especially building walls.

3.4.3.4 SPECIAL ROOF LOADS

Water can quickly pond on roofs which are not sufficiently sloped or drained. Thus, designers must consider the possibility of ponding water which can create substantial additional roof loads and leakage. Likewise special purpose roofs require extra attention and detailing.

ASCE 7 contains certain design considerations:

- Effect of an imbalanced load over the entire loaded roof (ASCE Section 7.5)
- Unbalanced snow load (ASCE Section 7.6)
- Drifting of snow loads greater than 5 psf (ASCE Section 7.7)
- Drift loads due to mechanical equipment, penthouses, parapets and other such projections above roof (ASCE Section 7.8)
- Additional loads due to sliding of snow off a sloped roof onto lower roofs (ASCE Section 7.9)
- For roofs with a slope of less than 1/2 in. per ft, design for rain-on-snow surcharge (ASCE Section 7.10)
- For roofs with a slope less than 1/4 in. per ft, include verification of ponding prevention (ASCE Section 7.11)

3.4.3.5 SPECIAL ANCHORAGE LOADS AND DESIGN REQUIREMENTS

Masonry walls must be anchored to roofs, floors, and other structural elements that provide lateral support or diaphragm forces to the walls. Such anchorage must be capable to withstand a minimum horizontal force of 280 plf of wall, as substituted for the E force in various load combinations. Required anchors in masonry walls of hollow units or cavity walls must be embedded in a reinforced grouted structural element of the wall. Decks that are supported by attachment to exterior masonry walls must also be designed to resist vertical and horizontal loads imposed upon the deck-to-wall interaction.

3.5 WIND LOADS

ASCE 7, Chapter 6 is most commonly used to determine wind loads on structures. ASCE 7 provides for three methods to determine design wind forces on the main wind-force resisting system (MWFRS) and on components and cladding. These three methods are:

1. Simplified Procedure – Method 1
2. Analytical Procedure – Method 2
3. Wind Tunnel Procedure – Method 3

Within each of these methods, ASCE 7 provides a way to determine wind loads on the main wind force resisting system and a way to determine wind loads on the components and cladding. The code distinguishes between these two conditions because wind pressures higher than those determined for the main wind force resisting system are often experienced on small areas of the overall structure, especially at areas of discontinuities such as eaves, ridges and building corners. Because these high pressures are generally distributed over only small areas at any one time, they do not threaten the overall stability of the structure. However, these high pressures can cause failure of individual elements or components of a structure if they are not properly designed and secured with adequate connections.

Designers typically use both the main wind for resisting system wind loads and the component and cladding wind loads in the design of exterior walls - the wall is designed as a shear wall for in plane forces due to the primary wind loads and for out-of-plane bending due to component and cladding wind loads.

The discussion in this Chapter will be based on ASCE 7's Method 2 as this is method of determining wind loads. Method 2 further distinguishes between low rise buildings and buildings with heights greater than sixty feet. The discussion in the section will focus on applying the analytical procedure to low rise buildings.

3.5.1 VELOCITY PRESSURE DETERMINATIONS

Using ASCE 7's Analytical Procedure (Method 2), the first step toward determining design wind pressure is to determine the velocity pressure by the formula:

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$q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (ASCE Eq 6-15)

Where:

q_z = velocity pressure, which varies with height and exposure,

K_z = velocity pressure exposure coefficient, which varies with height and exposure,

K_{zt} = topographic factor,

K_d = directionality factor,

V = basic wind speed-corresponds to a 3-s gust speed at 33 ft above ground in Exposure Category C,

I = Importance Factor,

The portion of the equation represented by $q_z = 0.00256\ V^2$ is designated as the stagnation pressure in some codes, especially the older ones, so it is included here for completeness of terminology. Since the stagnation pressure is only a function of wind speed, it is possible to construct a simple table for that portion of ASCE Equation 6-15.

TABLE 3.9 Wind Stagnation Pressure (q_z) at Standard Height of 33 feet

Basic wind speed (mph)	70	80	90	100	110	120	130
Pressure q_z (psf)	12.6	16.4	20.8	25.6	31.0	36.9	43.3

3.5.1.1 DEFINITIONS

The IBC and ASCE 7 have provided basic definitions of terms as applied to the wind load determinations. These are contained in IBC Section 1609.2 and ASCE 7 Section 6.2 and are as follows:

IBC Section 1609.2 and ASCE Section 6.2 Definitions

BUILDING, ENCLOSED: A building that does not comply with the requirements for open or partially enclosed buildings.

BUILDING AND OTHER STRUCTURE, FLEXIBLE: Slender buildings and other structures that have a fundamental natural frequency less than 1 Hz.

BUILDING, LOW-RISE: Enclosed or partially enclosed buildings that comply with the following conditions:

1. Mean roof height h less than or equal to 60 ft (18 m).
2. Mean roof height h does not exceed least horizontal dimension.

BUILDING, OPEN: A building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_o \geq 0.8A_g$ where

A_o = Total area of openings in a wall that receives positive external pressure, in ft² (m²)

A_g = The gross area of that wall in which A_o is identified, in ft² (m²)

BUILDING, PARTIALLY ENCLOSED: A building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent.
2. The total area of openings in a wall that receives positive external pressure exceeds 4 ft² (0.37 m²) or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

These conditions are expressed by the following equations:

1. $A_o > 1.10A_{oi}$
2. $A_o > 4$ sq ft (0.37 m²) or $> 0.01A_g$, whichever is smaller, and $A_{oi}/A_{gi} \leq 0.20$

where

A_o, A_g are as defined for Open Building.

A_{oi} = The sum of the areas of openings in the building envelope (walls and roof) not including A_o, in ft² (m²).

A_{gi} = The sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in ft² (m²).

BUILDING, SIMPLE DIAPHRAGM: A building in which both windward and leeward wind loads are transmitted through floor and roof diaphragms to the same vertical MWFRS (e.g., no structural separations).

COMPONENTS AND CLADDING: Elements of the building envelope that do not qualify as part of the MWFRS.

EFFECTIVE WIND AREA, *A*: The area used to determine GC_p. For component and cladding elements, the effective wind area in Figs. 6-11 through 6-17 and Fig. 6-19 is the span length multiplied by an effective

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width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

HURRICANE-PRONE REGIONS (*IBC*). Areas vulnerable to hurricanes defined as:

1. The U.S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed is greater than 90 mph (40 m/s) and
2. Hawaii, Puerto Rico, Guam, Virgin Islands and American Samoa.

IMPORTANCE FACTOR, *I*: A factor that accounts for the degree of hazard to human life and damage to property.

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS): An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.

MEAN ROOF HEIGHT, *h*: The average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to 10°, the mean roof height shall be the roof heave height.

WIND-BORNE DEBRIS REGION: Areas within hurricane prone regions located:

1. Within 1 mile of the coastal mean high water line where the basic wind speed is equal to or greater than 110 mi/h and in Hawaii, or
2. In areas where the basic wind speed is equal to or greater than 120 mi/h.

3.5.1.2 VELOCITY PRESSURE COEFFICIENT, K_Z

In order to determine the velocity pressure coefficient the Exposure Category for the building site must be established. The IBC and ASCE 7 recognize three Exposure Categories B, C, and D. Exposure A which was used by some previous editions of ASCE 7 to characterize building sites in large city centers, is no longer recognized. The exposure category criteria are given in IBC Section 1609.4.

IBC Section 1609.4

1609.4 Exposure category. For each wind direction considered, an exposure category that adequately reflects the characteristics of ground surface irregularities shall be determined for the site at which the building or structure is to be constructed. Account shall be taken of variations in ground surface roughness that arise from natural topography and vegetation as well as from constructed features.

1609.4.1 Wind directions and sectors. For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45 degrees (0.79 rad) either side of the selected wind direction. The exposures in these two sectors shall be determined in accordance with Sections 1609.4.2 and 1609.4.3 and the exposure resulting in the highest wind loads shall be used to represent winds from that direction.

1609.4.2 Surface roughness categories. A ground surface roughness within each 45-degree (0.79 rad) sector shall be determined for a distance upwind of the site as defined in Section 1609.4.3 from the categories defined below, for the purpose of assigning an exposure category as defined in Section 1609.4.3.

Surface Roughness B. Urban and suburban areas, wooded areas or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C. Open terrain with scattered obstructions having heights generally less than 30 feet (9144 mm). This category includes flat open country, grasslands, and all water surfaces in hurricane-prone regions.

Surface Roughness D. Flat, unobstructed areas and water surfaces outside hurricane-prone regions. This category includes smooth mud flats, salt flats and unbroken ice.

1609.4.3 Exposure categories. An exposure category shall be determined in accordance with the following:

Exposure B. Exposure B shall apply where the ground surface roughness condition, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 2,600 feet (792 m) or 20 times the height of the building, whichever is greater.

Exception: For buildings whose mean roof height is less than or equal to 30 feet (9144 mm), the upwind distance is permitted to be reduced to 1,500 feet (457 m).

Exposure C. Exposure C shall apply for all cases where Exposures B or D do not apply.

Exposure D. Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance of at least 5,000 feet (1524 m) or 20 times the height of the building, whichever is greater. Exposure D shall extend inland from the shoreline for a distance of 600 feet (183 m) or 20 times the height of the building, whichever is greater.

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Once the exposure category is known, the wind coefficients K_z can be found in Table 3.10.

TABLE 3.10 Wind Coefficients for K_z (ASCE 7, Table 6-3)

Height above ground level, z	Exposure (Note 1)			
	B		C	D
ft	Case 1	Case 2	Cases 1 & 2	Cases 1 & 2
0-15	0.70	0.57	0.85	1.03
20	0.70	0.62	0.90	1.08
25	0.70	0.66	0.94	1.12
30	0.70	0.70	0.98	1.16
40	0.76	0.76	1.04	1.22
50	0.81	0.81	1.09	1.27
60	0.85	0.85	1.13	1.31
70	0.89	0.89	1.17	1.34
80	0.93	0.93	1.21	1.38
90	0.96	0.96	1.24	1.40
100	0.99	0.99	1.26	1.43
120	1.04	1.04	1.31	1.48
140	1.09	1.09	1.36	1.52
160	1.13	1.13	1.39	1.55
180	1.17	1.17	1.43	1.58
200	1.20	1.20	1.46	1.61
250	1.28	1.28	1.53	1.68
300	1.35	1.35	1.59	1.73
350	1.41	1.41	1.64	1.78
400	1.47	1.47	1.69	1.82
450	1.52	1.52	1.73	1.86
500	1.56	1.56	1.77	1.89

Notes:

1. **Case 1:** a. All components and cladding.
   b. Main wind force resisting system in low-rise buildings designed using ASCE 7 Figure 6-10.

   Case 2: a. All main wind force resisting systems in buildings except those in low-rise buildings designed using ASCE 7 Figure 6-10.
   b. All main wind force resisting systems in other structures.
2. The velocity pressure exposure coefficient K_z may be determined from the following formula:

   For 15 ft. $\leq z \leq z_g$ For z < 15 ft.

   $K_z = 2.01\ (z/z_g)^{2/\alpha}$ $K_z = 2.01\ (15/z_g)^{2/\alpha}$

   Note: z shall not be taken less than 30 ft for Case 1 in exposure B.
3. α and z_g are tabulated in ASCE 7 Table 6-2.
4. Linear interpolation for intermediate values of height z is acceptable.
5. Exposure categories are defined in ASCE 7 Section 6.5.6.

3.5.1.3 TOPOGRAPHIC FACTOR, K_{zt}

The topographic factor, K_{zt}, accounts for the increase in the velocity pressure due to the local topography causing an increase in wind speed. ASCE 7 Section 6.5.7.1 defines when the local topography needs to be considered. If site conditions and locations of structures do not meet all the conditions specified in ASCE 7 Section 6.5.7.1 then $K_{zt} = 1.0$.

ASCE 7 Section 6.5.7.1

6.5.7.1 Wind Speed-Up over Hills, Ridges, and Escarpments. Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category, shall be included in the design when buildings and other site conditions and locations of structures meet all of the following conditions:

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100H) or 2 mi (3.22 km), whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.
2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 2-mi (3.22 km) radius in any quadrant by a factor of two or more.
3. The structure is located as shown in Fig. 6-4 in the upper one-half of a hill or ridge or near the crest of an escarpment.
4. $H/L_h > 0.2$.
5. H is greater than or equal to 15 ft (4.5 m) for Exposures C and D and 60 ft (18 m) for Exposure B.

When required, K_{zt} may be calculated according to ASCE 7 Section 6.5.7.2 using the formula:

$$K_{zt} = (1 = K_1K_2K_3)^2 \qquad \text{(ASCE Eq 6-3)}$$

where K_1, K_2, and K_3 are given in Figure 3.5 (ASCE 7 Figure 6-4).

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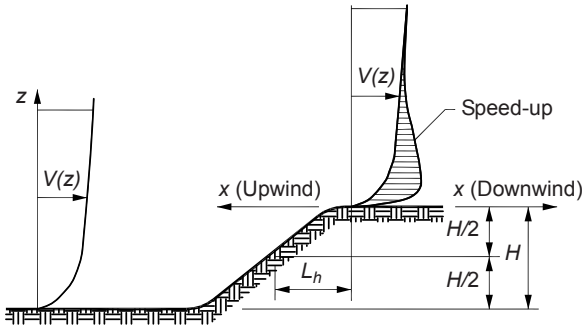
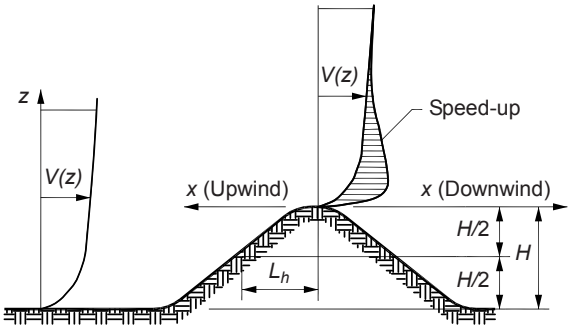


Escarpment



2-*D* Ridge or 3-*D* Axisymmetrical Hill

Topographic Multipliers for Exposure C

H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-*D* Ridge	2-*D* Escarp.	3-*D* Axisym. Hill		2-*D* Escarp.	All Other Cases		2-*D* Ridge	2-*D* Escarp.	3-*D* Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute $2H$ for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:
   - H: Height of hill or escarpment relative to the upwind terrain, in feet.
   - L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in feet.
   - K_1: Factor to account for shape of topographic feature and maximum speed-up effect.
   - K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   - K_3: Factor to account for reduction in speed-up with height above local terrain.
   - x: Distance (upwind or downwind) from the crest to the building site, in feet.
   - z: Height above local ground level, in feet.
   - μ: Horizontal attenuation factor.
   - γ: Height attenuation factor.

FIGURE 3.2 Topographic factor, K_{zt} (Based on ASCE 7 – Figure 6-4).

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Equations:

$K_{zt} = (1 + K_1 K_2 K_3)^2$

K_1 determined from table below

$$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$$

$$K_3 = e^{-\gamma z / L_h}$$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	$K_1/(H/L_h)$			γ	μ	
	Exposure				Upwind of Crest	Downwind of Crest
	B	C	D			
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. hill	0.95	1.05	1.15	4	1.5	1.5

FIGURE 3.2 (Continued) Topographic factor, K_{zt} (Based on ASCE 7 – Figure 6-4)

3.5.1.4 WIND DIRECTIONALITY FACTOR, K_d

The Wind Directionality Factor, K_d, is listed in the Table 3.11. As noted in the footnote to the table, K_d can only be used with the load combinations listed in ASCE 7. **It is partly for this reason that the IBC alternate ASD load case discussed in Section 3.2 requires the application of a 1.3 load factor to wind loads determined using ASCE 7**.

TABLE 3.11 Wind Directionality Factor, K_d (ASCE 7, Table 6-4)

Structure Type	Directionality Factor K_d*
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

* Directionality Factor K_d has been calibrated with combinations of loads specified in ASCE 7 Section 2. This factor shall only be applied when used in conjunction with load combinations specified in ASCE 7 2.3 and 2.4.

3.5.1.5 BASIC WIND SPEED, V

IBC Section 1609.3 defines the basic wind speed as follows:

IBC Section 1609.3

1609.3 Basic wind speed. The basic wind speed, in mph, for the determination of the wind loads shall be determined by Figure 1609. Basic wind speed for the special wind regions indicated, near mountainous terrain and near gorges shall be in accordance with local jurisdiction requirements. Basic wind speeds determined by the local jurisdiction shall be in accordance with Section 6.5.4 of ASCE 7.

In nonhurricane-prone regions, when the basic wind speed is estimated from regional climatic data, the basic wind speed shall be not less than the wind speed associated with an annual probability of 0.02 (50-year mean recurrence interval), and the estimate shall be adjusted for equivalence to a 3-second gust wind speed at 33 feet (10 m) above ground in Exposure Category C. The data analysis shall be performed in accordance with Section 6.5.4.2 of ASCE 7.

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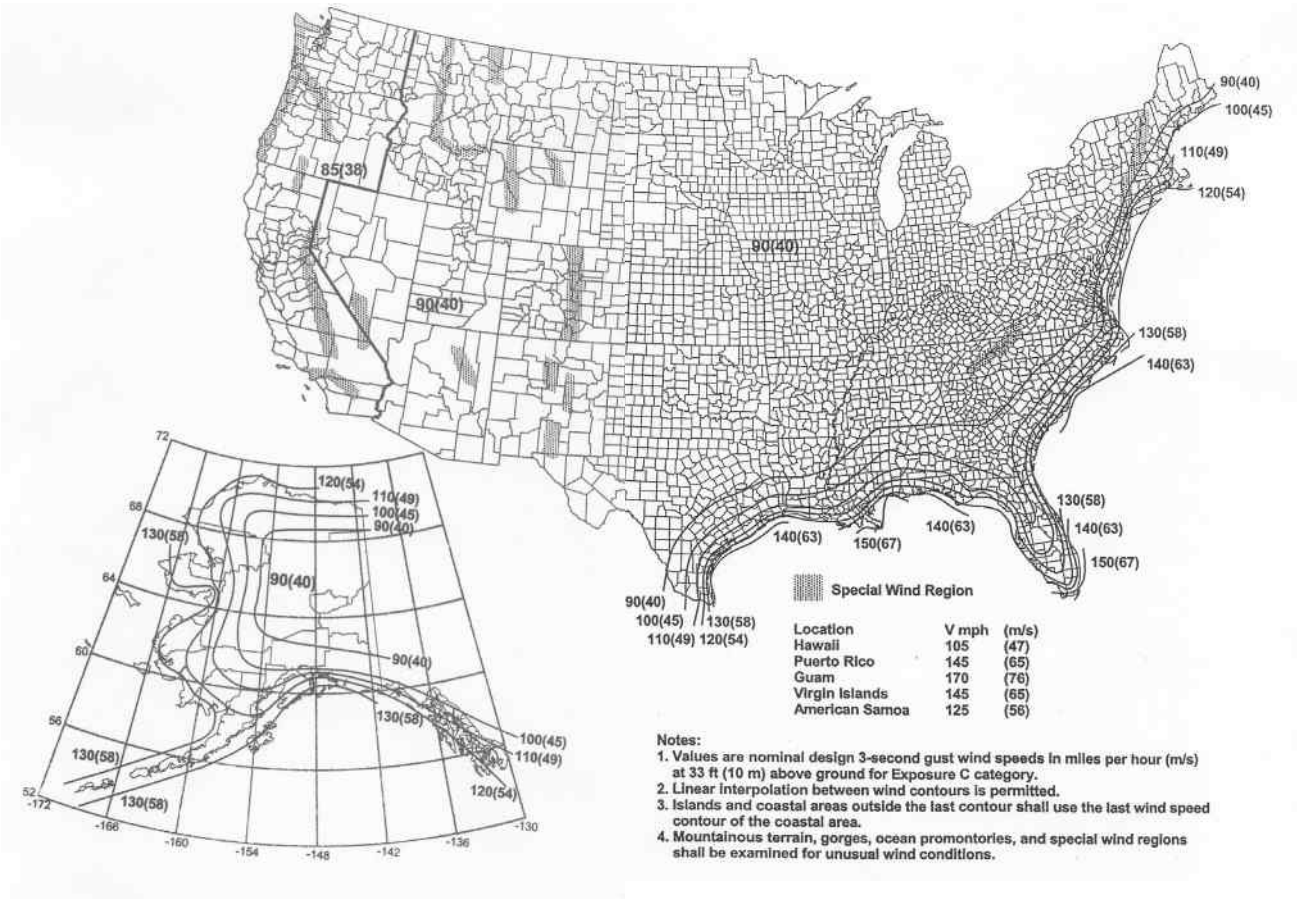

FIGURE 3.3 Basic wind (3-Second Gust) (IBC Figure 1609)

3.5.1.6 IMPORTANCE FACTOR, *I*

ASCE 7 provides for the Importance Factor, *I*, for wind loads in the following table:

TABLE 3.12 Importance Factor I for Wind Loads[1] (ASCE 7, Table 6-1)

Category[1]	Non-Hurricane Prone Regions and Hurricane Prone Regions with *V* = 85-100 mph and Alaska	Hurricane Prone Regions with V > 100 mph
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

1. See Table 3.7 or IBC Table 1604.5 or ASCE 7 Table 1-1 for the category definitions

3.5.2 WIND EXPOSURE CONDITIONS FOR THE MAIN WIND FORCE RESISTING SYSTEM

ASCE 7 Section 6.5.12.2 calculates the design wind pressure for low rise buildings with the following equation:

$$p = q_h(GC_{pf}) - (GC_{pi}) \qquad \text{(ASCE Eq 6-18)}$$

Where:

p = Design wind pressure in lbs per sq ft,

q_h = velocity pressure at mean roof height,

GC_{pf} = external pressure coefficient for MWFRS and varies depending upon the building geometry (discussed in Section 3.5.3),

GC_{pi} = internal pressure coefficient.

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This equation can be used for all structures that are low rise (height less than or equal to 60 ft) and which meet the limitations of the analytical procedure. The following additional criteria apply to the Method 2 Analytical Procedure for MWFRS:

1. The building must be structurally regular. ASCE Section 6.2 defines this as "A building or other structure having no unusual geometrical irregularity in spatial form".

2. The building must not have response characteristics from cross winds creating vortex shedding, flutter, or a location creating channeling or topographic effects.

If a building does not meet all of the above criteria, then it must be designed by one of the other methods and equations provided in ASCE 7 as follows:

1. Buildings with response characteristics from cross winds creating vortex shedding, flutter, or a location creating channeling or topographic effects – Method 3 (wind tunnel testing) per ASCE 7 Section 6.6.

2. Buildings without response characteristics from cross winds creating vortex shedding, flutter, or a location creating channeling or topographic effects:

   - Flexible Buildings – Method 2, following ASCE 7 Section 6.5.12.2.3 for flexible buildings.

   - Rigid Buildings, taller than 60 ft – Method 2, following ASCE 7 Section 6.5.12.2.1 for rigid buildings of all heights.

The abbreviated simple steps for Method 2 are to determine the following:

1. The velocity pressure at the mean roof height, q_h.

2. The external pressure coefficient, GC_{pf}, from Figure 3.5.

3. The internal pressure coefficient, GC_{pi}, from Figure 3.5.

4. The design wind pressure, $p = q_h[(GC_{pf}) - (GC_{pi})]$

Enclosure Classification	GC_{pi}
Open Buildings	0.00
Partially Enclosed Buildings	+0.55 -0.55
Enclosed Buildings	+0.18 -0.18

Notes:

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of GC_{pi} shall be used with q_z or q_h as specified in ASCE 7 Section 6.5.12.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:

   (i) a positive value GC_{pi} applied to all internal surfaces.
   (ii) a negative value GC_{pi} applied to all internal surfaces.

FIGURE 3.4 Main wind force resisting system/components and cladding/walls & roofs (Based on ASCE 7 – Figure 6-5).

3.5.3 WIND LOADS FOR COMPONENTS AND CLADDING

ASCE 7 Section 6.5.12.2 calculates the design wind pressure for low rise buildings with the following equation

$$p = q_h[(GC_{pf}) - (GC_{pi})] \qquad \text{(ASCE Eq 6-22)}$$

Where each term is defined as follows:

p = design wind pressure in lbs per sq ft,

q_h = velocity pressure at mean roof height,

GC_p = external pressure coefficient for components and cladding and varies depending upon the building geometry,

GC_{pi} = internal pressure coefficients.

This equation can be used for all structures that are low rise (height less than or equal to 60 ft) and which meet the limitations of the analytical procedure as described in the discussion of the main wind force resisting system above.

If a building does not meet all of the above criteria, then it must be designed by one of the other methods and equations provided in ASCE 7 as follows:

Continued on Page 85

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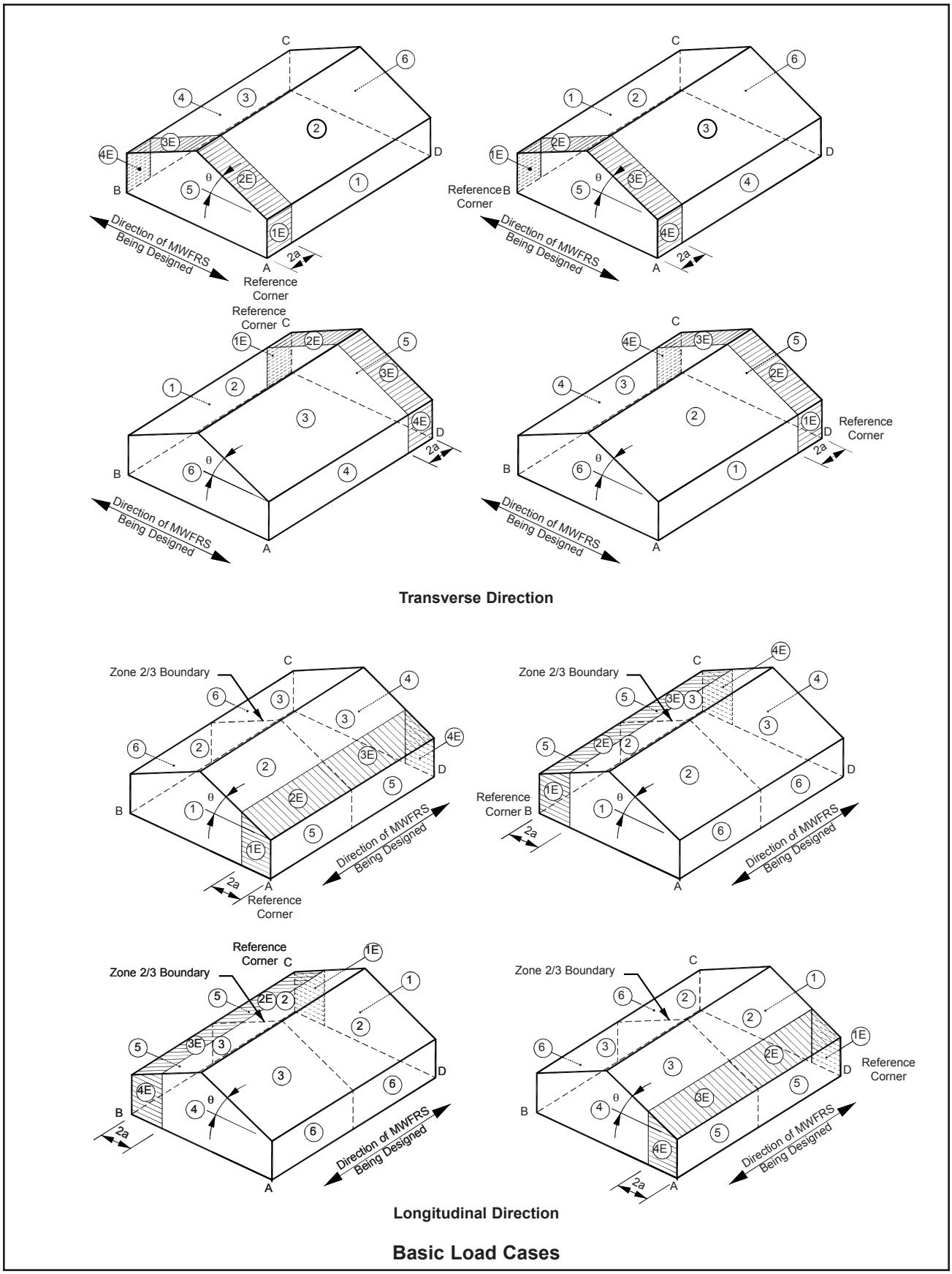

FIGURE 3.5 Main wind force resisting system/low-rise walls & roofs (Based on ASCE 7 – Figure 6-10).

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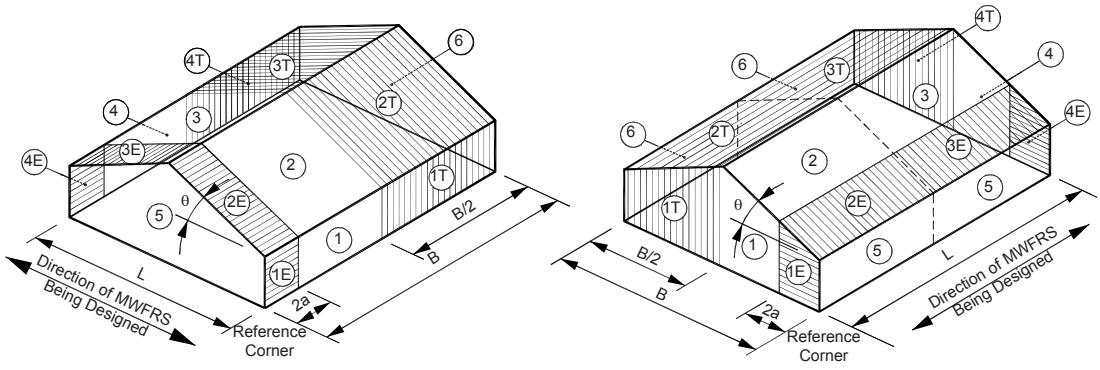
Roof Angle θ (degrees)	Building Surface									
	1	2	3	4	5	6	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	-0.45	-0.45	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	-0.45	-0.45	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	-0.45	-0.45	0.69	0.69	-0.48	-0.48

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. For values of θ other than those shown, linear interpolation is permitted.
3. The building must be designed for all wind directions using the 8 loading patterns shown. The load patterns are applied to each building corner in turn as the Reference Corner.
4. Combinations of external and internal pressures (see ASCE 7 Figure 6-5) shall be evaluated as required to obtain the most severe loadings.
5. For the torsional load cases shown below, the pressures in zones designated with a "T" (1T, 2T, 3T, 4T) shall be 25% of the full design wind pressures (Zones 1, 2, 3, 4).
   Exception: One story buildings with *h* less than or equal to 30 ft, buildings two stories or less framed with light frame construction, and buildings two stories or less designed with flexible diaphragms need not be designed for the torsional load cases.
   Torsional loading shall apply to all eight basic load patterns using the figures below applied at each reference corner.
6. Except for moment-resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
7. For the design of the MWFRS providing lateral resistance in a direction parallel to a ridge line or for flat roofs, use θ = 0° and locate the Zone 2/3 boundary at the mid-length of the building.
8. The roof pressure coefficient GC_{pf}, when negative in Zone 2 or 2E, shall be applied in Zone 2/2E for a distance from the edge of roof equal to 0.5 times the horizontal dimension of the building parallel to the direction of the MWFRS being designed or 2.5 times the eave height, h_e, at the windward wall, whichever is less; the remainder of Zone 2/2E extending to the ridge line shall use the pressure coefficient GC_{pf} for Zone 3/3E.
9. Notation:
   a: 10 percent of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft.
   h: Mean roof height, in feet, except that eave height shall be used for θ ≤ 10°.
   θ: Angle of plane of roof from horizontal, in degrees.



Torsional Load Cases

FIGURE 3.5 (Continued) Main wind force resisting system/low-rise walls & roofs (Based on ASCE 7 – Figure 6-10).

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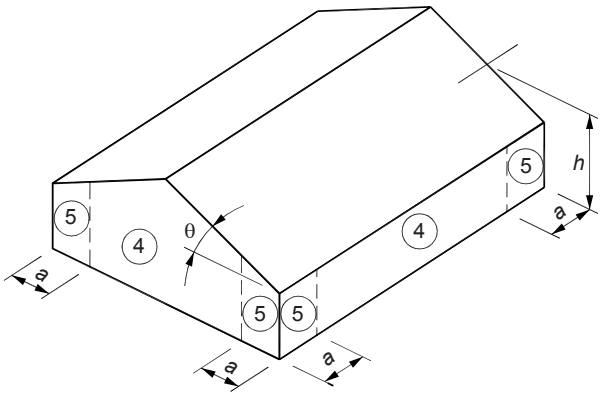

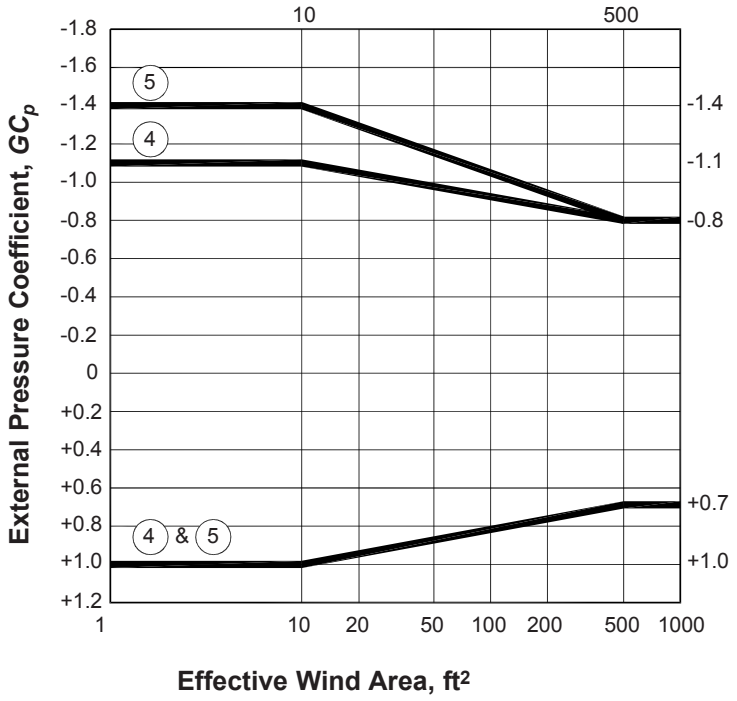

Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for walls shall be reduced by 10% when $\theta \leq 10°$.
6. Notation:
   - *a*: 10% of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft.
   - *h*: Mean roof height, in feet, except that eave height shall be used for $\theta \leq 10°$.
   - θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 3.6 Components and cladding/walls (Based on ASCE 7 – Figure 6-11A).

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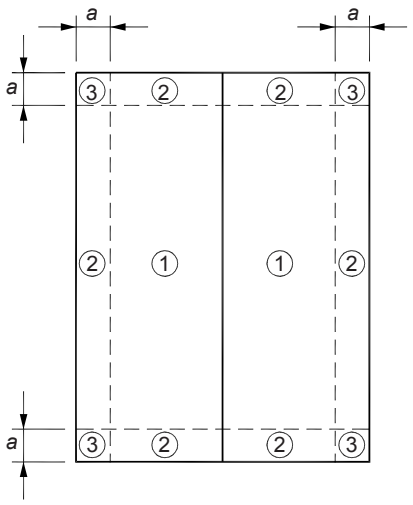

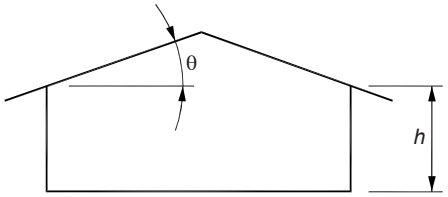

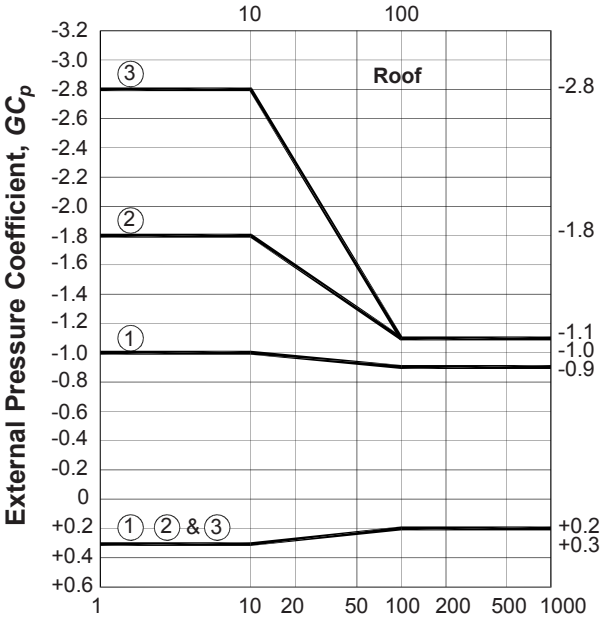

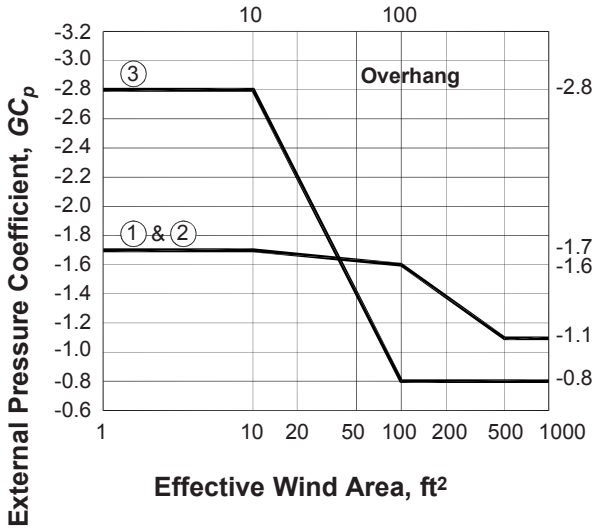

Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. If a parapet equal to or higher than 3 ft is provided around the perimeter of the roof with $\theta \leq 7°$, the negative values of GC_p in Zone 3 shall be equal to those for Zone 2 and positive values of GC_p in Zones 2 and 4 shall be set equal to those for wall Zones 4 and 5 respectively in ASCE 7 Figure 6-11A.
6. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
7. Notation:
   a: 10% of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft.
   h: Eave height shall be used for $\theta \leq 10°$.
   θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 3.6 (Continued) Components and cladding/gable roof $\theta \leq 7°$ (Based on ASCE 7 – Figure 6-11B).

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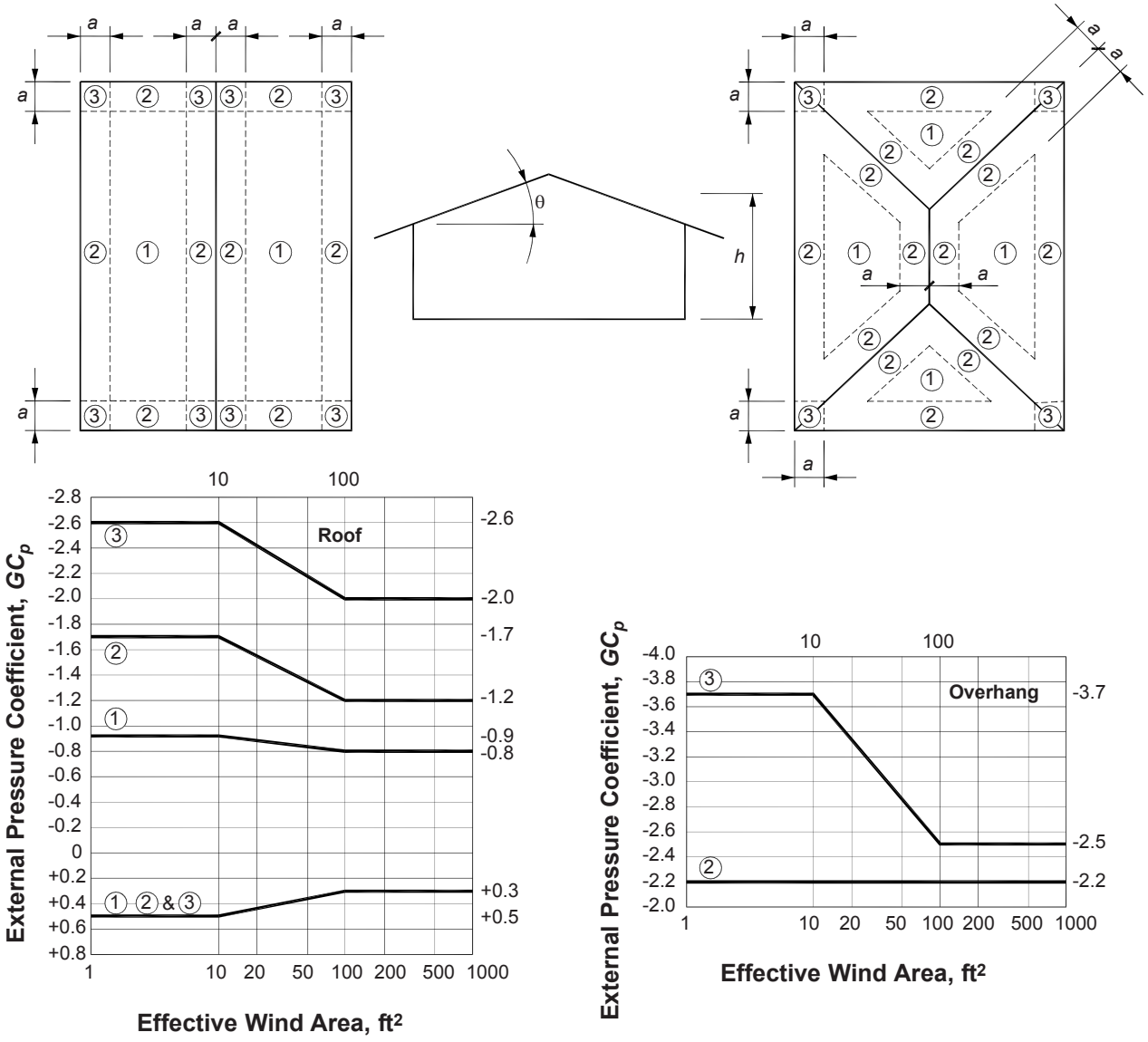

Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. For hip roofs with $7° < \theta \leq 27°$, edge/ridge strips and pressure coefficients for ridges of gabled roofs shall apply on each hip.
7. For hip roofs with $\theta \leq 25°$, Zone 3 shall be treated as Zone 2.
8. Notation:
   - *a*: 10% of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft.
   - *h*: Mean roof height, in feet, except that eave height shall be used for $\theta \leq 10°$.
   - θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 3.6 (Continued) Components and cladding/gable/hip roofs $7° < \theta \leq 27°$ (Based on ASCE 7 – Figure 6-11C).

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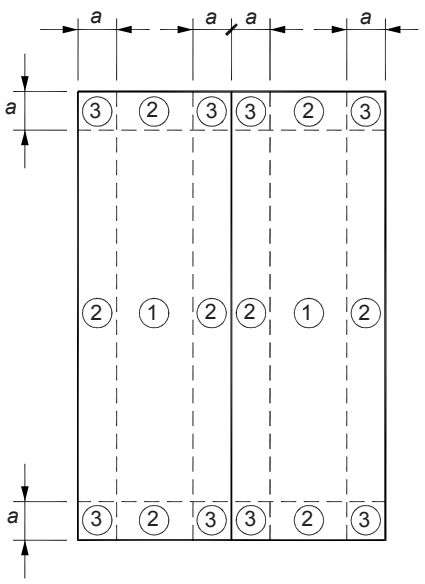

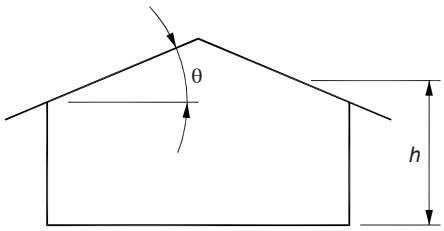

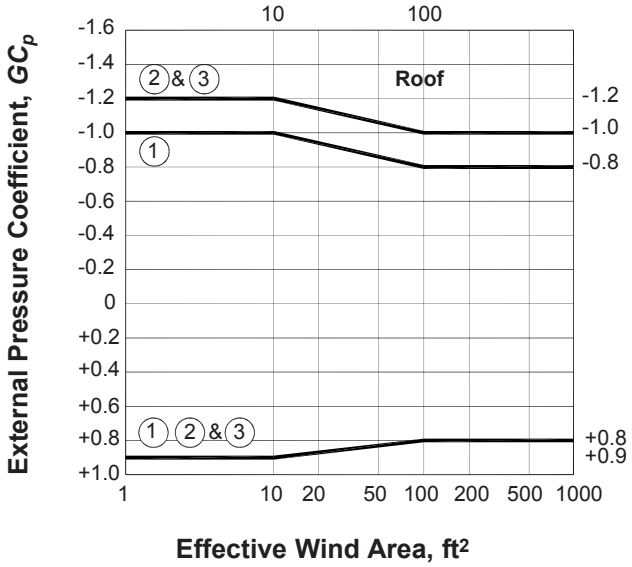

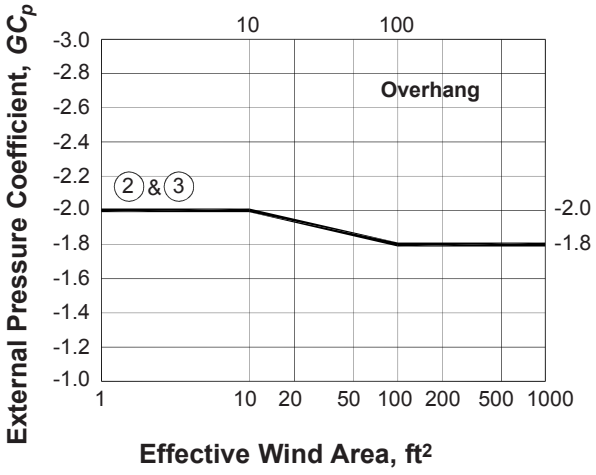

Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for roof overhangs include pressure contributions from both upper and lower surfaces.
6. Notation:
   - *a*: 10% of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft.
   - *h*: Mean roof height, in feet.
   - θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 3.6 (Continued) Components and cladding/gable roofs 27° < θ ≤ 45° (Based on ASCE 7 – Figure 6-11D).

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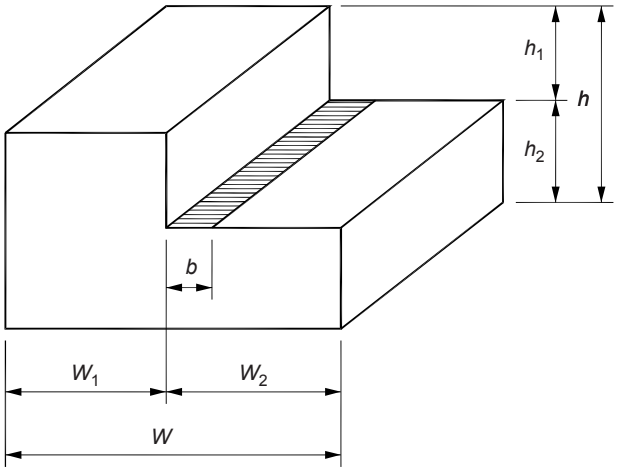

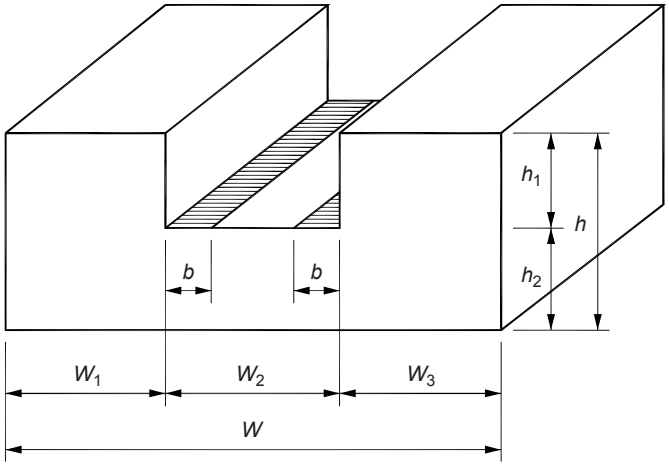

$h_1 \geq 10$ ft.
$b = 1.5\ h_1$
$b < 100$ ft.

$$\frac{h_i}{h} = 0.3 \text{ to } 0.7$$

$$\frac{W_i}{W} = 0.25 \text{ to } 0.75$$

Notes:

1. On the lower level of flat, stepped roofs shown in ASCE 7 Figure 6-12, the zone designations are pressure coefficients shown in ASCE 7 Figure 6-11B shall apply, except that at the roof-upper wall intersection(s), Zone 3 shall be treated as Zone 2 and Zone 2 shall be treated as Zone 1. Positive values of GC_p equal to those for walls in ASCE 7 Figure 6-11A shall apply on the cross-hatched areas shown in ASCE 7 Figure 6-12.
2. Notations:
   - b: $1.5h_1$ in ASCE 7 Figure 6-12, but not greater than 100 ft.
   - h: Mean roof height, in feet.
   - h_i: h_1 or h_2 in ASCE 7 Figure 6-12; $h = h_1 + h_2$; $h_1 \geq 10$ ft; h_i/h = 0.3 to 0.7.
   - W: Building width in ASCE 7 Figure 6-12.
   - W_i: W_1 or W_2 or W_3 in ASCE 7 Figure 6-12; $W = W_1 + W_2$ or $W_1 + W_2 + W_3$; W_i/W = 0.25 to 0.75.
   - θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 3.7 Components and cladding/stepped roofs (Based on ASCE 7 – Figure 6-12).

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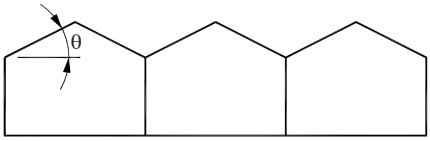
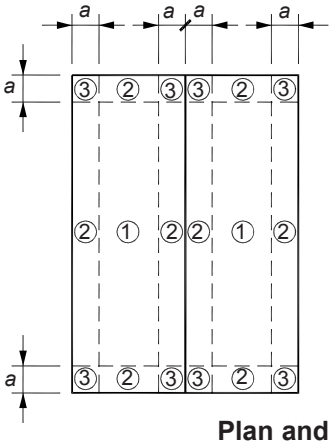
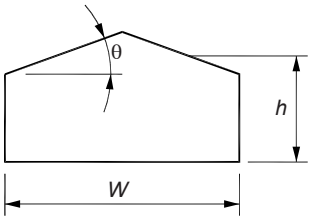
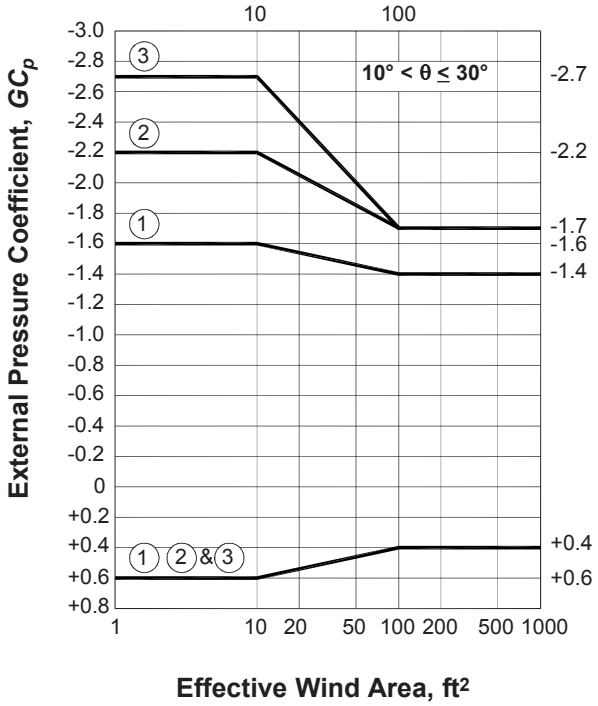


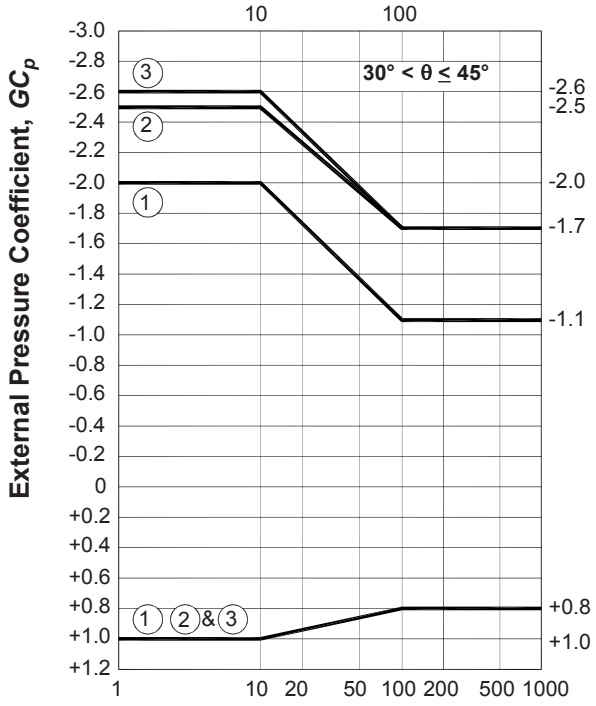
Elevation of Building
(2 or More Spans)





Plan and Elevation of a Single Span Module





Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square feet.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \leq 10°$, values of GC_p from ASCE 7 Figure 6-11 shall be used.
6. Notation:
   - *a*: 10% of least horizontal dimension of a single-span module or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension of a single-span module or 3 ft.
   - *h*: Mean roof height, in feet, except that eave height shall be used for $\theta \leq 10°$.
   - *W*: Building module width, in feet.
   - θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 3.8 Components and cladding/multispan gable roofs (Based on ASCE 7 – Figure 6-13).

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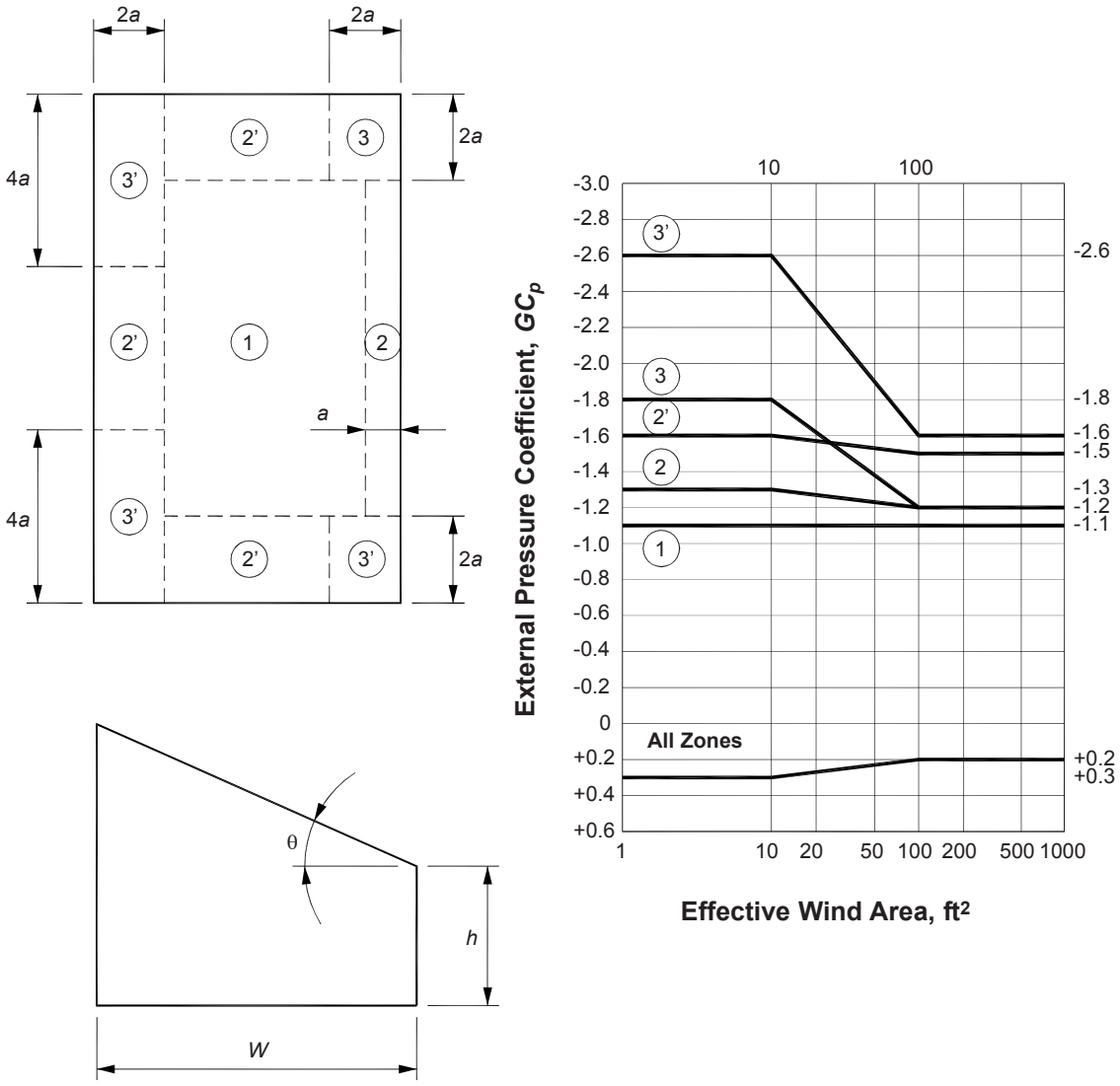

Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square feet.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \leq 3°$, values of GC_p from ASCE 7 Figure 6-11B shall be used.
6. Notation:
   - *a*: 10% of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft.
   - *h*: Eave height shall be used for $\theta \leq 10°$.
   - *W*: Building width, in feet.
   - θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 3.9 Components and cladding/monoslope roofs $3° < \theta \leq 10°$ (Based on ASCE 7 – Figure 6-14A).

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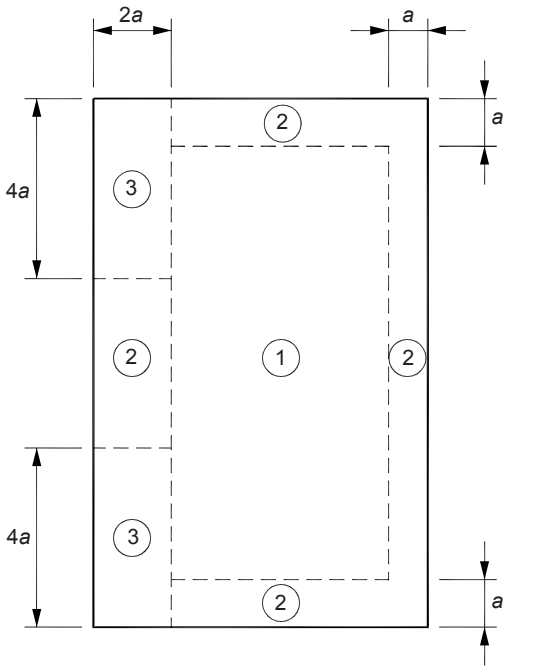

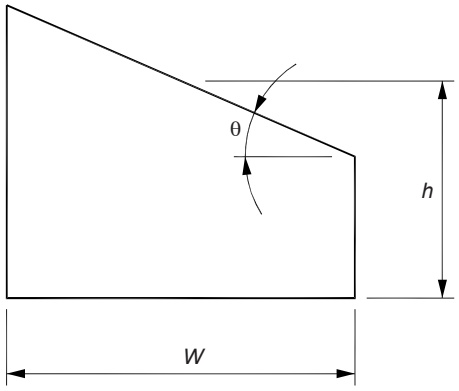

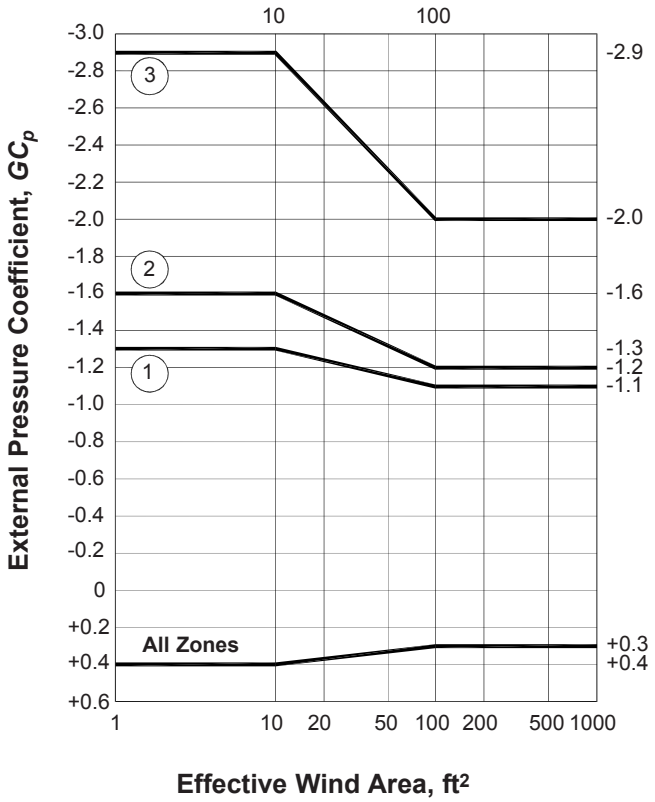

Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square feet.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Notation:
   - a: 10% of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft.
   - h: Mean roof height, in feet.
   - W: Building width, in feet.
   - θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 3.9 (Continued) Components and cladding/monoslope roofs $10° < \theta \leq 30°$ (Based on ASCE 7 – Figure 6-14B).

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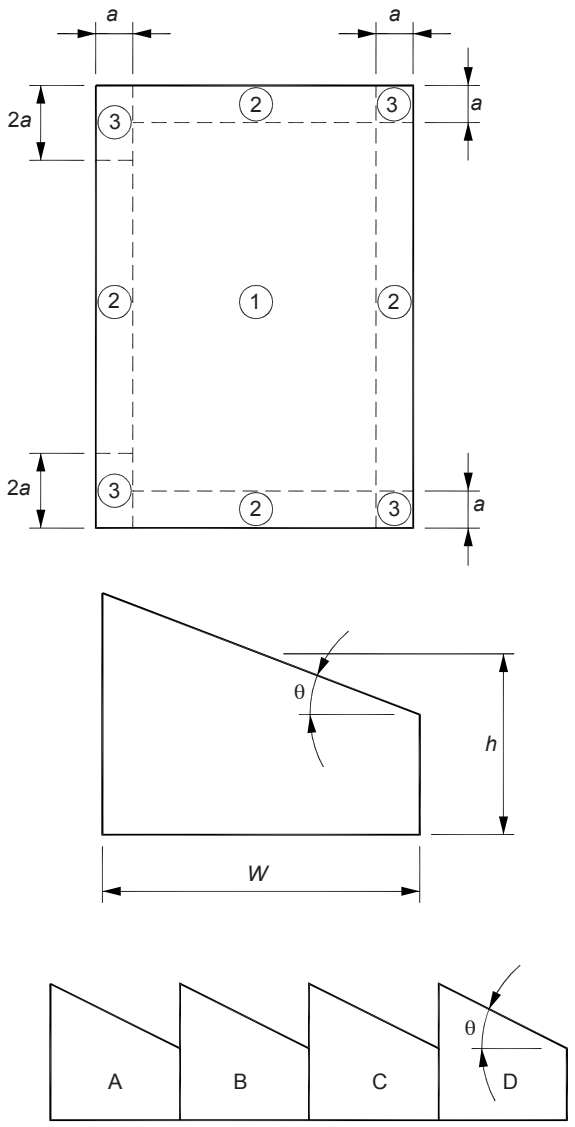
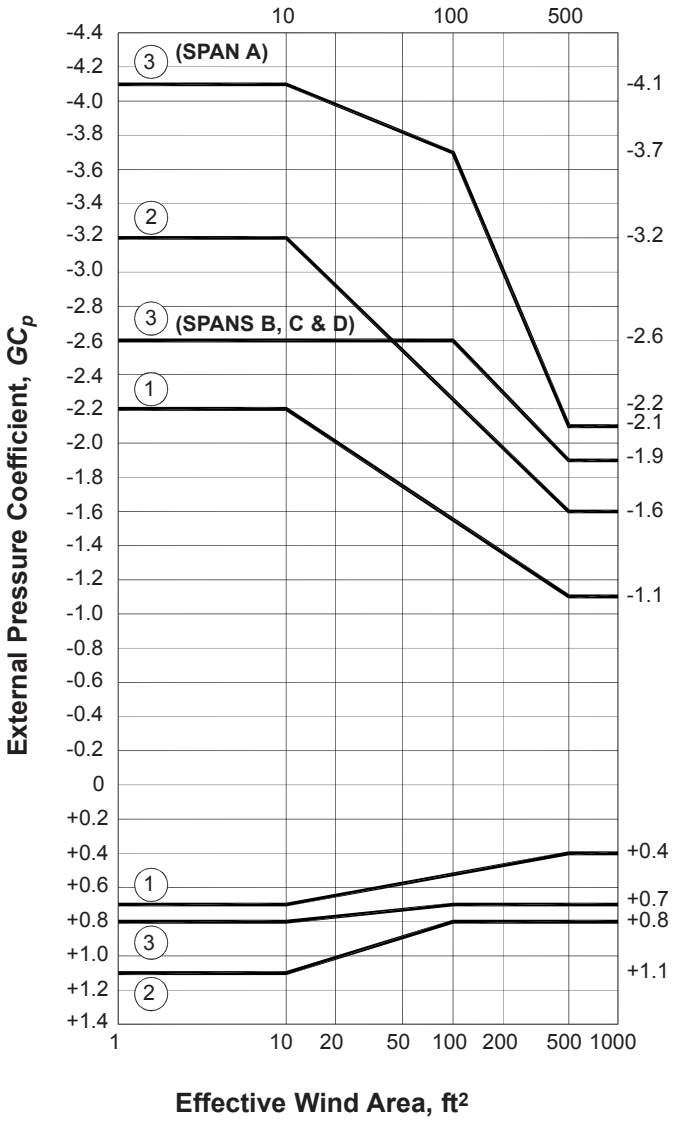


Elevation of Building
(2 or More Spans)



Notes:

1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square feet.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \leq 10°$, values of GC_p from ASCE 7 Figure 6-11 shall be used.
6. Notation:
   - *a*: 10% of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft.
   - *h*: Mean roof height, in feet, except that eave height shall be used for $\theta \leq 10°$.
   - *W*: Building width, in feet.
   - θ: Angle of plane of roof from horizontal, in degrees.

FIGURE 3.10 Components and cladding/sawtooth roofs (Based on ASCE 7 – Figure 6-15).

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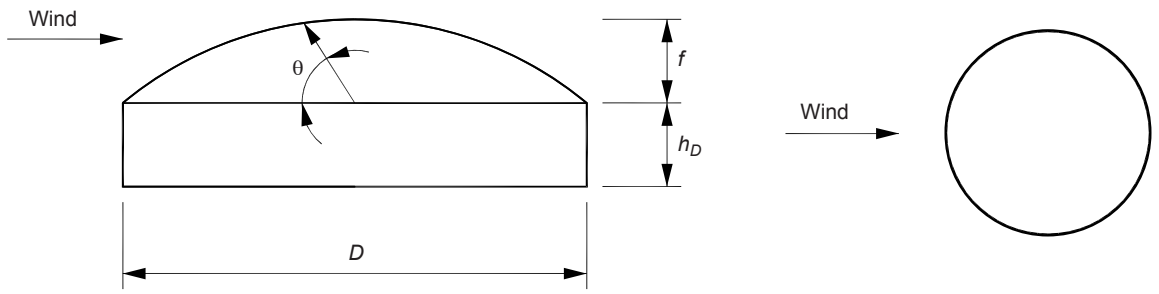

External Pressure Coefficients for Domes with a Circular Base			
θ, degrees	Negative Pressures	Positive Pressures	Positive Pressures
	0 – 90	0 – 60	61 – 90
GC_p	-0.9	+0.9	+0.5

Notes:

1. Values denote GC_p to be used with $q_{(h_D+f)}$ where $h_D + f$ is the height at the top of the dome.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. Each component shall be designed for maximum positive and negative pressures.
4. Values apply to $0 \leq h_D/D \leq 0.5$, $0.2 \leq f/D \leq 0.5$.
5. $\theta = 0$, degrees on dome springline, $\theta = 90$ degrees at dome center top point. f is measured from springline to top.

FIGURE 3.11 Components and cladding/domed roofs (Based on ASCE 7 – Figure 6-16).

Continued from page 73

1. Buildings with response characteristics from cross winds creating vortex shedding, flutter, or a location creating channeling or topographic effects – Method 3 (wind tunnel testing) per ASCE 7 Section 6.6.

2. Buildings without response characteristics from cross winds creating vortex shedding, flutter, or a location creating channeling or topographic effects:

   - Buildings, taller than 60 ft – Method 2, following ASCE 7 Section 6.5.12.4.2. for rigid buildings of all heights.
   - As an option, buildings with a height greater than 60 ft, but not exceeding 90 ft may be designed following ASCE 7 Section 6.5.12.4.3.

The abbreviated simple steps for Method 2 are to determine the following:

5. The velocity pressure at the mean roof height, q_h.

6. The external pressure coefficient, GC_p, from Figures 3.6 through 3.10.

7. The internal pressure coefficient, GC_{pi}, from Figure 3.4.

8. The design wind pressure, $p = q_h[(GC_p) – (GC_{pi})]$.

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3.5.4 WIND AND SEISMIC DETAILING

Regardless of whether wind or seismic loads result in the greatest demands on the structure, the designer must consider the provisions of IBC Section 1604.10 which provides for seismic detailing requirements and limitations.

IBC Section 1604.10

1604.10 Wind and seismic detailing. Lateral-force-resisting systems shall meet seismic detailing requirements and limitations prescribed in this code and ASCE 7, excluding Chapter 14 and Appendix 11A, even when wind code prescribed load effects are greater than seismic load effects.

This provision requires that seismic detailing provisions must be included even when wind loads govern. Specific attention must be given to prescriptive seismic reinforcement detailing requirements.

The seismic detailing requirements are discussed in Chapters 5 and 6.

EXAMPLE 3-A Wind Pressure Determination for Main Wind Force Resisting System.

Examples of calculation of wind pressures: The following examples illustrate the application of the wind criteria in Section 1609 and the ASCE 7 simplified procedure.

Example 1 [see Figure 3.12]

Given:

Enclosed, simple diaphragm building

V = 130 mph

Exposure Category = C

Roof slope = 7:12

Building width, W = 48 ft

Building length, L = 50 ft

Wall height = 18 ft

K_{zt} and I = 1.0

Find: Horizontal main windforce-resisting system (MWFRS) wind loads for end zones (A and B), using the simplified wind procedure.

First, the mean roof height, H, must be determined. From the building geometry, H is calculated:

H = 18 ft + 1/2 (7/12) (48 ft) = 25 ft

Roof slope = 7:12 = 30 degrees

Horizontal MWFRS wind loads for end zones A and B are determined using ASCE 7 Figure 6-2 for H = 30 ft and Exposure B. For V = 130 mph and roof angle of 30 degrees, the applicable end zone horizontal loads are:

Transverse direction

Zone A – wall p_{s30} = 30.1 psf

Zone B – roof p_{s30} = 20.6 psf

Longitudinal direction (Zone A) p_{s30} = 30.1 psf

These pressures must be modified for mean roof height, exposure category, topographic factor and importance factor using ASCE 7 Equation 6-1.

From ASCE 7 Figure 6-2, the height and exposure adjustment factor, λ, for H = 25 ft and Exposure C is 1.35.

Therefore, the design horizontal wind loads are:

p_s = (30.1 psf) x 1.35 x 1.0 = 40.6 psf (Zone A)

p_s = (20.6 psf) x 1.35 x 1.0 = 27.8 psf (Zone B)

These horizontal pressures on the MWFRS are to be applied as shown in Figures 3.13 and 3.14.

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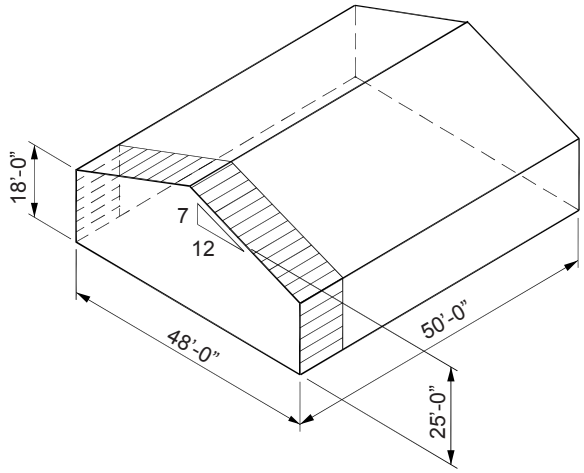
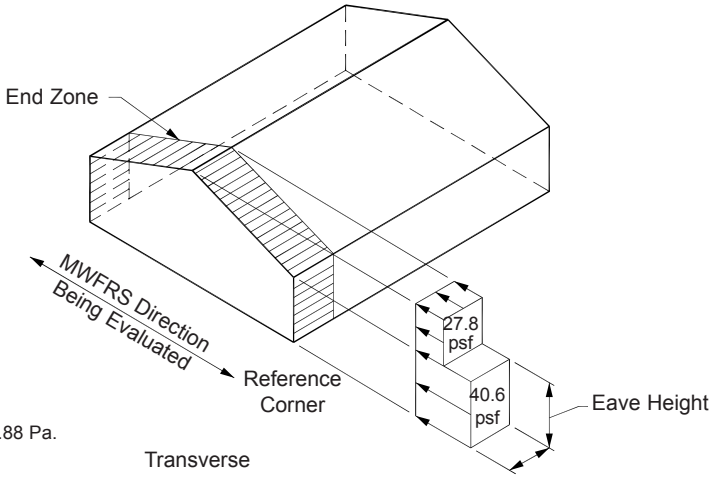


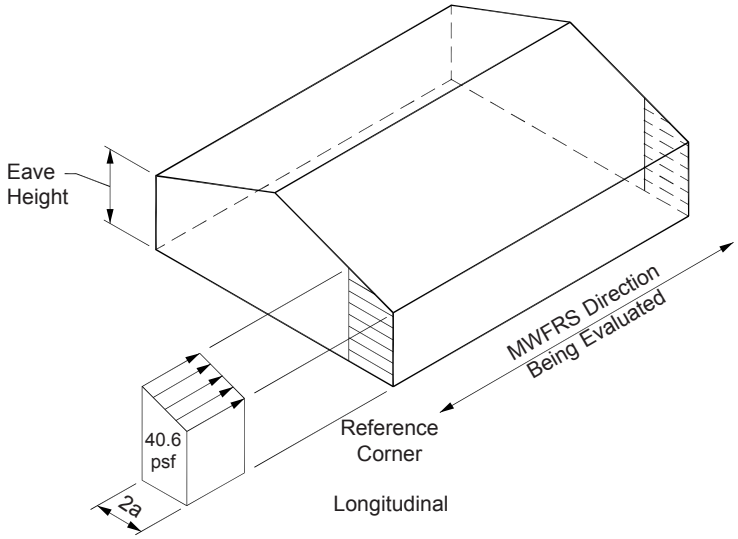
For SI: 1 foot = 304.8 mm.

FIGURE 3.12 Example 1 building (IBC Commentary Figure 1609.1.1(1)).



For SI: 1 pound per square foot = 47.88 Pa.

FIGURE 3.13 Horizontal MWFRS loads—transverse direction (IBC Commentary Figure 1609.1.1(2)).



For SI: 1 pound per square foot = 47.88 Pa.

FIGURE 3.14 Horizontal MWFRS loads—longitudinal direction (IBC Commentary Figure 1609.1.1(3)).

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EXAMPLE 3-B Additional Example of Wind Pressure Determination

Example 2

Given:

Basic wind speed, V = 120 mph

Building mean roof height, H = 45 ft

Exposure Category = D

I and K_{zt} = 1.0

Find: Design component and cladding wind pressure for a 20 sq ft window located in an edge strip of wall (Zone 5).

Obtain the component and cladding design wind pressures for a building with H = 30 ft and Exposure B, from ASCE 7 Figure 6-3. The window is located in a wall, Zone 5, with an effective wind area of 20 sq ft and V = 120 mph.

From ASCE 7 Figure 6-3, the design pressures are:

p_{net30} = + 24.7 psf, -32.4 psf

These pressures have to be modified for mean roof height, exposure category and importance factor using ASCE 7 Equation 6-2. From ASCE 7 Figure 6-3, the height and exposure adjustment factor, λ, for H = 45 ft and Exposure D is 1.78.

Therefore, the design wind pressures are calculated as:

p_{net} = (+24.7 psf) x 1.78 x 1.0 = +44 psf

p_{net} = (-32.4 psf) x 1.78 x 1.0 = -57.7 psf

3.6 SEISMIC LOADS

3.6.1 GENERAL

Earthquake loads are sudden, dynamic and can be of immense intensity. Rather than designing the structure to remain elastic during these extreme events, the design provisions of IBC, MSJC Code and ASCE 7 rely on the structure's ability to dissipate energy from the earthquake by responding inelastically. As a result, the seismic provisions include both design and detailing requirements, and loads applied to the structure are contingent upon the level of detailing provided in design.

Since design loads and building performance are dependent upon the ductility achieved through appropriate detailing, detailing provisions required for seismic design must be considered even if wind loads govern the design load combinations. This detailing will require compliance with prescriptive provisions for masonry design. For example, masonry walls whose design is controlled by wind loads will still need to include prescriptive reinforcement details as required by seismic provisions. If this detailed reinforcement is not provided, then the wall will not have an adequate ability to reduce loads through the dissipation of earthquake energy. As a result, the wall would experience higher forces than were considered in design, which could result in premature failure of the structure.

3.6.1.1 PRINCIPLES OF SEISMIC DESIGN

While the provisions only explicitly address structural performance in a major earthquake, the basic premise of seismic provisions is that code compliant structures should perform as follows:

- In minor earthquakes, structures should experience no damage.
- In moderate earthquakes, structural elements should experience no damage, but there may be some damage to non-structural elements.
- In major earthquakes, structural and non-structural damage may be severe, but the structure should not collapse. Designers rely on ductility and proper detailing to prevent collapse.

Fundamentally, seismic design involves three steps:

1. Defining the "design earthquake"
2. Determining the forces and displacements induced by the structure's response to the design earthquake, considering both elastic and inelastic behavior.
3. Evaluating the structure's response – Does the structure have sufficient strength? Are the displacements acceptable?

Iteration on the second and third steps is normally required to reach a final design.

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3.6.1.2 THE DESIGN EARTHQUAKE

As defined by ASCE 7, the "design earthquake" corresponds to a ground motion with effects equal to two-thirds of the "maximum considered earthquake" (MCE). The MCE corresponds to a ground motion with a 2 percent chance of being exceeded in fifty years; an event with an expected recurrence of 2,475 years.

While loads used with limit states design are normally associated with a 500 year recurrence, the writers of the code felt that would not be appropriate for seismic loads because in some regions a significant portion of the hazard associated with earthquakes comes from very strong, very rare events. Using the 2,475 year recurrence captures those hazards.

Recognizing the rarity of the 2,475 year event when compared to other loads, the code writers determined that it would reasonable to recognize that the actual strength of code compliant structures is greater than the design capacity. Real structures have a greater strength due to the factors of safety used in design, redundancy of structural systems and material over strengths. As a result of these factors, a structure would be expected to be able to resist loads up to 50% greater than it was designed to resist. For this reason, a structure can be designed for two-thirds of the MCE and yet still be expected to avoid collapse or significant damage when the structure is subjected to the MCE.

Ground motions induced by a given earthquake are quite complex. The building code has simplified that complexity into two parameters – the acceleration the earthquake induces into stiff, short period structures and the acceleration it induces into softer, structures having a period of one second.

The influence of local geology on ground motions also needs to be considered. The inter-relationship between the soil characteristics and the structure significantly affects the seismic forces imposed on a structure. A flexible building founded on a soft soil will respond to ground acceleration and will be subjected to high seismic forces because the building and soil will have longer periods. Conversely, a flexible building founded on a stiff, bedrock foundation will not be subjected to nearly as high forces because of the difference of periods between the foundation and the building. This phenomenon was evident in the Caracas earthquake of 1967 and the Mexico City earthquake of 1985. In Caracas, standard concrete framed, eight story apartment buildings were located throughout the city. When founded on hardpan soil or rock, these buildings performed very well, but buildings founded on soft alluvium soil were seriously damaged.

3.6.1.3 STRUCTURAL RESPONSE

The fundamental period of vibration is the single most important parameter for predicting the response of a structure in an earthquake. The fundamental period of a structure is dependent on the selected structural system and height of the building. The general term for the fundamental period of vibration (in seconds) is T.

The building code uses a response spectrum to predict the response of a building to an earthquake. Figure 3.15 is the design response spectrum used by ASCE 7 (Figure 11.4-1). The horizontal axis represents the building period; the vertical axis represents the acceleration induced into the structure by the earthquake.

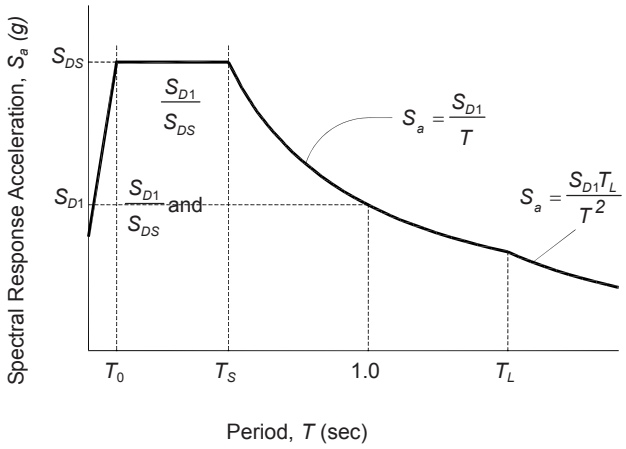

FIGURE 3.15 Design response spectrum (ASCE 7 Figure 11.4-1).

The response spectrum and building behavior can be thought of as having three regions:

- Stiff structures will move rigidly with the ground. The forces induced in the structure will be a result of ground acceleration. Behavior of these structures will be predicted by the portion of the response spectrum to the left of T_S in Figure 3.15. This portion of the response spectrum is referred to as the "constant acceleration" region.

- The tops of very flexible structures will remain in place, while the base displaces with the ground. Behavior of these structures will be predicted by the portion of the response spectrum to the right of T_L in Figure 3.15. This portion of the response spectrum is referred to as the "constant displacement" region.

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- Forces induced into flexible structures will be a result of the velocity of the ground. Behavior of these structures will be predicted by the portion of the response spectrum between T_S and T_L in Figure 3.15. This portion of the response spectrum is referred to as the "constant velocity" region.

Behavior of most real structures is more complex than this as most real structures have more than one mode of vibration. During an earthquake, a building will vibrate in at least one mode of vibration for a period of time (See Figure 3.16). It may vibrate only back and forth in the simple first mode of vibration (characterized by *T*) or it may vibrate in higher modes depending on the ground motion and duration of an earthquake. Since earthquakes produce erratic ground motions in various directions, the response of most buildings includes higher modes of vibration, allowing one part of the building to move in one direction while another part of the building moves in another direction. Note, however, that just after an earthquake, buildings may vibrate into lower modes which may cause even more severe stresses than those generated during the earthquake.

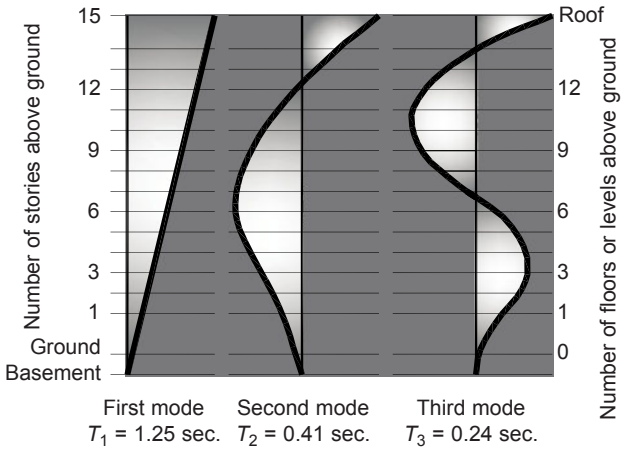

FIGURE 3.16 Three modes of vibration that a building may respond to in an earthquake (Blume, Newmark and Corning, 1961).

The code recognizes several ways to account for these higher mode effects. For the equivalent lateral force procedure that is the basis of discussion in this section, the higher mode effects are addressed in distribution of forces over the building height. This is addressed in more detail below.

No discussion of structural response would be complete without addressing damping and ductility.

Damping refers to the ability of the structure to dissipate energy while responding elastically. Damping is assumed to be 5% of critical for all structures, and is already accounted for in the MCE ground motions provided by the code. The designer does not normally need to account for damping.

Ductility refers the ability of the structure to dissipate energy through inelastic response. For reinforced masonry, most of the ductility is a result of reinforcing steel yielding in tension, or the compression of a masonry assembly that has been well confined by reinforcing steel. The designer is able to control the amount of ductility through detailing the lateral force resisting system. The more ductile a system is, the lower forces it can be designed for. Ductility is directly accounted for in the design process through the "R" factor.

3.6.1.4 INTRODUCTION TO ASCE 7

ASCE 7 forms the basis criteria for seismic design. Every structure, including the nonstructural components, must be designed in accordance with ASCE 7 (referenced by IBC Section 1613.1). The seismic provisions are organized into the following chapters in ASCE 7:

Chapter	Subject
11	Seismic Design Criteria
12	Seismic Design Requirements for Building Structures
13	Seismic Design Requirements for Nonstructural Components
15	Seismic Design Requirements for Nonbuilding Structures
16	Seismic Response History Procedures
17	Seismic Design Requirements for Seismically Isolated Structures
18	Seismic Design Requirements for Structures With Damping Systems
19	Soil Structure Interaction for Seismic Design
20	Site Classification Procedure for Seismic Design
21	Site-Specific Ground Motion Procedures for Seismic Design
22	Seismic Ground Motion and Long Period Transition Maps
23	Seismic Design Reference Documents

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Obviously, not all provisions of the IBC and ASCE 7 chapters can be covered in detail in this text, so key items to allow for masonry design for the seismic provisions will be addressed. This chapter will concentrate on the Equivalent Lateral Force Procedure and detailing requirements.

On of the fundamental concepts of ASCE 7, the IBC, and MSJC Code is the use of Seismic Design Categories (SDC) to establish basic requirements for structural design and detailing. The stronger the ground motion and the more critical the use of the building, the more stringent are the design and detailing requirements. SDC's are termed A, B, C, D, E, and F, with SDC A having the fewest requirements and SDC F having the most.

There are only a few areas in the United States where the design accelerations are low enough for structures to be classified as SDC A.

Structures are also classified by ASCE 7 for design based upon their use and importance. Essential facilities such as hospitals, fire stations, emergency centers and communication centers must remain functioning in a catastrophe and are therefore designed for greater safety factors using I values.

ASCE 7 recognizes the following methods for determination of structural response to earthquakes:

1. Equivalent Lateral Force Procedure (Section 12.8)
2. Modal Response Spectrum Analysis (Section 12.9)
3. Linear Response History Procedure (Section 16.1)
4. Nonlinear Response History Procedure (Section 16.2).

The most common of these techniques of is the equivalent lateral force procedure. Using this procedure, the dynamic seismic force is translated into an equivalent static force on the building and is distributed throughout the height of the building to each resisting element. The static seismic force is assumed to be an external base shear force, *V*, that is applied to the structure.

Use of the equivalent lateral force procedure assumes that the seismic force is an external force, *V*, applied to the structure. This is similar to design of wind forces on a building.

The balance of this chapter is based on the use of the equivalent lateral force procedure. Limitations on the use of the equivalent lateral force procedure are contained in ASCE 7 Section 12.7.

3.6.2 BASE SHEAR, *V*

When using the equivalent lateral force procedure, the seismic base shear force, *V*, is determined as follows:

$$V = C_s W \qquad \text{(ASCE Eq 12.8-1)}$$

Where:

C_s = the seismic response coefficient

W = the effective seismic weight.

As defined by ASCE 7 Section 12.7.2, the effective seismic weight of the building needs to include the total dead load imposed on the structure, 25% of any storage loads, an allowance for any movable partitions that are not less than 10 psf, the operating weight of permanent equipment, and 20% of the snow load where the flat roof snow load exceeds 30 psf.

The seismic response coefficient is determined using the following set of equations:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \qquad \text{(ASCE Eq 12.8-2)}$$

Where $\boldsymbol{C_s}$ **need not exceed** the following values:

$$C_s \le \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad \text{for } T \le T_L \qquad \text{(ASCE Eq 12.8-3)}$$

$$C_s \le \frac{S_{D1}T_L}{T^2\left(\frac{R}{I}\right)} \quad \text{for } T > T_L \qquad \text{(ASCE Eq 12.8-4)}$$

And where C_s shall not be less than the following values:

$$C_s \ge 0.01 \qquad \text{(ASCE Eq 12.8-5)}$$

$$C_s \ge \frac{0.5S_1}{\left(\frac{R}{I}\right)} \quad \text{for structures with } S_1 > 0.6g$$

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Where:

S_{DS} = Design spectral response acceleration parameter at short periods

S_{D1} = Design spectral response acceleration parameter at a 1 second period

S_1 = MCE spectral response acceleration at a 1 second period

R = Response modification coefficient

I = Importance Factor

T = Fundamental period of the building

T_L = Long-period transition period

These terms are explained in more depth in the following sections.

3.6.2.1 DESIGN GROUND MOTION (S_{DS}, S_{D1})

The first step for seismic design of a structure is the determination of the design spectral response accelerations S_{DS} and S_{D1}.

The following steps are needed to determine the design spectral response accelerations:

1. Determine the mapped spectral response acceleration for the maximum considered earthquake (MCE) at short (S_S) and one-second intervals (S_1).
2. Adjust the spectral response accelerations for the MCE to account for the effects of site geology:
   - Determine the soil site class.
   - Determine the site coefficients F_a and F_v from Tables 3.14 and 3.15, respectively.
   - Calculate the spectral response accelerations for the MCE, accounting for the effects of site class:

$$S_{MS} = F_a S_S \quad \text{(IBC Eq 16-37)}$$

$$S_{M1} = F_v S_1 \quad \text{(IBC Eq 16-38)}$$

3. Determine the five-percent damped design spectral response acceleration at short periods, S_{DS}, and at the one-second period, S_1, as follows:

$$S_{DS} = (2/3)S_{MS} \quad \text{(IBC Eq 16-39)}$$

$$S_{D1} = (2/3)S_{M1} \quad \text{(IBC Eq 16-40)}$$

3.6.2.1.1 MCE GROUND MOTION (S_S, S_1)

The IBC and ASCE 7 provide maps depicting ground motion resulting from the MCE. Figures 3.17 and 3.18 duplicate maps of the United States from the IBC which delineates the spectral response accelerations resulting from the maximum considered earthquake ground motion for structures with periods of 0.2 and 1.0 seconds, having 5% damping, and located Site Class B soil.

The spectral response accelerations shown on Figure 3.17 are defined as follows:

- S_S = 5% damped spectral response acceleration due to the MCE at short periods
- S_1 = 5% damped spectral response acceleration due to the MCE at a one-second period

Since the scale of the maps is somewhat large, it may be useful to use the following procedure to determine MCE ground motions:

- Determine the latitude and longitude of the building address by using the website: www.geocoder.us/.
- Input the latitude and the longitude into the software developed by USGS to determine S_S and S_1. The website access is www.earthquake.usgs.gov/research/hazmaps/design/.

The software allows the user to define site location by zip code or by latitude and longitude. As stated in the software documentation, USGS recommends defining the site location by latitude and longitude because "In some regions, there can be substantial variation between the spectral values at a zip code centroid and those at individual structures at some sites and excessively conservative designs at other sites."

USGS software will calculate the values of S_{DS} and S_{D1} if the user enters the site class.

3.6.2.1.2 SITE CLASS AND COEFFICIENTS (F_a, F_v)

The mapped values of accelerations due to the MCE are based on the assumption that the structure is founded on rock. Softer soils will typically amplify and stiffer soils typically de-amplify these accelerations.

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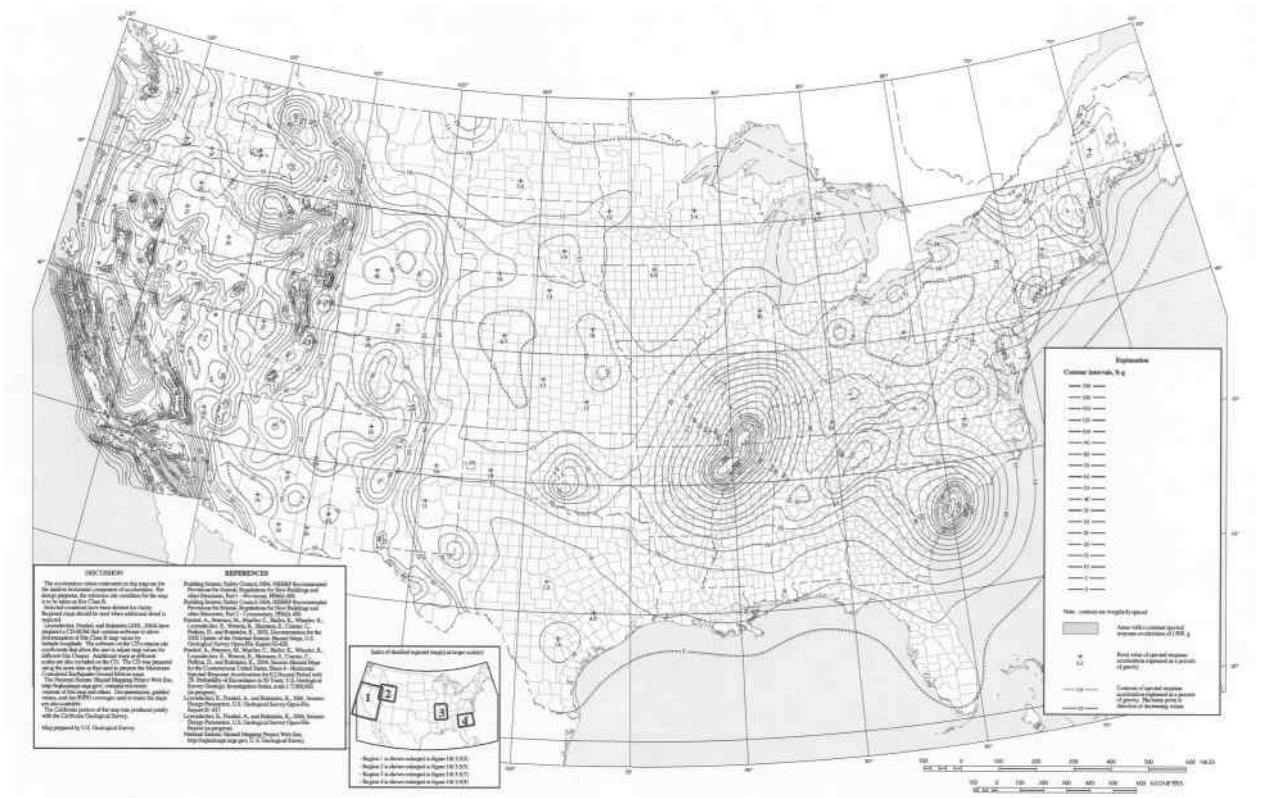

FIGURE 3.17 **Maximum considered earthquake ground motion for the conterminous United States of 0.2 Sec Spectral Response Acceleration (5% of critical damping), Site Class B (2006 IBC Figure 1613.5(1).**

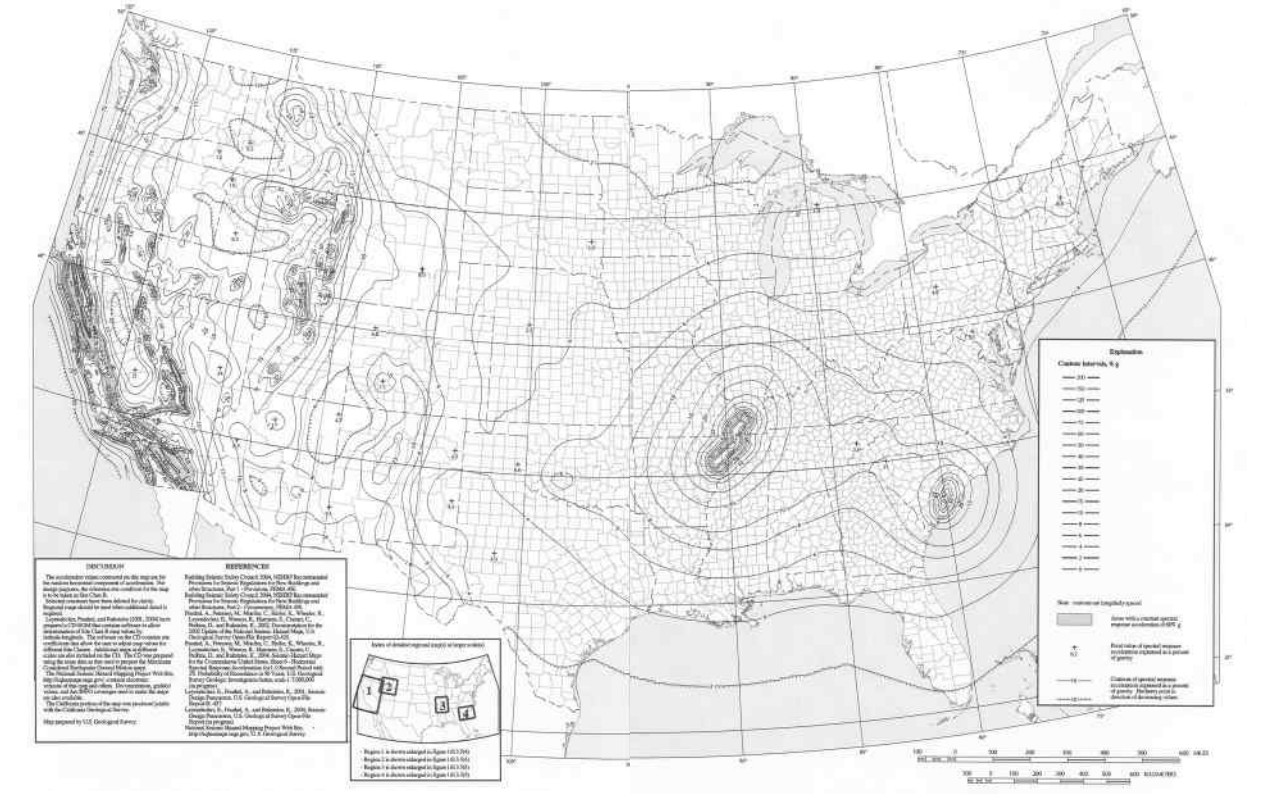

FIGURE 3.18 **Maximum considered earthquake ground motion for the conterminous United States of 1.0 Sec Spectral Response Acceleration (5% of critical damping), Site Class B (2006 IBC Figure 1613.5(2).**

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TABLE 3.13 Site Class Definitions (IBC Table 1613.5.2)

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 FT, SEE IBC SECTION 1613.5.5		
		Soil shear wave velocity, $\overline{v}_s$, (ft/s)	Standard penetration resistance, $\overline{N}$	Soil undrained shear strength, $\overline{s}_u$, (psf)
A	Hard rock	$\overline{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \overline{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \overline{v}_s \leq 2{,}500$	$\overline{N} > 50$	$\overline{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \overline{v}_s \leq 1{,}200$	$1.5 \leq \overline{N} \leq 50$	$1{,}000 \leq \overline{s}_u \leq 2{,}000$
E	Soft soil profile	$\overline{v}_s < 600$	$\overline{N} < 15$	$\overline{s}_u < 1{,}000$
E	–	Any profile with more than 10 ft of soil having the following characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\overline{s}_u < 500$ psf		
F	–	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ ft of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ ft with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ ft)		

For SI: 1 ft = 304.8 mm, 1 sq ft = 0.0929 m^2, 1 lb per sq ft = 0.0479 kPa. N/A = Not applicable

IBC and ASCE 7 account for the effect of soil type on the ground motion though the concept of site class. Site class is determined in accordance with IBC Section 1613.5.2 and depends upon the soil properties at the site. The site class is typically determined by a geotechnical engineer, using the site class definitions as shown in Table 3.13,

Where,

$\overline{v}_s$ = average shear wave velocity for soils beneath the foundation at large strains,

$\overline{N}$ = average standard penetration resistance (per ASTM D1586) for the top 100 ft of soil, which is N_{ch} for cohesionless soils.

$\overline{s}_u$ = average undrained shear strength in the top 100 ft of soil

If soil properties are not known in sufficient detail to determine the site class, then Class D may be used as the default site class, if approved by the building official.

Once the site class has been established, the site coefficients which will be used to adjust the MCE spectral response accelerations to account for the effect of ground conditions can be determined.

The short period acceleration will be adjusted by use of the site coefficient, F_a, as given in Table 3.14.

TABLE 3.14 Values of Site Coefficient, F_a[1] (IBC Table 1613.5.3(1))

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note 2	Note 2	Note 2	Note 2	Note 2

1. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_S.
2. Values shall be determined in accordance with ASCE 7 Section 11.4.7.

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The 1-second period acceleration will be adjusted by use of the site coefficient, F_v, as shown in Table 3.15.

TABLE 3.15 Values of Site Coefficient, F_v[1] (IBC Table 1613.5.3(2))

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note 2	Note 2	Note 2	Note 2	Note 2

1. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
2. Values shall be determined in accordance with ASCE 7 Section 11.4.7.

Taking the acceleration-related short-period site coefficient F_a, times S_S gives S_{MS}, which is the five-percent damped soil-modified MCE spectral response acceleration for short periods. The velocity related long-period site coefficient F_v times S_1 gives S_{M1}, which is the five-percent damped soil-modified MCE spectral response acceleration at the one-second period.

3.6.2.2 SEISMIC DESIGN CATEGORY (SDC)

Once the design level acceleration parameters S_{DS} and S_{D1} are determined and the Occupancy Category is known, it is possible to assign the Seismic Design Category (SDC) from Tables 3.16 and 3.17. The highest SDC from the two tables is the category assigned to the building design, unless IBC Section 1613.5.6.1 applies.

IBC Section 1613.5.6.1

1613.5.6.1 Alternative seismic design category determination. Where S_1 is less than 0.75, the seismic design category is permitted to be determined from Table 1613.5.6(1) alone when all of the following apply:

1. in each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 T_s determined in accordance with Section 11.4.5 of ASCE 7.
2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, C_s.
4. The diaphragms are rigid as defined in Section 12.3.1 in ASCE 7 or for diaphragms that are flexible, the distance between vertical elements of the seismic-force-resisting system does not exceed 40 feet (12 192 mm).

TABLE 3.16 Seismic Design Category Based On Short-Period Response Accelerations (IBC Table 1613.5.6(1))

VALUE OF S_{DS}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

TABLE 3.17 Seismic Design Category Based On 1-Second Period Response Acceleration (IBC Table 1613.5.6(2))

VALUE OF S_{D1}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

Once the Seismic Design Category has been determined, the designer should review the proposed structural system for irregularities in accordance with ASCE 7 Section 12.3.

3.6.2.3 RESPONSE MODIFICATION FACTOR (R)

The response modification factor represents how effective the structural system is in reducing seismic forces through dissipation energy by inelastic actions.

The IBC, ASCE 7 and MSJC Code recognize the following types of seismic force resisting systems for reinforced masonry construction:

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- **Bearing wall systems** in which the walls resist both lateral and gravity loads These are the most commonly used lateral force resisting systems, and are the focus of discussion in this book.

- **Building frame systems**, according to the definition in ASCE 7 Section 11.2, must have "a structural system with an essentially complete space frame providing support for vertical loads." This implies that nearly all portions of the floor would need to be supported by columns and beams. These systems are rarely encountered in masonry wall systems.

- **Dual systems**, according to the definition in ASCE 7 Section 11.2, must have "a structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment-resisting frames and shear walls." As with the building frame systems, nearly all portions of the floor would need to be supported by columns and beams. In addition, a concrete or steel moment frame would need to be provided and designed to take at least 25% of the load. These systems are not typical in masonry wall systems.

Within each of these system types, there are systems classified as special, intermediate, and ordinary. These terms can be described as follows:

- **Special systems** have the most stringent prescriptive detailing requirements, which makes them the most ductile systems, resulting in the greatest reduction in seismic forces (highest R value).

- **Intermediate systems** offer level of detailing, ductility and force reduction between special and ordinary systems.

- **Ordinary systems** have the least stringent prescriptive detailing requirements, which makes them the least ductile systems, resulting in the least reduction in seismic forces (lowest R value).

Available seismic force resisting systems for reinforced masonry construction are listed in Table 3.18, excerpted from ASCE 7 Table 12.2-1.

In selecting a structural system, the designer must first identify those systems that are permissible given the building's Seismic Design Category. This information is contained in the right columns of Table 3.18, under the heading "Structural System Limitations and Building Height (ft) Limit." The designer must select from the systems listed as NL (Not Limited) or with height limit not greater than the building height. NP indicates Not Permitted.

Once the lateral force resisting system has been selected, the response modification factor, *R*, is simply determined from the Table.

3.6.2.4 BUILDING PERIOD (T)

ASCE 7 recognizes two ways to determine the building period:

- Through a "properly substantiated analysis" (ASCE 7 Section 12.8.2). Note that the period determined by analysis is capped for the determination of forces. The advantage of this approach is that generally it will result in longer periods than the approximate method described next, which can result in lower forces,

- Through use of equations for approximated fundamental period contained in ASCE 7 Section 12.8.2.1. For masonry shear wall structures we have two options:

$$T_a = 0.02 h_n^{0.75} \qquad \text{(ASCE Eq 12.8-7)}$$

Where h_n is the height of the highest level.

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \qquad \text{(ASCE Eq 12.8-9)}$$

Where

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_1} \right)^2 \frac{A_i}{1 + 0.83 \left(\frac{h_i}{D_i} \right)^2}$$

(ASCE Eq 12.8-10)

Where

A_B = Area of base of structure, ft^2

A_i = Web area of shear wall "*i*" in ft^2

D_i = Length of shear wall "*i*" in ft

h_i = Height of shear wall "*i*" in ft

x = Number of shear walls in the building resisting lateral forces in the direction under consideration.

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TABLE 3.18 Design Coefficients and Factors for Seismic Force-Resisting Systems (ASCE 7, Excerpt from Table 12.2-1)

Seismic Force-Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R[1]	System Overstrength Factor, Ω_0[6]	Deflection Amplification Factor, C_d[2]	Structural System Limitations and Building Height (ft) Limit[3]				
					Seismic Design Category				
					B	C	D[4]	E[4]	F[5]
A. BEARING WALL SYSTEMS									
7. Special reinforced masonry shear walls	14.4 and 14.4.3	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4 and 14.4.3	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
B. BUILDING FRAME SYSTEMS									
17. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
18. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP
19. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
10. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP

1. Response modification coefficient, R, for use throughout the standard. Note R reduces forces to a strength level, not an allowed stress level.
2. Reflection amplification factor, C_d, for use in ASCE 7 Sections 12.8.6, 12.8.7, and 12.9.2.
3. NL = Not Limited and NP = Not Permitted. For metric units use 30.5 m for 100 ft and use 48.8 m for 160 ft. Heights are measured from the base of the structure as defined in ASCE 7 Section 11.2.
4. See ASCE 7 Section 12.2.5.4 for a description of building systems limited to buildings with a height of 240 ft (73.2 m) or less.
5. See ASCE 7 Section 12.2.5.4 for building systems limited to buildings with a height of 160 ft (48.8 m) or less.
6. The tabulated value of the overstrength factor, Ω_0, is permitted to be reduced by subtracting one-half for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.

Most masonry structures tend to be stiff enough that their period is in the short period range, regardless of how the period is determined. Since the demands on these structures are insensitive to the the way the period is calculated, the calculation of the period should be as simple as possible.

ASCE 7 equation 12.8-7 can be used for this purpose.

3.6.2.5 IMPORTANCE FACTOR (I)

ASCE 7 assigns an Importance Factor, I, to each structure, based on the occupancy category. Occupancy category is discussed in more detail in Section 3.4.

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TABLE 3.19 Importance Factors (ASCE 7, Table 11.5-1)

Occupancy Category	I
I or II	1.0
III	1.25
IV	1.5

EXAMPLE 3-C Determination of Seismic Spectral Acceleration Value

PART A:

Using seismic maps, determine spectral acceleration values for an essential facility (such as a hospital) to be located at the following latitudes and longitudes (for different areas of the US):

Item	State*	Latitude (°N)	Longitude (-°W)	S_S	S_1
1	CA	35	119	2.319	0.803
2	CA	35.3	119	1.239	0.462
3	IA	42	93.8	0.070	0.040
4	MO	37	89.6	2.296	0.600
5	SC	33	80	2.208	0.559

* Note that the state is only a general information item, solution is based upon exact latitude and longitude.

From the U.S. Geological Survey (USGS) website: www.earthquake.usgs.gov/research/hazmaps/design/ Type in the five latitude and longitude locations. The website ground motion parameter calculator will determine the S_S and S_1 from the seismic maps and the calculator portion will determine the parameters needed in Part B below. Note the amount of significant change that occurs in the S_S and S_1 by geographic location. Also, a very small change in location, (from Item 1 to Item 2) can result in a significant change in the spectral acceleration values. All the items shown are for higher seismic design, except for Item 3 which is representative of many areas of the country where earthquake forces are not large.

PART B:

Selecting Item 2 in Part A, determine the response parameters, design response spectrum, design parameters and design category. The geotechnical engineer has determined that the soil properties indicate that the project is located on a Site Class C.

Referring to Table 3.14 for F_a and Table 3.15 for F_v, for site Class C determine that:

$F_a = 1.0$

$F_v = 1.36$ (interpolating)

Thus, the spectral response accelerations are:

$S_{MS} = F_a S_S = 1.0(1.239) = 1.24$ g

$S_{M1} = F_v S_1 = 1.36(0.462) = 0.63$ g

And, the 5% damped, design spectral response values are:

$S_{DS} = (2/3)(1.24) = 0.826$ g

$S_{D1} = (2/3)(0.63) = 0.42$ g

The control periods are:

$T_o = 0.2(S_{D1}/S_{DS}) = 0.2(0.42/0.826) = 0.102$ sec.

$T_s = S_{D1}/S_{DS} = 0.42/0.826 = 0.51$ sec.

Based upon Tables 3.16 and 3.17, the Seismic Design Category is D. The resulting Design Response Spectrum is shown in Figure 3.15.

3.6.3 VERTICAL DISTRIBUTION OF TOTAL SEISMIC FORCES

Once the seismic base shear has been determined, the next step is to distribute the base shear over the height of the building. The vertical distribution of seismic forces must account for dynamic action and response of the structure. In the equivalent lateral force procedure as defined by ASCE 7, the following equation is used to distribute seismic forces:

$$F_x = C_{vx} V \qquad \text{(ASCE Eq 12.8-11)}$$

Where F_x is the force to be applied at any level "x", C_{vx} is the vertical distribution factor and V is the base shear. The vertical distribution factor for each level is determined from the following equation:

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(ASCE Eq 12.8-12)}$$

Where:

w_i and w_x = the weight of story "I" and "x," respectively

h_i and h_x = the height of story "I" and "x," respectively from the base of the building

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k = an exponent that is dependent on the fundamental period of the structure, T:

$T \leq 0.5$ seconds, $k = 1$

$T \geq 2.5$ seconds, $k = 2$

Linear interpolation is used to determine the value of k for structures having a period between 0.5 and 2.5 seconds.

The exponent "k" is how the equivalent lateral force procedure accounts for higher mode effects. Short, stiff buildings respond primarily in the first mode and have k value of 1. Very flexible buildings having a k value of 2 are strongly influence by higher modes. The difference in the vertical distribution of seismic forces between a stiff building and a flexible building can be seen in Figure 3.19.

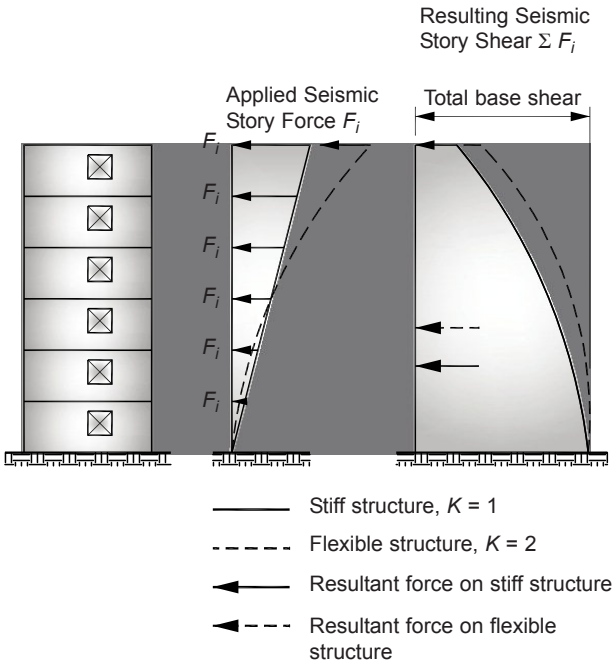

FIGURE 3.19 Vertical distribution of seismic forces.

3.6.4 SEISMIC LOADS ON STRUCTURAL ELEMENTS

3.6.4.1 ELEMENTS

Individual elements of a building such as walls, parapets, or partitions, must be designed to resist forces due to seismic motions. This applies both to elements such as interior partition walls that are not part of the lateral force resisting system as well as to elements that are part of the lateral force resisting system such as shear walls.

In the case of the shear walls, the design for in-plane forces will be based on forces determined for the lateral force resisting system as described above. Out-of-plane forces also need to be considered and are determined as described in this section.

Seismic forces on elements, F_p, are calculated using a horizontal force coefficient that is contingent upon; the component importance factor, I_p, the appropriate spectral response acceleration coefficient, $S_{DS,}$ the component amplification factor, a_p, the component response modification factor, R_p and the weight of the component, W_p. Criteria for masonry walls is contained in ASCE 7 Section 13.5 "Architectural Components".

The design of interior partition walls may also be governed by IBC Section 1607.13 which requires that all interior walls and partitions in excess of 6 ft in height be designed for a minimum lateral force perpendicular to the wall of 5 psf. This load should be treated as an allowable load and a wind load factor applied if designing the wall in accordance with strength design provisions.

Components that are not part of the lateral force resisting system must be detailed to accommodate expected building movements without restraint. For example, the connection of the top of an interior masonry partition wall to the floor above must allow the floor above to move freely in the plane of the wall. If this is not done, the wall will provide resistance to seismic loads and must be considered as part of the lateral force resisting system.

3.6.4.2 ANCHORAGE OF MASONRY WALLS

IBC Section 1604.8.2 requires that masonry walls be anchored to the structure to resist horizontal forces, F_p, or a minimum of 280 pounds per linear foot of wall, whichever is greater.

IBC Section 1604.8.2

1604.8.2 Concrete and masonry walls. Concrete and masonry walls shall be anchored to floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter but not less than a minimum strength design horizontal force of 280 plf (4.10 kN/m) of wall, substituted for "E" in the load combinations of Section 1605.2 or 1605.3. Walls shall be designed to

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resist bending between anchors where the anchor spacing exceeds 4 feet (1219 mm). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1609 for wind design requirements and see Section 1613 for earthquake design requirements.

3.6.5 ASCE 7 MASONRY SEISMIC REQUIREMENTS

IBC Section 1613.1 requires that seismic design and detailing follow the requirements of ASCE 7, excluding (ASCE 7) Chapter 14 and Appendix 11A. IBC contains material-specific seismic detailing requirements in Chapters 19 through 23. Masonry seismic detailing requirements are contained in IBC Section 2106, and are presented in Section 5.4.3 of this handbook. Any jurisdiction using ASCE 7 and not IBC will have ASCE 7, Chapter 14 and Appendix 11A to follow.

ASCE 7 Chapter 14, Section 14.4, gives special requirements for masonry in order to use the seismic requirements according to the analysis and determination of the earthquake forces. Section 14.4 requires that provisions contained in MSJC Code for material design and construction must be met in order to use the *R* factors, except as modified by Section 14.4. Intermediate and special reinforced masonry walls designed by MSJC Code Section 2.3 must also be coordinated with the requirements of ASCE 7 Section 14.4.

ASCE 7 Section 14.4.2

14.4.2 *R* factors. To qualify for the *R* factors set forth in this standard, the requirements of ACI 530/ASCE 5/TMS 402 and ACI 530.1/ASCE 6/TMS 602, as amended in subsequent sections, shall be satisfied.

Intermediate and special reinforced masonry shear walls designed in accordance with Section 2.3 of ACI 530/ASCE 5/TMS 402 shall also comply with the additional requirements contained in Section 14.4.6.

14.4.3 Classification of Shear Walls. Masonry walls, unless isolated from the seismic force-resisting system, shall be considered shear walls.

14.4.4 Anchorage Forces. The anchorage forces given in Section 1.14.3.3 of ACI 530/ASCE 5/TMS 402 shall not be interpreted to replace the anchorage forces set forth in this standard.

14.4.5 Modifications to Chapter 1 of ACI 530/ASCE 5/TMS 402.

14.4.5.1 Separation Joints. Add the following new Section 1.16.3 to ACI 530/ASCE 5/TMS 402:

1.16.3 Separation Joints. *Where concrete abuts structural masonry and the joint between the materials is not designed as a separation joint, the concrete shall be roughened so that the average height of aggregate exposure is 1/8 in. (3 mm) and shall be bonded to the masonry in accordance with these requirements as if it were masonry. Vertical joints not intended to act as separation joints shall be crossed by horizontal reinforcement as required by Section 1.9.4.2.*

14.4.5.2 Flanged Shear Walls. Replace Section 1.9.4.2.3 of ACI 530/ASCE 5/TMS 402 with the following:

1.9.4.2.3 *The width of flange considered effective in compression on each side of the web shall be the lesser of six times the flange thickness or the actual flange on either side of the web wall. The width of flange considered effective in tension on each side of the web shall be taken equal to 0.75 times the floor to floor wall height or the actual width of the flange on that side, whichever is less.*

14.4.6 Modifications to Chapter 2 of ACI 530/ASCE 5/TMS 402.

14.4.6.1 Stress Increase. If the increase in stress given in Section 2.1.2.3 of ACI 530/ASCE 5/TMS 402 is used, the restriction on load reduction in Section 2.4.1 of this standard shall be observed.

14.4.6.2 Reinforcement Requirements and Details.

14.4.6.2.1 Reinforcing Bar Size Limitations. Reinforcing bars used in masonry shall not be larger than No. 9 (M#29). The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimension of the cell, course, or collar joint in which it is placed. The area of reinforcing bars placed in a cell or in a course of hollow unit construction shall not exceed 4 percent of the cell area.

14.4.6.2.2 Splices. Lap splices shall not be used in plastic hinge zones of special reinforced masonry shear walls. The length of the plastic hinge zone shall be taken as at least 0.15 times the distance between the point of zero moment and the point of maximum moment. Reinforcement splices shall comply with ACI 530/ASCE 5/TMS 402 except paragraphs 2.1.10.7.2 and 2.1.10.7.3 shall be modified as follows:

2.1.10.7.2 Welded Splices: A welded splice shall be capable of developing in tension 125 percent of the specified yield strength, f_y, of the bar. Welded splices shall only be permitted for ASTM A706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls of masonry.

2.1.10.7.3 Mechanical Connections: Mechanical splices shall be classified as Type 1 or Type 2 according to Section 21.2.6.1 of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-wall joint of intermediate or special reinforced masonry shear wall system. Type 2 mechanical splices shall be permitted in any location within a member.

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14.4.6.2.3 Maximum Area of Flexural Tensile Reinforcement. Special reinforced masonry shear walls having a shear span ratio, M/Vd, equal to or greater than 1.0 and having an axial load, P, greater than $0.05 f'_m A_n$, which are subjected to in-plane forces, shall have a maximum reinforcement ratio, ρ_{max}, not greater than that computed as follows:

$$\rho_{max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'_m}\right)}$$

The maximum reinforcement ratio does not apply in the out-of-plane direction.

14.4.7 Modifications to Chapter 3 of ACI 530/ASCE 5/TMS 402.

14.4.7.1 Walls with Factored Axial Stress Greater Than $0.05 f'_m$. Add the following exception following the second paragraph of Section 3.3.5.4 of ACI 530/ASCE 5/TMS 402.

EXCEPTION: *A nominal thickness of 4 in. (102 mm) is permitted where load-bearing reinforced hollow clay unit masonry walls satisfy all of the following conditions.*

1. *The maximum unsupported height-to-thickness or length-to-thickness ratios do not exceed 27.*
2. *The net area unit strength exceeds 8,000 psi (55 MPa).*
3. *Units are laid in running bond.*
4. *Bar sizes do not exceed No. 4 (13 mm).*
5. *There are no more than two bars or one splice in a cell.*
6. *Joints are not raked.*

14.4.7.2 Splices in Reinforcement. Replace Sections 3.3.3.4(b) and 3.3.3.4(c) of ACI 530/ASCE 5/TMS 402 with the following:

(b) A welded splice shall be capable of developing in tension 125 percent of the specified yield strength, f_y, of the bar. Welded splices shall only be permitted for ASTM A706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls of masonry.

(c) Mechanical splices shall be classified as Type 1 or Type 2 according to Section 21.2.6.1 of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls. Type 2 mechanical splices are permitted in any location within a member.

Add the following new Section 3.3.3.4.1 to ACI 530/ASCE 5/TMS 402:

3.3.3.4.1 *Lap splices shall not be used in plastic hinge zones of special reinforced masonry shear walls. The length of the plastic hinge zone shall be taken as at least 0.15 times the distance between the point of zero moment and the point of maximum moment.*

14.4.7.3 Coupling Beams. Add the following new Section 3.3.4.2.6 to ACI 530/ASCE 5/TMS 402:

3.3.4.2.6 Coupling Beams. *Structural members that provide coupling between shear walls shall be designed to reach their moment or shear nominal strength before either shear wall reaches its moment or shear nominal strength. Analysis of coupled shear walls shall comply with accepted principles of mechanics.*

The design shear strength, ϕV_n, of the coupling beams shall satisfy the following criterion:

$$\phi V_n \geq \frac{1.25(M_1 + M_2)}{L_c} + 1.4V_g$$

where

M_1 *and* M_2 = *nominal moment strength at the ends of the beam*

L_c = *length of the beam between the shear walls*

V_g = *unfactored shear force due to gravity loads*

The calculation of the nominal flexural moment shall include the reinforcement in reinforced concrete roof and floor systems. The width of the reinforced concrete used for calculations of reinforcement shall be six times the floor or roof slab thickness.

14.4.7.4 Deep Flexural Members. Add the following new Section 3.3.4.2.7 to ACI 530/ASCE 5/TMS 402:

3.3.4.2.7 Deep Flexural Member Detailing. *Flexural members with overall-depth-to-clear-span ratio greater than 2/5 for continuous spans or 4/5 for simple spans shall be detailed in accordance with this section.*

3.3.4.2.7.1 *Minimum flexural tension reinforcement shall conform to Section 3.3.4.3.2.*

3.3.4.2.7.2 *Uniformly distributed horizontal and vertical reinforcement shall be provided throughout the length and depth of deep flexural members such that the reinforcement ratios in both directions are at least 0.001. Distributed flexural reinforcement is to be included in the determination of the actual reinforcement ratios.*

14.4.7.5 Shear Keys. Add the following new Section 3.3.6.11 to ACI 530/ASCE 5/TMS 402:

3.3.6.11 Shear Keys. *The surface of concrete upon which a special reinforced masonry shear wall is constructed shall have a minimum surface roughness of 1/8 in. (3 mm). Shear keys are required where the calculated tensile strain in vertical reinforcement from in-plane loads exceeds the yield strain under load combinations that include seismic forces based on an R factor equal to 1.5. Shear keys that satisfy the following*

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requirements shall be placed at the interface between the wall and the foundation.

1. *The width of the keys shall be at least equal to the width of the grout space.*
2. *The depth of the keys shall be at least 1.5 in. (38 mm).*
3. *The length of the key shall be at least 6 in. (152 mm).*
4. *The spacing between keys shall be at least equal to the length of the key.*
5. *The cumulative length of all keys at each end of the shear wall shall be at least 10 percent of the length of the shear wall (20 percent total).*
6. *At least 6 in. (150 mm) of a shear key shall be placed within 16 in. (406 mm) of each end of the wall.*
7. *Each key and the grout space above each key in the first course of masonry shall be grouted solid.*

14.4.7.6 Anchoring to Masonry. Add the following as the first paragraph in Section 3.1.6 to ACI 530/ASCE 5/TMS 402:

3.1.6 Headed and Bent-Bar Anchor Bolts. *Anchorage assemblies connecting masonry elements that are part of the seismic force-resisting system to diaphragms and chords shall be designed so that the strength of the anchor is governed by steel tensile or shear yielding. Alternatively, the anchorage assembly is permitted to be designed so that it is governed by masonry breakout or anchor pullout provided that the anchorage assembly is designed to resist not less than 2.5 times the factored forces transmitted by the assembly.*

14.4.7.7 Anchor Bolts. Replace the existing Section 3.1.4.4 of ACI 530/ASCE 5/TMS 402 with the following:

3.1.4.4 Anchor Bolts. *For cases where the nominal strength of an anchor bolt is controlled by masonry breakout or masonry pryout, φ shall be taken as 0.50. For cases where the nominal strength of an anchor bolt is controlled by anchor bolt steel, φ shall be taken as 0.90. For cases where the nominal strength of an anchor bolt is controlled by anchor pullout, φ shall be taken as 0.65.*

14.4.7.8 Nominal Shear Strength of Headed and Bent-Bar Anchor Bolts. Replace the existing Section 3.1.6.3 of ACI 530/ASCE 5/TMS 402 with the following:

3.1.6.3 Nominal Shear Strength of Headed and Bent-Bar Anchor Bolts. *The nominal shear strength, B_{vn}, shall be computed by Eq. (3-8) (strength governed by masonry breakout) and Eq. (3-9) (strength governed by steel), and shall not exceed 2.0 times that computed by Eq. (3-4) (strength governed by masonry pryout). In computing the capacity, the smallest of the design strengths shall be used.*

{No change to Eqs. 3-8 and 3-9.}

14.4.8 Modifications to Chapter 6 of ACI 530/ASCE 5/TMS 402.

14.4.8.1 Corrugated Sheet Metal Anchors. Add Section 6.2.2.10.2.3 to ACI 530/ASCE 5/TMS 402 as follows:

6.2.2.10.2.3 *Provide continuous single wire joint reinforcement of wire size W1.7 (MW11) at a maximum spacing of 18 in. (457 mm) on center vertically. Mechanically attach anchors to the joint reinforcement with clips or hooks. Corrugated sheet metal anchors shall not be used.*

14.4.9 Modifications to ACI 530.1/ASCE 6/TMS 602.

14.4.9.1 Construction Procedures. Add the following new Article 3.5 H to ACI 530.1/ASCE 6/TMS 602:

3.5 H. *Construction procedures or admixtures shall be used to facilitate placement and control shrinkage of grout.*

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3.7 QUESTIONS AND PROBLEMS

3-1 Define dead load and live load.

3-2 What are the design live loads for apartments, office buildings, schools and corridors?

3-3 A member supports 300 sq ft of a floor dead load which is 80 lbs per sq ft and a floor live load of 50 lbs per sq ft. What is the allowable live load reduction?

3-4 What are the five load combinations to be considered in the design of a structure?

3-5 What is the area considered for a concentrated load? What are the design concentrated loads for a library and a manufacturing plant?

3-6 What is the minimum roof live load for a flat roof in which the tributary area for the structural member is over 600 sq ft?

3-7 What is the uniform load for a roof that has a rise of 4 on 12 and an area of 425 sq ft?

3-8 A roof in Alaska has a pitch of 5 in. per ft and a potential snow live load of 100 lbs per sq ft What is the design snow load for the roof if the structure is sheltered and has an importance factor is 1.15?

3-9 Figure 3.3 shows the minimum basic wind speeds for various areas in the United States. Explain the significance of these wind speeds and describe the importance of the special wind speed regions. What is the standard height where wind velocities are measured? How does this affect the wind speed at ground level?

3-10 What is the wind load to be considered in the design of a masonry building 90 ft high located in Seattle.

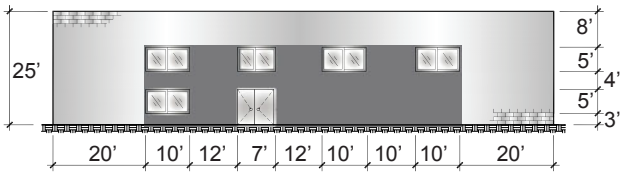
3-11 What are the factors to be considered in the design for wind pressure.

3-12 What are occupancy categories and the importance factors based upon these occupancy categories?

3-13 Describe wind exposure B, C, and D and explain their significance. What pressure coefficients are needed for each exposure and explain their use for primary frames and elements or components not in areas of discontinuity and chimneys.

3-14 What is the lateral load perpendicular to a 6 in. thick solid grouted interior masonry wall which is to be built in (a) Denver, Colorado, (b) San Francisco, California, and (c) Phoenix, Arizona?

3-15 Given a two-story building shown in the Figure below, determine the wind loads on the structure and on the pier elements A, B and C to be used in the lateral force calculations based upon 2006 IBC/ASCE 7-05 Wind Loading. Assume exposure B with a wind speed of 70 mph and an importance factor 1.0. What are the maximum pressures windward and leeward to be considered on the wall and on the roof?



3-16 What is the factor of safety for the stabilizing moment of the dead load against an overturning moment from wind pressure?

3-17 In the design of a structure for earthquake loading, what are three basic premises upon which the seismic provisions are based?

3-18 What is the basic equation for base shear given in the 2006 IBC/ASCE 7-05 and define the terms and tell how they are determined.

3-19 What is the significance of the fundamental period of vibration of a structure? What is the equation for this period? What is the whiplash effect and when must it be considered?

3-20 What is meant by modes of vibration? What is the first mode of vibration?

3-21 What is the effect of foundation soils on the period of a building? If a stiff building is founded on soft soil as opposed to base rock, what are the consequences?

3-22 What is the significance of the framing factor, R_w, and how do shear wall buildings compare to frame buildings? What is the effect of each on drift of the structure?

3-23 What is the period in each direction for a 10 story shear wall building 120 ft high and 60 ft wide?

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3-24 Why is the lateral seismic force on an element greater than the force on the building?

3-25 Give the equation for the seismic force on an element and explain each of the terms. Why is the lateral force coefficient, C_p, greater for a parapet than for a wall?

3-26 What is the minimum anchorage force that a wall must be designed for when connecting it to a floor or a roof diaphragm?

3-27 An 8 ft high cantilevered wall retains a back fill with a slope of 2 to 1. What is the lateral force and overturning moment on the wall?

3-28 A 6 ft high cantilever wall retains a level backfill of type 3 soil and has a surcharge from a parking lot of 200 lbs per sq ft. What is the lateral force on the wall?

3-29 What is the minimum factor of safety to be considered for a retaining wall for sliding and overturning?

3-30 What are the allowable foundation and the lateral force resistance pressures for a sandy gravel soil and for a clay and sand clay soil?

3-31 What are the lateral sliding coefficients for bed rock, sandy gravel and sandy silty gravel? What is the sliding resistance for sand clay soil?

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CHAPTER 4

DISTRIBUTION AND ANALYSIS FOR LATERAL FORCES

4.1 GENERAL

Buildings must resist not only vertical dead and live loads but also lateral forces from winds and earthquakes. Generally, these lateral forces are resisted by shear walls, perforated shear walls and/or moment resistant space frames. This Chapter will discuss shear walls and diaphragms, although there is a brief explanation of the concept of ductile masonry frames in Section 6.9.

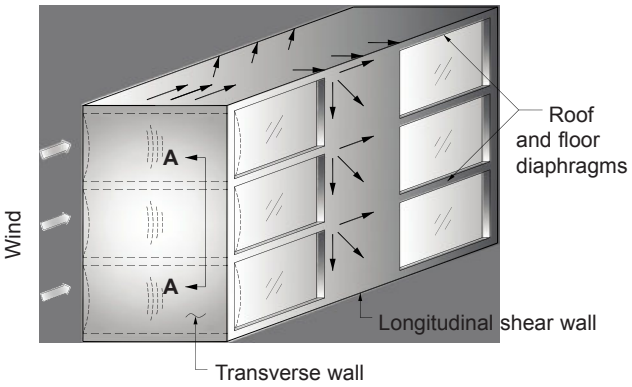

FIGURE 4.1 Lateral force distribution in a shear wall type building *(lateral load is transferred via roof and floor diaphragm action to the walls to create in-plane bending and shear on the "shear walls")*.

As shown in Figures 4.1 and 4.2, lateral forces from severe winds or earthquakes bend transverse walls between the floors. In box-type buildings, the lateral loads are transmitted from the transverse walls to the side shear walls by horizontal floor and roof diaphragms to cause in-plane bending or shear in the walls.

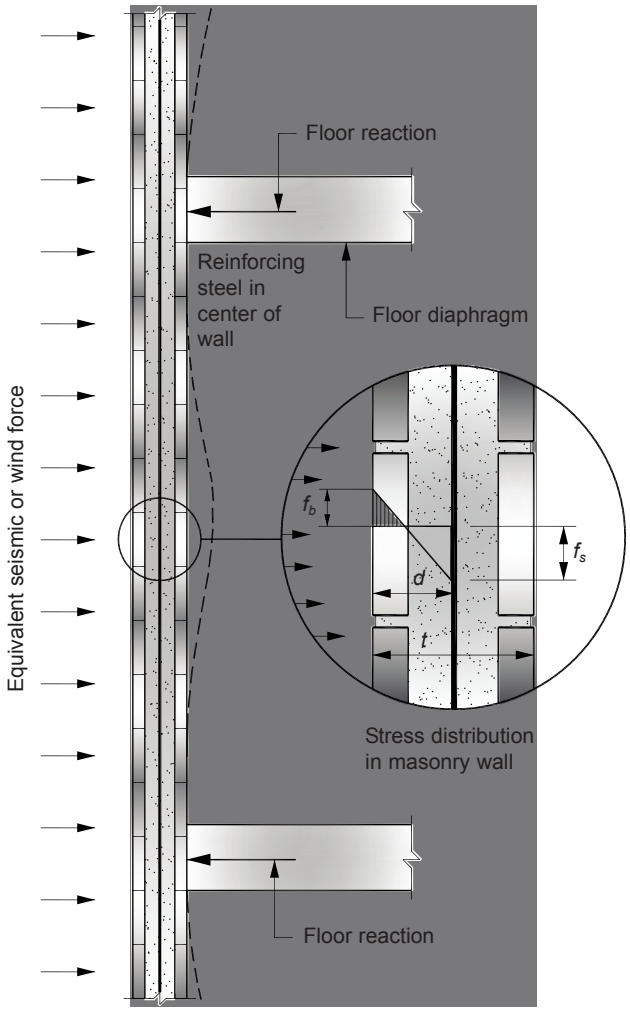

FIGURE 4.2 Load and stress distribution on wall *(out-of-plane bending due to direct lateral load on the wall)*.

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The shear wall in-plane rigidity or relative stiffness is significantly dependent upon the amount and area of openings, such as the windows, doors and other open areas as shown in Figure 4.3. Walls with openings may also be called perforated walls.

FIGURE 4.3 Masonry shear wall with openings.

4.2 HORIZONTAL DIAPHRAGMS

Diaphragms are often designed as horizontal beams where the roof or floor systems act as the webs and the bond beams or edge members act as the flange elements. Lateral forces imposed on the horizontal diaphragm cause it to deflect in beam action between the resisting shear walls and/or drag struts (Figure 4.4). As the diaphragm deflects, shear forces develop at the interface between the diaphragm and the chord members within the walls, placing the chords into tension or compression. In Figure 4.5, the shear forces are transferred from the wood structural panels into the ledger by nailing, and from the ledger into the masonry wall through anchor bolts embedded in the masonry wall. Shear walls must be capable of resisting shear and overturning forces while drag struts must carry both axial and flexural forces. Likewise, masonry bond beams, which act as chords for the diaphragm, must be adequately reinforced to resist the resulting tension and compression forces.

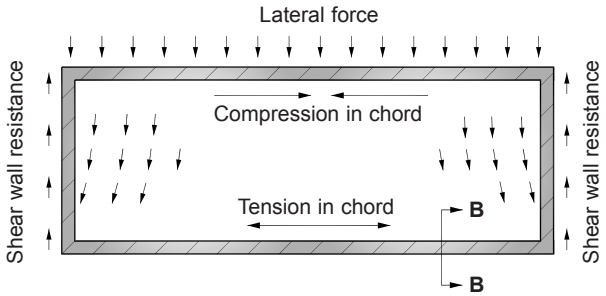

FIGURE 4.4 Beam action of diaphragm.

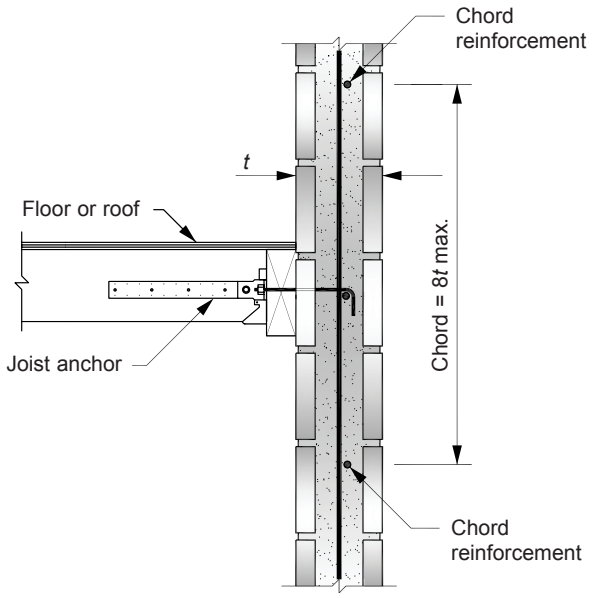

FIGURE 4.5 Diaphragm chord, Section B–B.

Diaphragms differ somewhat from beams in several special ways, as listed:

1. The span (of the diaphragm) is usually very short relative to depth; therefore, plane sections are not likely to remain plane, contrary to the usual assumption in the analysis of bending.

2. Web shear stresses and deflections due to shear are relatively more significant in diaphragms than stresses and deflections due to flexural action.

3. The diaphragm's components (flange, web, and connection devices) are often made of different materials. The "flanges" may be the walls normal to the direction of loading of the diaphragm, and the "flange" forces at the midspan of the diaphragm would be progressively diminished by the reduction in bending moment toward the diaphragm ends. The boundary members or chords are intended to resist these "flange" forces which are typically located near the plane of the diaphragm.

4. Relative and absolute deflections under prescribed lateral loading are often important design limitations.

Numerous types of diaphragm systems are used, most of which are reinforced concrete, metal or wood. Diaphragms may be flat, inclined or curved and may have openings, although large openings should be avoided.

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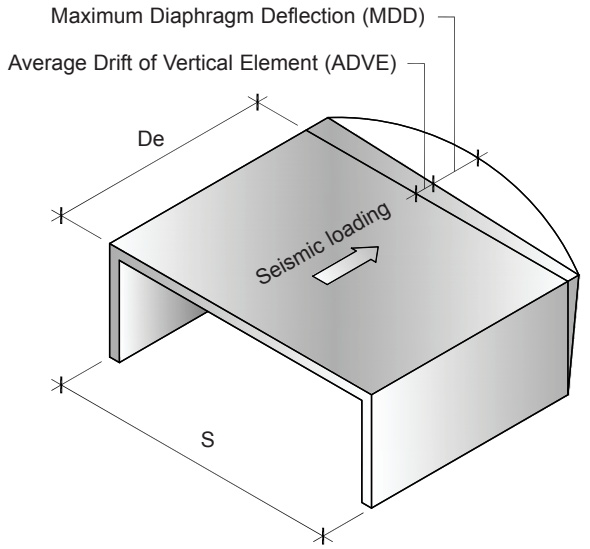
The IBC defines several types and conditions for diaphragms. The sections of the IBC and the corresponding types of diaphragms affecting masonry are shown below:

- **Diaphragm, general coverage – Sections 1602.1 and 2102.1.** A diaphragm is "a horizontal or sloped system acting to transmit lateral forces to the vertical-resisting elements. When the term "diaphragm" is used, it shall include horizontal bracing systems". For masonry structures, these diaphragms are generally either roofs or floors. Therefore, in the masonry chapter of the IBC, Section 2102 defines a diaphragm (for masonry structures) as "a roof or floor system designed to transmit lateral forces to shear walls or other lateral-load-resisting elements."
- **Diaphragm, chord – Section 1602.1.** A diaphragm chord is "a boundary element perpendicular to the applied load that is assumed to take axial stresses due to the diaphragm moment".
- **Diaphragm, flexible – Section 1602.1.** "A diaphragm is flexible for the purpose of distribution of story shear and torsional moment where indicated in Section 12.3.1 of ASCE 7, as modified in Section 1613.6.1 of the IBC. The determination for a flexible diaphragm is illustrated in Figure 4.6. Note that the deflection of the diaphragm is more than twice the deflection or story drift for the adjoining vertical resisting element (shear wall) for one story.



Note: Diaphragm is flexible if MDD > 2 (ADVE).

FIGURE 4.6 Flexible diaphragm (ASCE 7, Figure 12.3-1).

- **Diaphragm, rigid – Section 1602.1.** "A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift".
- **Diaphragm, semirigid – ASCE 7, Section 12.3.1.1.** "Semirigid diaphragms require an analysis that explicitly considers diaphragm flexibility. With the ASCE 7, most diaphragms are semirigid".
- **Diaphragm, collector – ASCE 7, Section 12.10.2.** A collector is "a horizontal diaphragm element parallel and in line with the applied force that collects and transfers diaphragm shear forces to the vertical elements of the lateral-force-resisting system and/or distributes forces within the diaphragm".
- **Diaphragm, drag strut** – see "Diaphragm collector".

A "simple diaphragm building" is a building in which wind loads are transmitted through floor and roof diaphragms to the vertical lateral-force-resisting systems. "Flexible buildings" are slender buildings and other structures that have a fundamental natural frequency less than 1 Hz.

Boundary members are strengthened portions along shear wall and diaphragm edges and are also called "boundary elements". Boundary elements include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities and reentrant corners. The 2005 MSJC Code includes requirements for special boundary elements attached to masonry walls and are reinforced "jamb" elements that may be thickened.

4.2.1 DIAPHRAGM ANCHORAGE REQUIREMENTS

Damage resulting from the 1971 San Fernando earthquake indicated that connections between walls and diaphragms were often inadequate. Accordingly, the UBC was revised to provide more stringent connection requirements which are now contained in IBC Sections 1604.8.2, 1609, 1613, and 2109.2.1.2. For example, IBC Section 1604.8.2 states:

IBC Section 1604.8.2

1604.8.2 Concrete and masonry walls. Concrete and masonry walls shall be anchored to floors, roofs and other structural elements that provide lateral

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support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter but not less than a minimum strength design horizontal force of 280 plf (4.10 kN/m) of wall, substituted for "*E*" in the load combinations of Section 1605.2 or 1605.3. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet (1219 mm). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1609 for wind design requirements and see Section 1613 for earthquake design requirements.

The response of building elements to severe earthquake ground motion on elements of a larger mass has caused some concern for masonry and concrete walls potentially pulling away from their support of roofs and floors. Therefore, the above IBC sections prescribe a minimum strength design for anchorages between masonry and concrete walls and horizontal diaphragms that are intended to provide lateral support.

The following is a brief list of some major IBC and MSJC Code anchorage and sub-diaphragm requirements:

1. Masonry walls must be positively anchored to all diaphragms with reinforcing steel, anchor bolts or joist anchors. Connections relying on shear friction are not permitted. Several items of minimum connections are required, (IBC Sections 1604.8.2, 1613.5.6.1, 1613.6.1 and 2109.2.1.2).

2. Connections must be capable of resisting the larger of the forces determined by IBC Section 1604.8.2, or 200 pounds per linear foot of horizontal force in any direction. MSJC Code Section 1.14.2.2.2.2 states "connectors shall be designed to transfer horizontal design forces acting either perpendicular or parallel to the wall, but not less than 200 lb per lineal foot of wall".

3. Anchors are spaced no more than 4 ft on centers unless the wall is designed to resist bending between the anchors (MSJC Code Section 1.14.2.2.2.2).

4. Anchors must be embedded in a structural, reinforced grouted element such as a bond beam (IBC Section 1604.8.2).

5. Diaphragms which support masonry walls must have continuous ties or struts between diaphragm chords to properly distribute anchorage forces. Sub-diaphragms may be used to transmit the anchorage forces into the main diaphragm (IBC Section 1604.8.2).

EXAMPLE 4-A Lateral Load on Diaphragm.

A 40 ft by 100 ft building is subjected to lateral load (determined by the various force criteria of wind and seismic load conditions) of 700 pounds per linear foot at the roof line. What is the stress in the chord?

Determine the tension or compression in the chord, reinforcement required at the roof line bond beam and anchor bolt requirements.

Solution 4-A

Calculate the moment and chord forces

$$M = \frac{wl^2}{8} = \frac{700 \times 100^2}{8} = 875{,}000 \text{ ft lbs}$$

$$\text{Tension or compression in chord} = \frac{M}{d} = \frac{875{,}000}{40}$$

$$= 21{,}875 \text{ lbs}$$

The "*d*" distance is smaller due to the half wall thickness or collector thickness on each wall; however, this correction is small and often neglected to save design time. In this case, for an 8 in. wall, $d = 40 - 8/12 = 39.33$ and the chord forces changes a small amount from 21,875 to 22,245 lbs, only a 1.6% change, and in this example problem does not significantly change the outcome.

The steel required in a wall bond beam at the roof line may be determined as follows: (assuming a one-third stress increase).

$$A_s = \frac{T}{F_s} \quad \text{where } F_s = 1.33 \times 24{,}000 \text{ psi}$$

$$= 32{,}000 \text{ psi}$$

$$A_s = \frac{21{,}875}{32{,}000} = 0.68 \text{ sq in.}$$

Conservatively use two #6 bars (A_s = 0.88 sq in.)

Shear between the ledger and bond beam flange elements.

$$= \frac{21{,}875}{\frac{1}{2} \times 100}$$

= 438 lbs/ft

Use 5/8 in. anchor bolts, from Table ASD-8a

Allowable shear in masonry = 1,330 lbs

Anchor design criteria is provided in MSJC Code Section 2.1.4 and the material criteria is given in the MSJC Specification Article 2.4 D. Edge distances and other anchor bolt conditions may change these allowable values.

$$\text{Spacing of bolts on long wall} = \frac{1330 \times 1.33}{438} \times 12$$

= 48 in. o.c.

$$\text{Shear to end walls (shear walls)} = \frac{700 \times 50}{40}$$

= 875 plf

$$\text{Spacing of bolts on short wall} = \frac{1330 \times 1.33 \times 12}{875}$$

= 24 in. o.c.

4.2.2 DEFLECTION OF DIAPHRAGMS AND WALLS

Lateral loads on buildings due to wind or earthquake will cause the diaphragm to deflect, which will impose out-of-plane deformations on walls that are perpendicular to the applied loads. Since masonry walls are relatively flexible perpendicular to the plane of the wall, they can tolerate a significant amount of bending and translation without impairing the shear resisting capacity parallel to the wall. Numerous horizontal mortar joints can crack and open up to provide an articulated wall which allows significant deflections up to 0.007*h*. The Slender Wall Research Project (1980-1982) conducted by an ACI - SEAOSC Task Committee demonstrated this effectively. Overstressing the masonry is not critical as there is a significant safety factor included.

Deflection of the diaphragm can be calculated by assuming that walls are flange elements which resist bending and deflection. These flange elements can be considered as half the distance between floors or parapet plus half the height of the wall from the floor to the ledger member. The flange height may also be conservatively assumed as 6 times the wall thickness. This "6*t*" criterion is used for the flange determination for intersecting walls in MSJC Code Section 1.9.4.2.3. Thus, "the width of the flange considered effective on each side shall be the lesser of six times the flange thickness or the actual flange on either side of the wall".

EXAMPLE 4-B Diaphragm Deflections.

Assume that the diaphragm in Figure 4.7 is 100 ft long by 40 ft wide, the parapet is 3 ft high and the wall is 14 ft from the floor to the ledger. The grouted clay masonry wall is 9 in. thick and the lateral load is 500 lbs/ft. Calculate the diaphragm deflection.

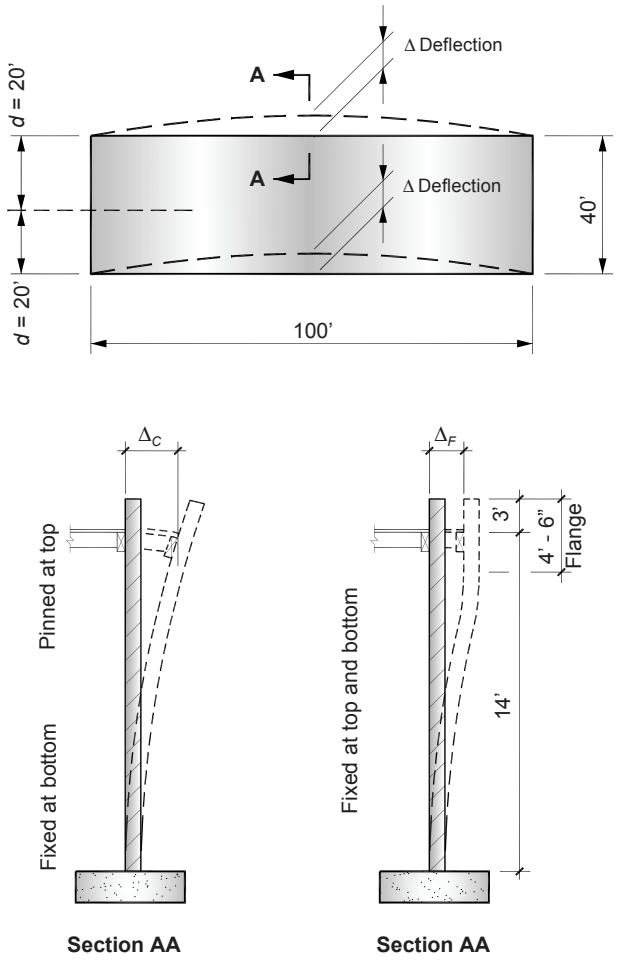


FIGURE 4.7 Deflection of diaphragm and walls.

Solution 4-B

Use f'_m = 1500 psi

$E_m = 700\ f'_m$ = 1,050,000 psi

$d = w/2 = 40/2 = 20$ ft

Effective width of diaphragm flange

$6t = 6 \times 9 = 54$ in.

Area of flange = 9 x 54 = 486 sq in.

$I = 2Ad^2 = 2 \times 486 \times (20 \times 12)^2$

$= 56 \times 10^6$ in.^{4}

A composite moment of inertia could be computed with grout and clay values, but simplicity and conservativeness is used in this example.

For a simply supported beam subjected to a uniform load:

$$\frac{5wl^4}{384EI} = \frac{5 \times 500 \times 100^4 \times 1728}{384 \times 1.050 \times 10^6 \times 56 \times 10^6}$$

$$\Delta = 0.019 \text{ in.}$$

The moment of inertia is based only on the chords (walls) and does not consider shear deformation or the type of diaphragm. Section 4.2.3 describes various types of diaphragms that influence the deflection.

The deflection of walls is prescribed by MSJC Code Section 1.14.3.2 as a service drift limitation and is stated as $0.007h$.

For Example 4-B the deflection limitation of the wall is

$$\Delta = 0.007(14)12$$

$= 1.176$ in.

This allowable deflection is significantly more than the diaphragm deflection of 0.019 in. and this is a satisfactory design.

4.2.3 TYPES OF DIAPHRAGMS

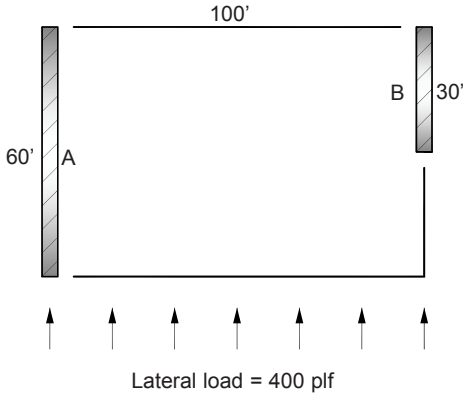
Diaphragms may be constructed of concrete, metal, wood or other suitable materials. They may be flat, inclined, curved, warped or folded and may have openings. The Tri-Services Technical Manual, *Seismic Design for Buildings* classifies diaphragms in five categories; very flexible, flexible, semi-flexible, semi-rigid and rigid, and is based on an F factor. The F factor is equal to the average deflection, in micro inches, of the diaphragm web per foot of span when stressed with a shear of one pound per foot. Generally, diaphragms are classified as either flexible or rigid depending on the diaphragm deflection relative to the deflections of the resisting vertical walls.

4.2.3.1 FLEXIBLE DIAPHRAGMS

Since wood and plywood sheathing floors and roofs are relatively flexible in comparison to the much stiffer masonry walls, they are considered as flexible diaphragms. Because of this flexibility, they are assumed to load the shear walls in proportion to the tributary area supported by each wall. They are also considered incapable of transmitting rotational or torsional forces.

EXAMPLE 4-C Shear Force to Walls.

Find the shear force on Walls A and B assuming, the roof is a flexible diaphragm.



Lateral load to wall A = 400 x 100/2 = 20,000 lbs

Lateral load per foot to wall A = 20,000/60 = 333 plf

Lateral load to wall B = 400 x 100/2 = 20,000 lbs

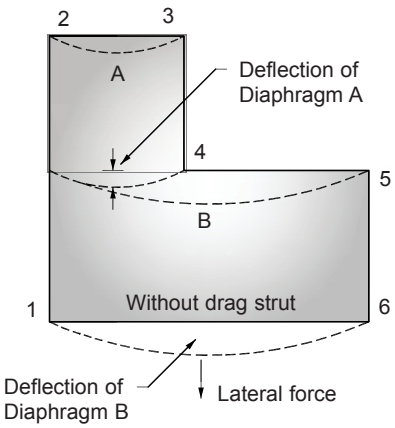
Lateral load per foot to wall B = 20,000/30 = 667 plf

As a point of reference only, Table 4.1 shows IBC and MSJC Code empirical limitations for the diaphragm length-to-width ratios.

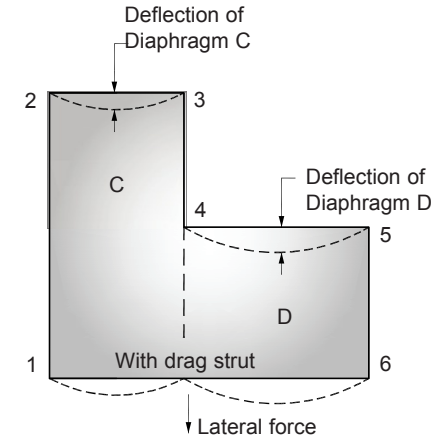
TABLE 4.1 Maximum Length-To-Width Ratios (IBC Table 2109.2.1.2 or MSJC Code Table 5.3.1)

Floor or Roof Diaphragm Construction	Maximum Length-to-Width Ratio of Diaphragm Panel
Cast-in-place concrete	5:1
Precast concrete	4:1
Metal deck with concrete fill	3:1
Metal deck with no fill	2:1
Wood	2:1

Flexible diaphragms that have plans in the shape of a T, L or Z can generate variable and incompatible deflections under lateral loads due to the discontinuities in the structure. Figure 4.8(a) illustrates that the deflection of Diaphragm A is not compatible with the deflection of Diaphragm B.



(a)



(b)

FIGURE 4.8 Relative deflection of diaphragm in building with irregular plan.

Thus, substantial tearing forces can develop along the boundary between Diaphragms A and B especially at Point 4.

In order to resist tearing forces and to resolve incompatible deflections, members, called drag struts, are used to subdivide irregular plans into a series of rectangular diaphragms such as C and D in Figure 4.8(b).

Lateral forces are transmitted from a diaphragm into a drag strut by shear while the drag strut transmits the load into shear walls by appropriate anchorage. Depending upon the direction of wind or earthquake forces, the drag strut may be in tension or compression and must be designed for either force. ASCE 7, Section 12.10.2.1 requires drag struts in higher design categories to consider the omega factor. This would include anchorage of the drag strut into the masonry wall.

EXAMPLE 4-D Determination of Lateral Shear Force to Walls Flexible Diaphragm.

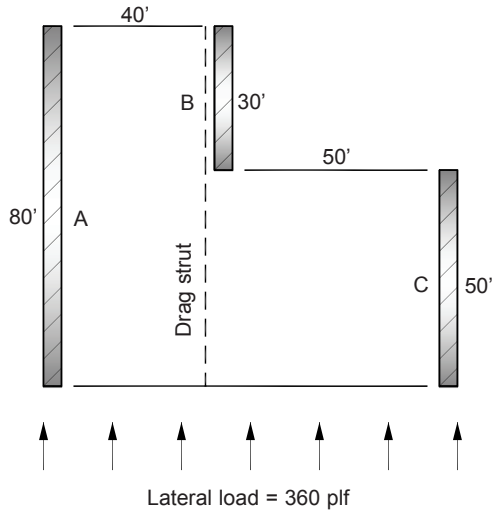
Calculate the shear force in the shear walls and the drag strut and determine the anchor bolt size and spacing requirements in wall B.

Lateral load to:

$$\text{Wall A} = 360 \times \frac{40}{2} = 7{,}200 \text{ lbs}$$

$$\text{Wall B} = 360 \times \frac{40 + 50}{2} = 16{,}200 \text{ lbs}$$

$$\text{Wall C} = 360 \times \frac{50}{2} = 9{,}000 \text{ lbs}$$



Lateral load per foot:

$$\text{Wall B and drag strut must resist} = \frac{16{,}200}{80}$$

$$= 202.5 \text{ plf}$$

Drag strut delivers 202.5 x 50 = 10,125 lbs to wall B

$$\text{Wall B must resist} = \frac{16{,}200}{30} = 540 \text{ plf}$$

Use 5/8 in. anchor bolts; Table ASD-8a

Allowable Shear = 1330 lbs

$$\text{Spacing of anchor bolts in wall B} = \frac{1330 \times 1.33 \times 12}{540}$$

$$= 39 \text{ in. o.c. max.}$$

As shown in Example 4-D, flexible diaphragms with irregular plans such as L, T, Z, etc., are designed so that each rectangular element will transmit shear forces to their respective resisting elements. The amount of force transferred to shear resisting elements is in proportion to the tributary areas they support since flexible diaphragms are considered incapable of distributing forces in relation to the rigidity of the shear walls.

Figures 4.9 and 4.10 show plans of irregular buildings along with tributary areas supported by each resisting element.

Force in the N-S direction, Figure 4.9(*a*).

Tributary Load Areas

Shear Wall 1-2. The tributary load area is *a*

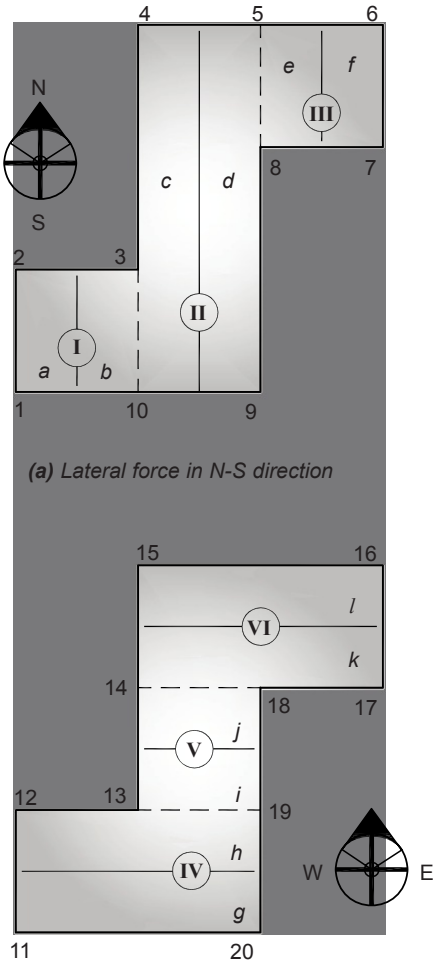
Shear Wall 3-4. The tributary load areas are *b* and *c*

Shear Wall 8-9. The tributary load areas are *d* and *e*

Shear Wall 6-7. The tributary load area is *f*

Diaphragm I is resisted by Shear Wall 1-2 and Drag Strut 3-10 which transmits the force to Shear Wall 3-4.

Diaphragm II is resisted by Shear Wall 3-4 and Drag Strut 3-10 which transmits the force to Wall 3-4 on the west side and on the east side by Shear Wall 8-9 and Drag Strut 5-8 which transmits the force to Wall 8-9.



***(a)** Lateral force in N-S direction*

***(b)** Lateral force in E-W direction*

FIGURE 4.9 Tributary load areas to lateral force resisting shear wall in Z plan building.

Diaphragm III is resisted by Shear Wall 6-7 and Drag Strut 5-8 which transmits the force to Wall 8-9.

Force in the E-W direction Figure 4.9(*b*).

Tributary Load Areas

Shear Wall 11-20. The tributary load area is *g*

Shear Wall 12-13. The tributary load areas are *h* and *i*

Shear Wall 17-18. The tributary load areas are *j* and *k*

Shear Wall 15-16. The tributary load area is *l*

Diaphragm IV is resisted by Shear Walls 11-20, 12-13 and Drag Strut 13-19 which transmits the force to Shear Wall 12-13.

Diaphragm V is resisted by Drag Strut 14-18 which transmits the force to Shear Walls 18-17 and by Drag Strut 13-19 which transmits the force to Shear Wall 12-13.

Diaphragm VI is resisted by Shear Walls 15-16, 17-18 and Drag Strut 14-18 which transmits the force to Shear Wall 18-17.

Force in the N-S direction, Figure 4.10(a).

Tributary Load Areas

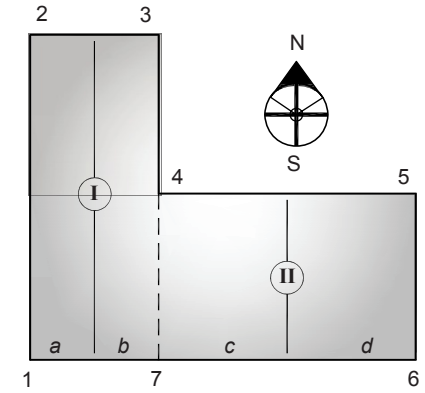
Shear Wall 1-2. The tributary load area is *a*

Shear Wall 3-4. The tributary load areas are *b* and *c*

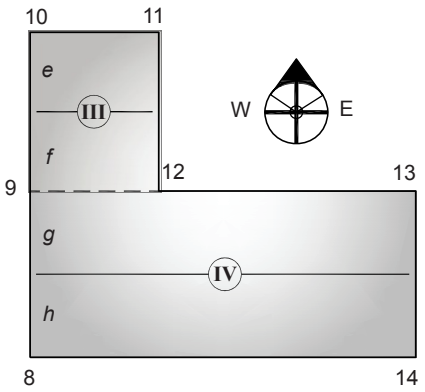
Shear Wall 5-6. The tributary load area is *d*

Diaphragm I is resisted by Shear Walls 1-2 and 3-4 and Drag Strut 4-7 which transmits the force to Shear Wall 3-4.

Diaphragm II is resisted by Shear Wall 5-6 and Drag Strut 4-7 which transmits the force to Shear Wall 3-4.



***(a)** Lateral force in N-S direction*



***(b)** Lateral force in E-W direction*

FIGURE 4.10 Tributary load areas to lateral force resisting shear walls in L plan building.

Force in the E-W direction Figure 4.10(b).

Tributary Load Areas

Shear Wall 10-11. The tributary load area is *e*

Shear Wall 12-13. The tributary load areas are *f* and *g*

Shear Wall 8-14. The tributary load area is *h*

Diaphragm III is resisted by Shear Wall 10-11 and Drag Strut 9-12 which transmits the force to Shear Wall 12-13.

Diaphragm IV is resisted by Shear Wall 8-14 and Drag Strut 9-12 which transmits the force to Shear Wall 12-13.

4.2.3.2 RIGID DIAPHRAGMS

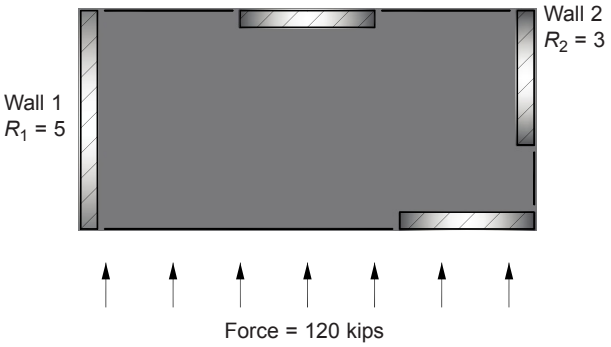
Floors or roofs constructed of concrete and poured gypsum on steel decking are generally considered as rigid diaphragms which can transmit both shear and rotational forces into shear walls.

Rigid diaphragms are assumed to load shear wall resisting elements in proportion to the walls' relative rigidities. Thus, even if a rigid diaphragm is loaded uniformly along its edge, the diaphragm is assumed to distribute the load to shear walls in proportion to wall rigidity or stiffness. The more rigid and stiff walls will proportionately receive more force from the diaphragm.

Modeling between the floor diaphragm rigidity and stiffness versus that of the walls provides a range of variability of proportions of forces between walls and floors. Usually, a steel deck roof is considered to be a flexible diaphragm; whereas, a reinforced concrete floor or roof is considered to be a rigid diaphragm. However, a light concrete topping on a metal deck can provide a flexible behavior when compared to a solid-grouted masonry wall of significant thickness. Relative span lengths also can modify the proportional rigidity behavior.

EXAMPLE 4-E Rigid Diaphragm, Distribution of Lateral Force to Shear Walls.

A lateral wind or seismic load of 120 kips is imposed on a building with a rigid diaphragm roof. If the end shear walls have relative rigidities of 3 and 5, how much lateral force does each wall resist? Ignore torsional effect. Distribute direct lateral force only.



Solution 4-E

Total resistance = $\Sigma R = R_1 + R_2 = 5 + 3 = 8$

Force to Wall 1

$$\text{Force } x \frac{R_1}{\Sigma R} = 120 \text{ x } \frac{5}{8} = 75 \text{ kips}$$

Force to Wall 2

$$\text{Force } x \frac{R_2}{\Sigma R} = 120 \text{ x } \frac{3}{8} = 45 \text{ kips}$$

Sum of Forces = $\overline{120 \text{ kips}}$

4.3 WALL RIGIDITIES

The rigidity of a wall element is dependent on its dimensions, the modulus of elasticity, E_m, the modulus of rigidity or shear modulus, E_v or, G, and the conditions of support at the top and the bottom of the wall.

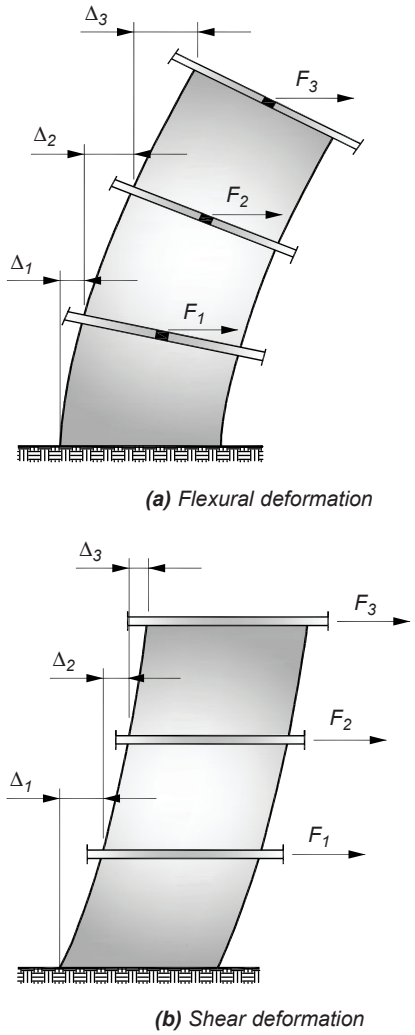
A wall fixed securely to the foundation with the top free to translate and rotate, is considered a cantilever wall. This is similar to a cantilever beam which deflects and rotates at the ends.

A pier or wall fixed at the top as well as the bottom, is considered a fixed or restrained wall. This is similar to a beam fixed at both ends.

The rigidity of the wall is defined as the reciprocal of the total deflection which is made up of both flexural and shear deformations as shown in Figure 4.11.

4.3.1 CANTILEVER PIER OR WALL

For a pier or wall fixed at only the bottom cantilevering from the foundation, the deflection is:



***(a)** Flexural deformation*

***(b)** Shear deformation*

FIGURE 4.11 Shear wall deformation.

$$\Delta_c = \Delta_m + \Delta_v = \frac{Ph^3}{3E_m I} + \frac{1.2Ph}{AE_v}$$

Where

Δ_m = deflection due to flexural bending, inches

Δ_v = deflection due to shear, inches

P = lateral force on pier, lbs

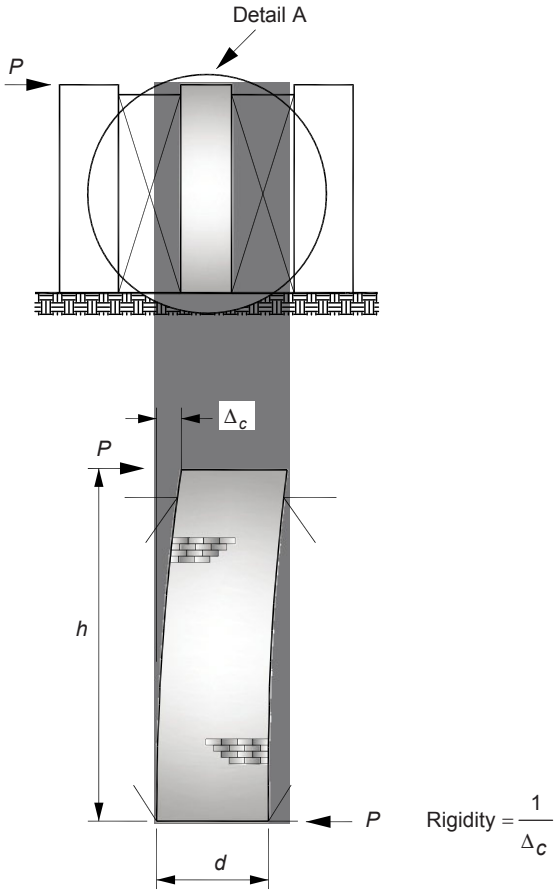
h = height of pier, inches

A = cross-sectional area of pier, sq in.

I = cross-sectional movement of inertia of pier in direction of bending, (inches^4). $I = td^3/12$.

E_m = modulus of elasticity in compression, psi

E_v = G = modulus of elasticity in shear, psi



Detail A

FIGURE 4.12 Wall pier displaced at top and cantilevering from fixed bottom.

For masonry design, assume E_m and E_v are constant, E_v = 0.4 E_m, with the same strength material throughout the wall. If it is also assumed that E_m = 1,000,000 psi (as a conservative round number), the wall thickness t, is 1 in. and P = 100,000 lbs, the deflection equations become:

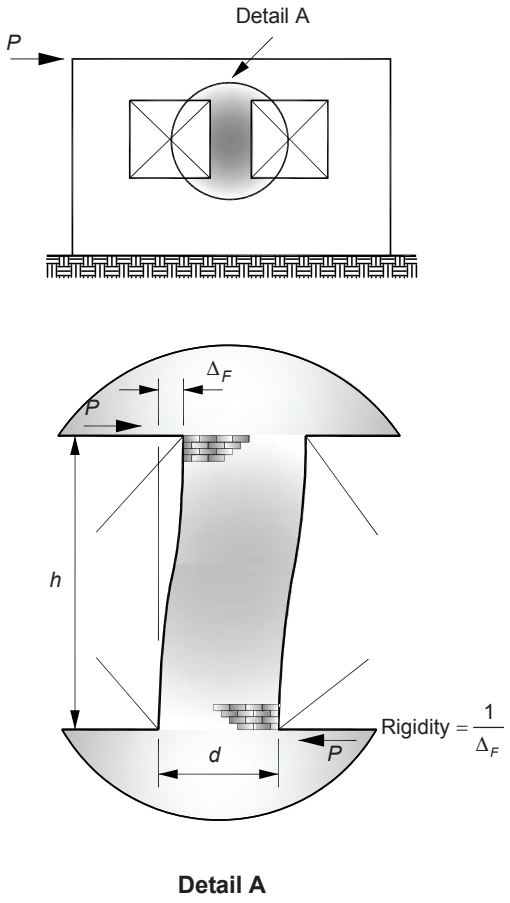
$$\Delta_c = \Delta \text{ cantilever} = 0.4\left(\frac{h}{d}\right)^3 + 0.3\frac{h}{d}$$

$$\text{Rigidity of Cantilever Pier } R_c = \frac{1}{\Delta \text{ cantilever}} = \frac{1}{\Delta_c}$$

4.3.2 FIXED PIER OR WALL

For a pier or wall fixed at the top and the bottom the deflection resulting from a force, P is:

$$\Delta_f = \Delta_m + \Delta_v = \frac{Ph^3}{12E_mI} + \frac{1.2Ph}{AE_v}$$



Detail A

FIGURE 4.13 Wall pier with top displaced and fixed top and bottom.

Assuming E_m = 1,000,000 psi and the wall thickness is constant, t = 1 in. and P = 100,000 lbs, the deflection equations become:

$$\Delta_f = \Delta \text{ fixed} = 0.1\left(\frac{h}{d}\right)^3 + 0.3\frac{h}{d}$$

$$\text{Rigidity of Fixed Pier } R_f = \frac{1}{\Delta \text{ fixed}} = \frac{1}{\Delta_f}$$

Tables ASD-89a through ASD-89g provide deflection coefficients and rigidities for both fixed and cantilever walls based on a wall thickness of 1 in., a lateral force = 100 kips, a modulus of elasticity of 1,000,000 psi and modulus of rigidity of 400,000 psi.

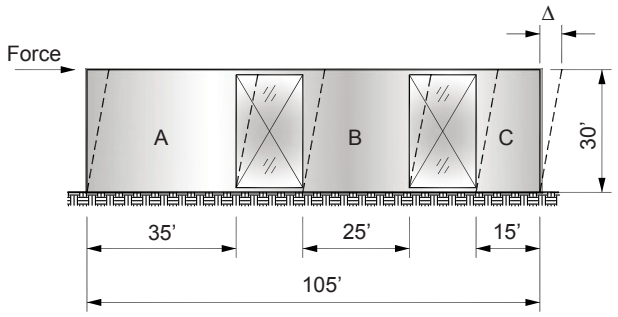
To determine the absolute deflection of a wall, factor the table values by the actual values of modulus of elasticity, shear modulus, thickness and lateral force. The effects of rotation could also be considered.

4.3.3 COMBINATIONS OF WALLS

Wall elements can be individual walls resisting lateral forces or portions of walls that are added to increase the resisting capacity of the wall system. Wall systems may be combined and the relative rigidity calculated. High rise walls may be considered as cantilevering from the foundation, with rigidity determined for each floor level based on the properties of the wall element below that floor level.

EXAMPLE 4-F Relative Rigidity, One Story.

What is the relative rigidity of a wall 105 ft long consisting of two openings, and three masonry walls cantilevering from the foundation? Assume the walls are connected to a rigid diaphragm and therefore deflect the same amount.



Solution 4-F

The resistance of each wall is additive to obtain the total resistance of the full length of the wall. Assume all walls are the same thickness and strength.

Wall	*h/l or h/d*	R_C*
A	0.86	1.952
B	1.20	0.951
C	2.00	0.263

*From Table ASD-89

$\Sigma = \overline{3.166}$

Rigidity of wall = $\Sigma R + R_A + R_B + R_C = 3.166$

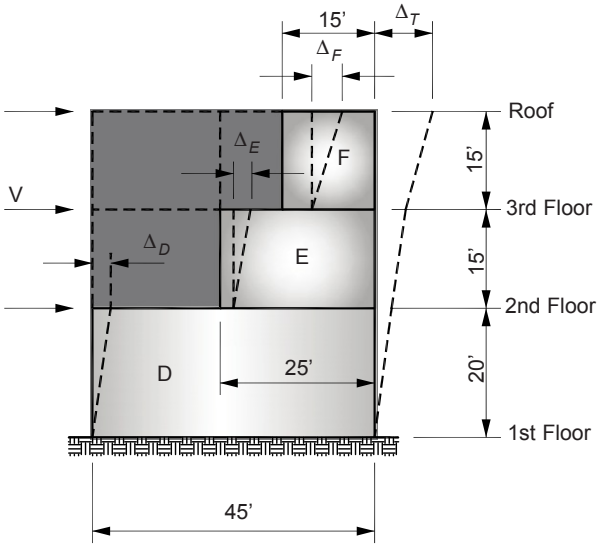
Deflection of wall $= \Delta = \dfrac{1}{\Sigma R} = \dfrac{1}{3.166} = 0.316$

If the wall is continuous in one element, 75 ft long, (35 ft + 25 ft + 15 ft) and all the glass is at one end, the *h/l* = 30/75 = 0.40, the rigidity would be 6.868 and the deflection would be equal to 0.146. This wall would thus be approximately twice as stiff as the above example.

EXAMPLE 4-G Relative Rigidity, Multi-Story.

What is the relative rigidity of the 45 ft long three story wall shown below? Walls D, E and F are connected and the deflection of each wall adds to the deflection of the walls above.

Assume all walls are the same thickness and strength. Also assume floor-to-floor cantilever action.



Solution 4-G

Deflection $\Delta_T = \Delta_D + \Delta_E + \Delta_F +$ rotational effects.

For simplicity, ignore rotational effects.

The deflection of walls D, E, and F are due to force *V*.

Wall	*h/l or h/d*	Δ_C *
D	0.44	0.166
E	0.60	0.266
F	1.00	0.700

*From Table ASD-89

$\Delta_T = \overline{1.132}$

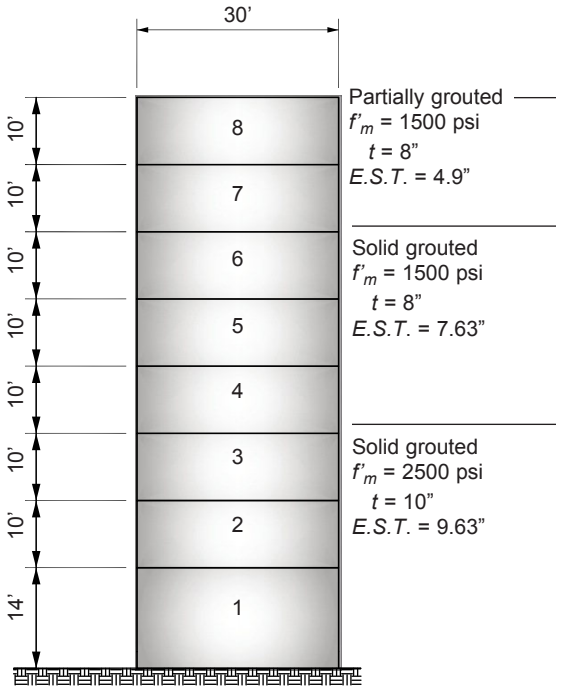
$R_{DEF} = \dfrac{1}{\Delta_T} = \dfrac{1}{1.132} = 0.883$

If the wall is solid 50 ft high and 45 ft long, the h/l = 50/45 = 1.11, the deflection, Δ = 0.88, and the rigidity R_C, = 1.136.

4.3.4 HIGH RISE WALLS

EXAMPLE 4-H Relative Rigidity, High Rise.

For the elevation shown in the following drawing what is the relative rigidity of the wall at each floor level? Wall strengths and equivalent solid thicknesses (E.S.T.) are given. Walls are concrete block masonry.



Solution 4-H

Table ASD-89 is based on t = 1 in. and E_m = 1,000,000 psi. Corrections to the cantilever deflection value, Δ_C can be made by multiplying the value given by

$$\frac{1{,}000{,}000}{900f'_m} \text{ or } \frac{1111}{f'_m} \text{ and } \frac{1}{t} \text{ for concrete masonry.}$$

Correction coefficient for

$$\text{Walls 7, 8} \quad = \Delta_C = \frac{1111}{1500} \times \frac{1}{4.9} = 0.1512\Delta_C$$

$$\text{Walls 4, 5, 6} \quad = \Delta_C = \frac{1111}{1500} \times \frac{1}{7.63} = 0.0971\Delta_C$$

$$\text{Walls 1, 2, 3} \quad = \Delta_C = \frac{1111}{2500} \times \frac{1}{9.63} = 0.0461\Delta_C$$

Rigidity of 8 Story Wall

Floor Level	H (ft)	l (ft)	$\frac{h}{l}$	Δ_C From T Tables	Correction Coefficient	Actual Δ_C	$\Sigma\Delta$	Rigidity $\frac{1}{\Sigma\Delta}$
8	10	30	.033	0.113	0.1512	0.0171	0.0860	11.62
7	10	30	.033	0.113	0.1512	0.0171	0.0689	14.51
6	10	30	.033	0.113	0.0971	0.0110	0.0518	19.31
5	10	30	.033	0.113	0.0971	0.0110	0.0408	24.51
4	10	30	.033	0.113	0.0971	0.0110	0.0298	33.56
3	10	30	.033	0.113	0.0461	0.0052	0.0188	53.19
2	10	30	.033	0.113	0.0461	0.0052	0.0136	73.53
1	14	30	.047	0.183	0.0461	0.0084	0.0084	119.05

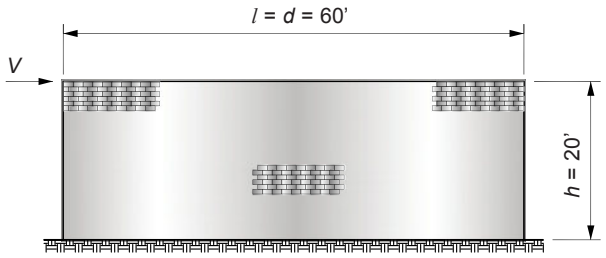
4.3.5 RELATIVE STIFFNESS OF WALLS

Walls with different configurations can have different stiffnesses or rigidities which, in turn, will change the period of the building, the response of the building and the amount of force resisted by each wall or configuration. For instance, walls with expansion joints will have much lower rigidities than solid walls of equal total length.

EXAMPLE 4-I Wall Rigidities

Use Table ASD-89 to compute the rigidity of the concrete block walls shown, assuming they are cantilevered from the base.

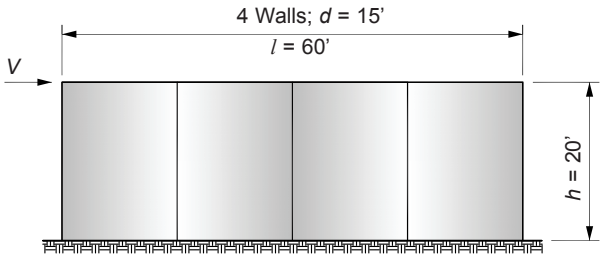
a) Solid wall



$$\frac{h}{d} = \frac{20}{60} = 0.33$$

R_C = 8.820 (Table ASD-89a)

b) Wall with vertical slots (no head joints)



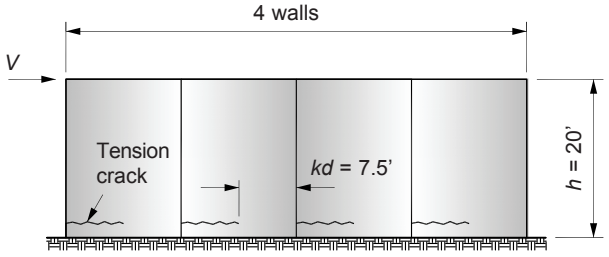
$$\frac{h}{d} = \frac{20}{15} = 1.33$$

R_C = 0.746 (Table ASD-89b)

$4R_C = 4 \times 0.746 = 2.984$

c) Wall with vertical slots and wall elements are assumed to be cracked; k = 0.50

compression length $kd = 0.50 \times 15 = 7.5'$

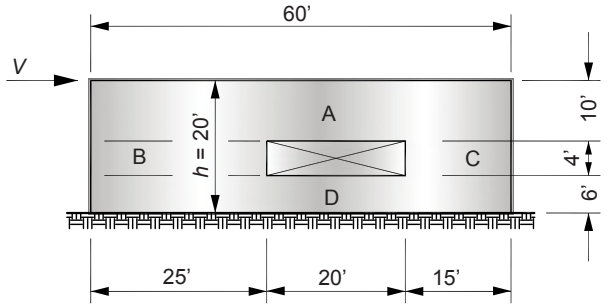


$$\frac{h}{kd} = \frac{20}{7.5} = 2.67$$

R_C = 0.119 (Table ASD-89c)

$4R_C = 4 \times 0.119 = 0.476$

d) Wall contains a window opening



1) Deduct from solid wall the effect of the opening

Solid Wall ABCD

$$\frac{h}{d} = \frac{20}{60} = 0.33$$

R_{solid} = 8.82 (Table ASD-89a) $\Delta = 0.113$

Deduct deflection of middle strip

$$\frac{h}{d} = \frac{4}{60} = 0.067 \qquad -\Delta = 0.020$$

$$\Delta = 0.093$$

2) Add deflection of fixed wall piers B + C

Pier B

$$\frac{h}{d} = \frac{4}{25} = 0.16 \quad R_B = 20.657$$

Pier C

$$\frac{h}{d} = \frac{4}{15} = 0.27 \quad R_C = 12.053$$

$$\Sigma(R_B + R_C) = 32.710$$

$$\frac{1}{\Sigma R_{BC}} = \frac{1}{32.710} = \Delta = 0.031$$

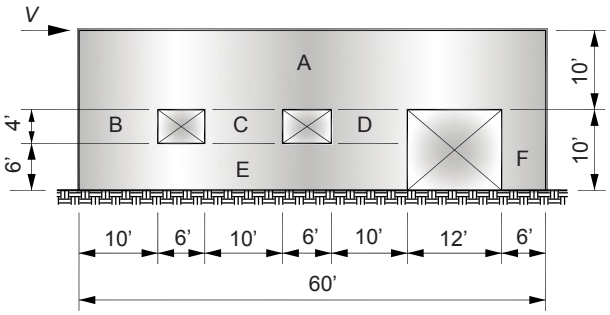
$$\Sigma\Delta = 0.124$$

$$R_{ABCD} = \frac{1}{\Sigma\Delta} = \frac{1}{0.124} = 8.06$$

e) Wall contains window and door openings

1) Solid wall ABCDEF

$$\frac{h}{d} = \frac{20}{60} = 0.33$$



R_{solid} = 8.820 $\Delta = 0.113$

2) Deduct bottom strip BCDEF

$$\frac{h}{d} = \frac{10}{60} = 0.17$$

From Tables ASD-89 for a fixed pier,

$$\frac{-\Delta = \ 0.051}{\Delta_A = 0.062}$$

3) Add back the fixed Piers B, C and D

$$\frac{h}{d} = \frac{4}{10} = 0.40$$

For one pier

R_B = 7.911

$\Sigma(R_B + R_C + R_D) = 3R_B$ = 23.27

Δ_{BCD} = 0.043

Add Pier E

$$\frac{h}{d} = \frac{6}{42} = 0.14; \qquad \frac{\Delta_E = 0.042}{\Sigma\Delta = 0.085}$$

4) $$R_{BCDE} = \frac{1}{\Sigma\Delta} = \frac{1}{0.085} = 11.76$$

5) Add pier F

$$\frac{h}{d} = \frac{10}{6} = 1.67; \ R_F = 1.034$$

$\Sigma R_F + R_{BCDE}$ = 1.034 + 11.76 = 12.80

$$\Delta_{BCDEF} = \frac{1}{R_{BCDEF}} = \frac{1}{12.80} = 0.078$$

$\Sigma\Delta = \Delta_A + \Delta_{BCDEF}$ = 0.062 + 0.078 = 0.140

6) $$R_{ABCDEF} = \frac{1}{\Sigma\Delta}$$

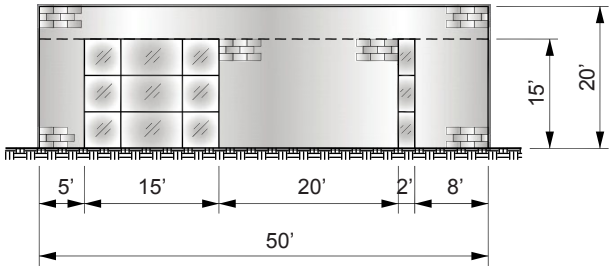
$$= \frac{1}{0.140} = 7.14$$

EXAMPLE 4-J Shear Stresses in Walls with a Rigid Diaphragm.

Calculate the shear stresses in the walls shown below, assuming a rigid diaphragm transmits a total seismic force of 135 kips (including any increases necessary for the rigid diaphragm due to ASCE 7 analysis) to 9 in. thick reinforced brick shear walls. These walls are designed assuming f'_m = 1500 psi. Elevations of the end walls are as shown below. Do not include torsional effects.

Use Tables and Diagrams ASD-5 and ASD-6 to find the allowable shear stress.

Solution 4-J

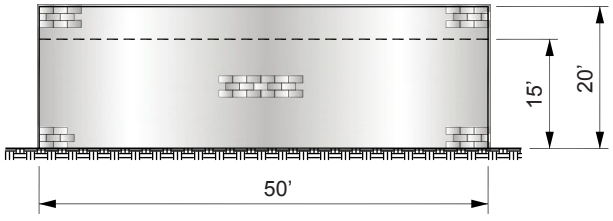


Wall 1

Relative rigidity of Wall 1

Pier	*h* (ft)	*d* (ft)	*h/d*	R_f	Table
A	15	5	3.00	0.278	ASD-89c
B	15	20	0.75	3.743	ASD-89a
C	15	8	1.88	0.814	ASD-89b

ΣR = 4.835



Wall 2

Relative rigidity of Wall 2 (Rigidity of total wall)

h (ft)	*d* (ft)	*h/d*	R_c (From Table ASD-89a)
15	50	0.30	9.921

$$\text{Wall 1} = \frac{4.835}{4.835 + 9.921}(135)$$

= 0.328 (135) = 44 kips

$$\text{Wall 2} = \frac{9.921}{4.835 + 9.921}(135)$$

= 0.672 (135) = 91 kips

Wall 1 resists 33% of the load and Wall 2 will resists 67% of the load.

Distribute the shear force into Wall 1

$$V_{Pier\ A} = \frac{0.278}{4.835}(44) = 2.5\,\text{kips}$$

$$f_v = \frac{V}{td} = \frac{2500}{9(60)} = 4.6\,\text{psi}$$

From Table ASD-5 for *h/d* = 3.0; the allowable shear stress is 35 psi. Increase by one-third for wind or seismic forces:

F_v = 35 (1.33) = 46 psi > 4.6 psi O.K.

$$V_{Pier\ B} = \frac{3.743}{4.837}(44) = 34.1\text{kips}$$

$$f_v = \frac{V}{td} = \frac{34{,}100}{9(240)} = 15.8\,\text{psi}$$

For *h/d* = 0.75; F_v = 42 (1.33)

= 56 psi > 15.8 psi O.K.

$$V_{Pier\ C} = \frac{1.88}{4.835}(44) = 7.4\,\text{kips}$$

$$f_v = \frac{V}{td} = \frac{7400}{9(96)} = 8.6\,\text{psi}$$

For *h/d* = 1.88; F_v = 35 (1.33)

= 46 psi > 8.6 psi O.K.

No shear reinforcement is required in any of the piers. Use minimum temperature steel; A_s = 0.0007 *bt* minimum.

= 0.0007(12)(9) = 0.0756 in.^{2}

(Use #4 bars @ 30 in.)

This constitutes the minimum prescriptive reinforcement for SDC (Seismic Design Category) D, E and F. For SDC C the requirement is only one #4 bar @ 48 inches. There are additional prescriptive requirements for top and bottom of walls and around doors and openings.

4.4 OVERTURNING

Lateral forces from winds and earthquakes can create severe overturning moments on buildings. If the overturning moment is large enough, it may overcome the dead weight of the structure and induce tension at the ends of shear walls. It will also cause high compression forces that may require an increase in the specified masonry strength, f'_m, an increase in the amount of compression steel in the wall, or an increase in the thickness or size of the shear wall.

In evaluating the stabilizing effect of the dead load to the overturning moment the designer has the option of including dead load gravity effects as a means of offsetting the horizontal earthquake load which in turn causes a vertical overturning load potentially offset by the downward overturning force of the one side. Thus, not all of the load is considered and therefore a percentage reduction is a slightly conservative and reasonable approach which allows the designer to reduce the uplift effect on the footing and reduce the amount of tension reinforcement that would be needed if the offsetting dead load were not considered.

- 90% of the dead load can be used to offset seismic loads for SD (IBC Section 1605.2.1)
- 60% can be used to offset 70% of the earthquake loads for ASD (IBC Section 1605.3.1)
- 90% can be used to offset E/1.4 (IBC Section 1605.3.2)
- 90% can be used to offset the maximum earthquake, E_m of vertical and horizontal effects (ASCE 7 Section 12.4.3)

The load distributions may account for seismic vertical ground acceleration similar to those experienced in the January 17, 1994 Northridge Earthquake where the vertical ground accelerations were the highest ever recorded.

The overturning moment (OTM) at the base of a structure may be determined by using the equation:

$$OTM = F_n h_n + \sum_{i=1}^{n} F_i h_i$$

As the equation states, the OTM equals the force at the top, F_n, times its height above the base, h_n, plus the sum of the forces at each level, F_i, times their heights above the base, h_i. This is for all floors, n, taken at each level, i = 1.

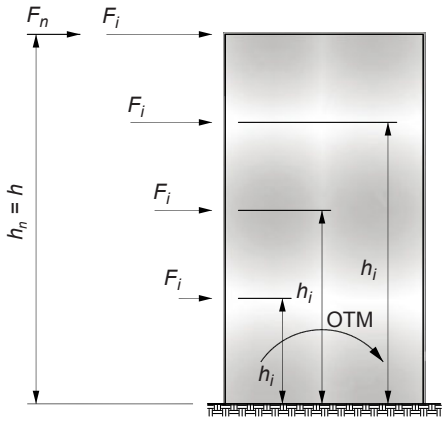


FIGURE 4.14 Overturning moment at base.

The overturning moment for each wall may also be determined at various floor levels to establish the amount of reinforcement required and the loads and stresses on the masonry.

$$OTM_x = F_n(h_n - h_x) + \sum_{i=1}^{n} F_i(h_i - h_x)$$

The overturning moment at level, x, above the base is equal to force at the top, F_n times the height from level x to the top (h_n - h_x), plus the sum of the forces at each level F_i times the height from level i to level x (h_i - h_x).

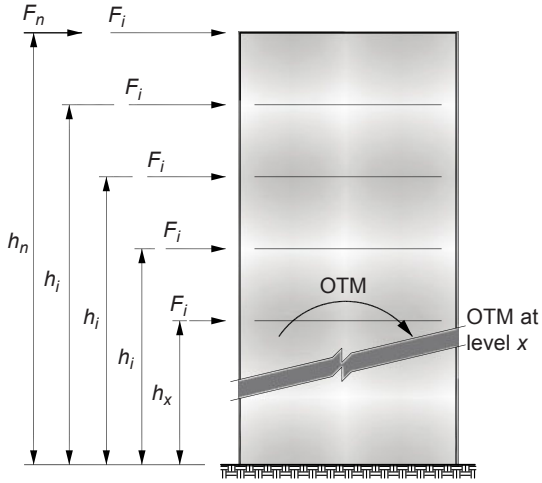
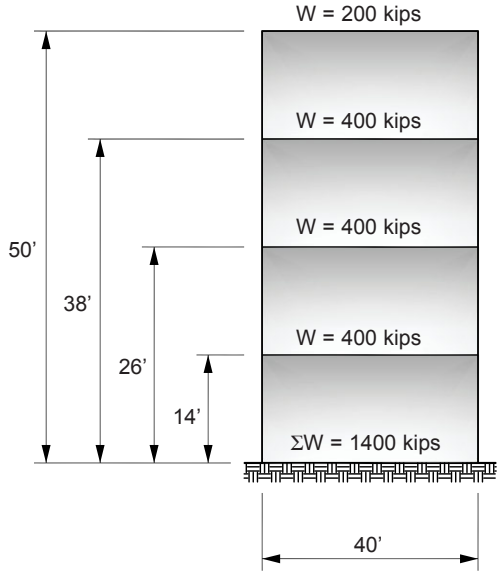


FIGURE 4.15 Overturning moment at any level.

EXAMPLE 4-K Base and Story Shear and Overturning Moment.

Determine the base shear, story shear and overturning moment for the masonry shear wall structure shown. The structure is located in Seismic Design Category D using S_{DS} = 0.4 sec., S_{D1} = 0.7 sec., and I = 1.0.



Solution 4-K

In Seismic Design Category D, only "Special Reinforced Shear Walls" are permitted (MSJC Code Section 1.14.6.4) thus the selected R factor from Table 12.2-1 of ASCE 7 is:

R = 5.50

The total weight of the building is

W = 200 + 3(400) = 1400 kips

Using the Equivalent Lateral Force Procedure (ASCE 7, Section 12.8), the base shear is

$V = C_s W$ (ASCE 7 Eq 12.8-1)

where C_s need not exceed the values below:

$$C_s = \frac{S_{DS}}{\frac{R}{I}} \le \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad \text{for } T \le T_L$$

(ASCE 7 Eq 12.8-2 and 12.8-3)

$$C_s = \frac{S_{DS}}{\frac{R}{I}} \le \frac{S_{D1}T_L}{T^2\left(\frac{R}{I}\right)} \quad \text{for } T > T_L$$

(ASCE 7 Eq 12.8-2 and 12.8-4)

However, C_s cannot be less than 0.01. When S_1 is greater or equal to 0.6g:

$$C_s = \frac{0.5S_1}{\frac{R}{I}} \qquad \text{(ASCE 7 Eq 12.8-6)}$$

The fundamental period, T, cannot exceed the combined product of the coefficient for upper limit on calculated period, C_u, from ASCE 7 Table 12.8-1 and the proximate fundamental, T_a, determined from ASCE 7 Equation 12.8-7. As an alternative to performing an analysis to determine the fundamental period, T, the use of the approximate building period, T_a, calculated in accordance with ASCE 7 Section 12.8.2.1, directly is permitted.

The approximate fundamental period (T_a), in *s*, can be determined from the following equation:

$$T_a = C_t h_n^x \qquad \text{(ASCE 7 Eq 12.8-7)}$$

where h_n is the height in ft above the base to the highest level of the structure.

$$T_a = C_t h_n^x \qquad \text{(ASCE 7 Eq 12.8-7)}$$

$x = 0.75$, $C_t = 0.020$ (ASCE 7 Table 12.8-2)

$T_a = 0.020\ (50)^{0.75} = 0.38$ sec.

$$C_s = \frac{0.4}{\frac{5.5}{1}} \le \frac{0.7}{0.38\left[\frac{5.5}{1}\right]}$$

$0.073 \le 0.335$

Thus, $C_s = 0.073$

$V = 0.073\ (1400) = 102$ kips

Distribution of Forces and Overturning Moments

$$F_x = C_{vx}V \qquad \text{(ASCE 7 Eq 12.8-11)}$$

where F_x is the lateral force at level x

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(ASCE 7 Eq 12.8-12)}$$

where h_i and h_x are the heights of the stories above the base at levels i and x, respectively, and

k = 1.0 for periods ≤ 0.5 sec.

k = 2.0 for periods ≥ 2.5 sec. Values of periods between 0.5 and 2.5 sec., may be determined by linear interpolation.

w_i and w_x are the respective portions of the total weight, w, assigned to levels i and x.

With a period in this case of 0.38 sec., k = 1.0

Level	w_i or w_x (kips)	h_i or h_x (ft)	$w_i h_i$	C_{vx}	$F_i = C_{vx}V$ (kips)	Lateral Force (kips)	Story Force (kips)	$F_i h_i$ (ft kips)
4	200	50	10,000	0.243	24.8	24.8	–	1240
3	400	38	15,200	0.369	37.7	37.7	24.8	1432
2	400	26	10,400	0.252	25.8	25.8	62.5	671
1	400	14	5,600	0.136	13.9	13.9	88.3	195
Base	1400	–	–	–	–	102	102	–
			Σ = 41,200		Σ = 102			Σ = 3538

Overturning moment; OTM

$M_B = 24.8(50) + 37.7(38) + 25.8(26) + 13.9(14)$

$= 3{,}538$ ft kips

4.5 DIAPHRAGMS, CHORDS, COLLECTORS, BUILDING IRREGULARITIES, AND WALL CONNECTIONS

ASCE 7 Section 12.10 contains special provisions for diaphragms, chords and collectors. ASCE 7 Section 12.10.1 states that general diaphragm design shall include both shear and flexural capacities. Openings, reentrant corners and other diaphragm discontinuities must be considered in design.

Section 12.10.1.1 of ASCE 7 contains provisions for the diaphragm design forces as follows:

ASCE 7 Section 12.10.1.1

12.10.1.1 Diaphragm Design Forces. Floor and roof diaphragms shall be designed to resist design seismic forces from the structural analysis, but shall not be less than that determined in accordance with Eq. 12.10-1 as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (12.10\text{-}1)$$

where

F_{px} = the diaphragm design force
F_i = the design force applied to Level i
w_i = the weight tributary to Level i
w_{px} = the weight tributary to the diaphragm at Level x

The force determined from Eq. 12.10-1 need not exceed $0.4S_{DS}\,Iw_{px}$, but shall not be less than $0.2S_{DS}\,Iw_{px}$. Where the diaphragm is required to transfer design seismic force from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offsets in the placement of the elements or to changes in relative lateral stiffness in the vertical elements, these forces shall be added to those determined from Eq. 12.10-1. The redundancy factor, ρ, applies to the design of diaphragms in structures assigned to Seismic Design Category D, E, or F. For inertial forces calculated in accordance with Eq. 12.10-1, the redundancy factor shall equal 1.0. For transfer forces, the redundancy factor, ρ, shall be the same as that used for the structure. For structures having horizontal or vertical structural irregularities of the types indicated in Section 12.3.3.4, the requirements of that section shall also apply.

Section 12.10.2 of ASCE 7 contains provisions for the collector elements of a diaphragm as follows:

ASCE 7 Section 12.10.2

12.10.2 Collector Elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces.

12.10.2.1 Collector Elements Requiring Load Combinations with Overstrength Factor for Seismic Design Categories C through F. In structures assigned to Seismic Design Category C, D, E, or F, collector elements (see Fig. 12.10-1), splices, and their connections to resisting elements shall resist the load combinations with overstrength of Section 12.4.3.2.

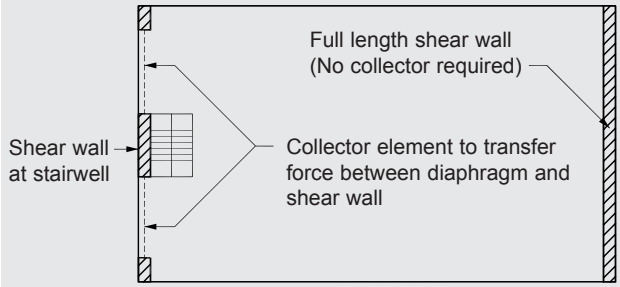


FIGURE 12.10-1 COLLECTORS

Note that ASCE 7 Section 12.11.2 provides for special connection requirements for the walls to the structure:

ASCE 7 Section 12.11.2

12.11.2 Anchorage of Concrete or Masonry Structural Walls. The anchorage of concrete or masonry structural walls to supporting construction shall provide a direct connection capable of resisting the greater of the following:

a. The force set forth in Section 12.11.1.
b. A force of $400S_{DS}\,I$ lb/linear ft ($5.84S_{DS}\,I$ kN/m) of wall
c. 280 lb/linear ft (4.09 kN/m) of wall

Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1,219 mm).

12.11.2.1 Anchorage of Concrete or Masonry Structural Walls to Flexible Diaphragms. In addition to the requirements set forth in Section 12.11.2, anchorage of concrete or masonry structural walls to flexible diaphragms in structures assigned to Seismic Design Category C, D, E, or F shall have the strength to develop the out-of-plane force given by Eq. 12.11-1:

$$F_p = 0.8S_{DS}IW_p \qquad (12.11\text{-}1)$$

where

F_p = the design force in the individual anchors
S_{DS} = the design spectral response acceleration parameter at short periods per Section 11.4.4
I = the occupancy importance factor per Section 11.5.1
W_p = the weight of the wall tributary to the anchor

For embedded straps see ASCE 7 Sections 12.11.2.2.5. For walls with pilasters ASCE 7 Section 12.11.2.2.7 requires:

ASCE 7 Section 12.11.2.2.5 and 12.11.2.2.7

12.11.2.2.5 Embedded Straps. Diaphragm to structural wall anchorage using embedded straps shalll be attached to , or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

12.11.2.2.7 Walls with Pilasters. Where pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall not be reduced.

Additional requirements for vertical and horizontal structural irregularities are given in ASCE 7 Tables 12.3-1 and 12.3-2. The tables state irregularity types and guides the user to the applicable code section and gives the Seismic Design Category application.

ASCE 7 TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force-resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

ASCE 7 TABLE 12.3-2 VERTICAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness-Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness-Extreme Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
2.	**Weight (Mass) Irregularity** is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
3.	**Vertical Geometric Irregularity** is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity** is defined to exist where an in-plane offset of the lateral force-resisting elemetns is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
5a.	**Discontinuity in Lateral Strength-Weak Story Irregularity** is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
5b.	**Discontinuity in Lateral Strength-Extreme Weak Story Irregularity** is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

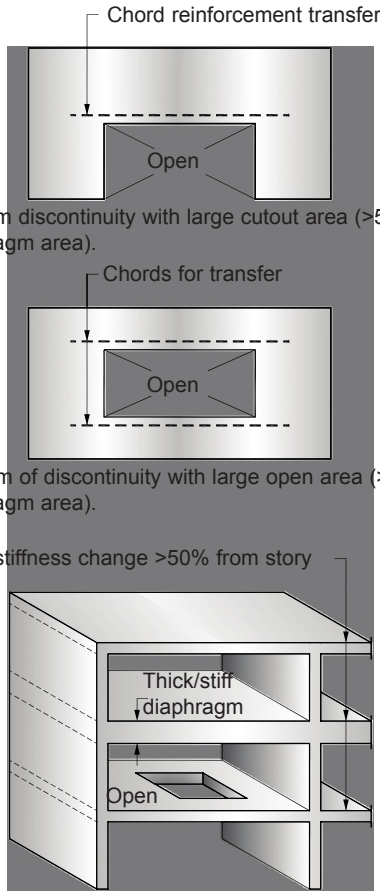
The tables point out special cases of irregularities that need to be considered in some cases. These special cases include:

- Horizontal (Plan View) Structural Irregularities:
  - Torsional Irregularity,
  - Extreme Torsional Irregularity,
  - Reentrant Corners,
  - Diaphragm Discontinuity,
  - Out-of-plane Offsets,
  - Nonparallel Systems
- Vertical Structural Irregularities:
  - Stiffness – Soft Story
  - Stiffness – Extreme Soft Story
  - Weight or Mass Irregularity
  - Vertical Geometry Irregularity
  - In-Plane Discontinuity in Vertical Lateral Force Resisting Elements
  - Lateral Strength – Weak Story
  - Lateral Strength – Extreme Weak Story

An example of a reentrant corner is shown in Figure 4.8. This case is defined as having more than 15 percent of the plan dimension for both projections in the direction being considered. For these reentrant corner cases, design forces for connectors and chord transfer forces must be increased 25 percent for Seismic Design Categories D, E, and F.

The diaphragm discontinuity irregularity results from abrupt changes in the diaphragm stiffness, openings which exceed 50 percent of the gross area of the diaphragm, or a change of diaphragm stiffness exceeding 50 percent between floors. See Figure 4.16.

Design requirements for diaphragm discontinuities are similar to the case of reentrant corners in that the design forces on the connections, chords, and drag members are increased by 25 percent in Seismic Design Categories D, E, and F. The design may require separation of the overall diaphragm into small diaphragms with joints in between to transfer the forces and provide for independent deflection capabilities.



(a) Diaphragm discontinuity with large cutout area (>50% of gross diaphragm area).

(b) Diaphragm of discontinuity with large open area (>50% of gross diaphragm area).

(c) Diaphragm discontinuity with a change in diaphragm stiffness >50% from story to story.

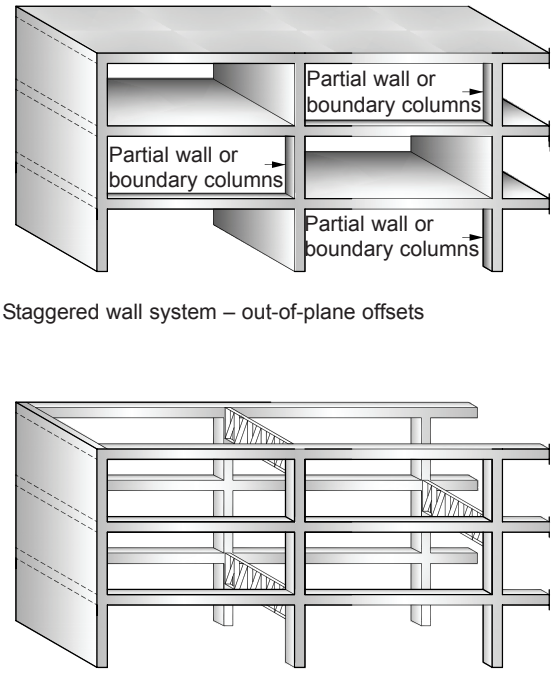
FIGURE 4.16 Diaphragm discontinuity.

Staggered truss/wall systems are a common out-of-plane offset example as shown in Figure 4.17.

Offset walls can subject diaphragms to large demands to transfer shear forces from the wall above to the wall below. Connections, chords, and drag members are subjected to a 25 percent increase in design forces in Seismic Design Categories D, E, and F.

The above-described irregularities are for those appearing in plan view and most often applied to the floor or diaphragm system of the building.

ASCE 7 Table 12.3-2 addresses cases for vertical irregularities. Soft story and extreme soft story cases are adequately described in the table and are not further described in this text, except to point out the In-Plane Discontinuity in Lateral Force Resisting Element Case, as depicted in Figure 4.18.



(a) Staggered wall system – out-of-plane offsets

(b) Staggered truss system – out-of-plane offsets

FIGURE 4.17 Staggered wall/truss system showing out-of-plane offset.

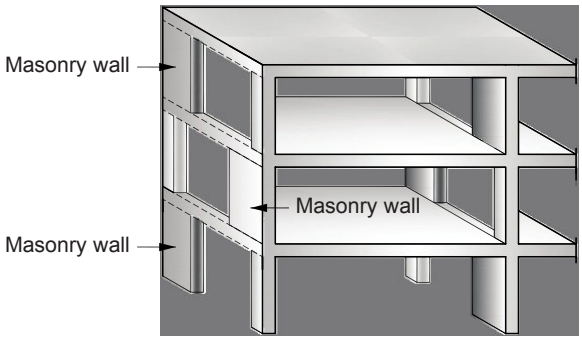


FIGURE 4.18 In-plane discontinuity in a lateral force resisting element.

4.6 DRIFT AND DEFORMATION

Section 12.12 of ASCE 7 provides drift and deformation requirements or limitations for story drift, diaphragm deflections and related items.

ASCE 7 Section 12.12.1

12.12.1 Story Drift Limit. The design story drift (Δ) as determined in Sections 12.8.6, 12.9.2, or 16.1, shall not exceed the allowable story drift (Δ_a) as obtained from Table 12.12-1 for any story. For structures with significant

torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Types 1a or 1b of Table 12.3-1, the design story drift, Δ, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

12.12.1.1 Moment Frames in Structures Assigned to Seismic Design Categories D through F. For seismic force-resisting systems comprised solely of moment frames in structures assigned to Seismic Design Categories D, E, or F, the design story drift (Δ) shall not exceed Δ_a/ρ for any story. ρ shall be determined in accordance with Section 12.3.4.2.

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a[a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.

[b] For seismic force-resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.

[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.

[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

The diaphragm deflection requirements are contained in ASCE 7 Section 12.2.2:

ASCE 7 Section 12.12.2

12.12.2 Diaphragm Deflection. The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

The "Building Separation" and "Deformation Compatibility for Seismic Design Categories D through F" are contained in ASCE 7 Sections 12.12.3 and 12.12.4, respectively.

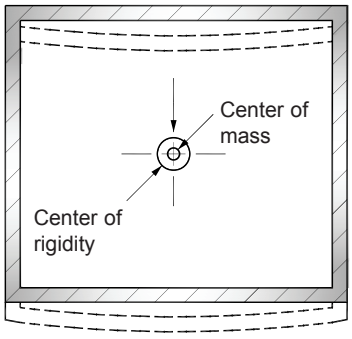
4.7 TORSION

4.7.1 GENERAL

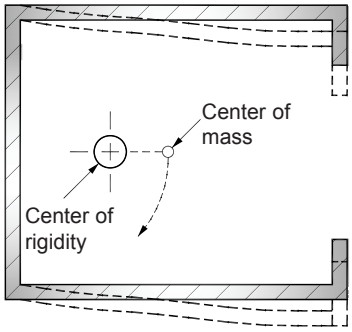
In a shear wall building with rigid floor and roof diaphragms, the seismic forces are resisted by shear wall elements in proportion to their rigidities. If all lateral force resisting elements have the same stiffness and are symmetrically located, they will be equally loaded by lateral forces.

However, if some walls are stiffer than others, or if they are unsymmetrically located, some lateral force resisting elements will resist more load than others. This condition of the center of rigidity not coinciding with the center of mass creates torsional moments. The center of mass tends to rotate about the center of rigidity.

If a building has an open front, severe torsional stresses may occur since a large eccentricity exists between the building's center of mass and the center of rigidity (see Figure 4.19). Because of the torsion, lateral forces resisted by some shear walls will be significantly increased.



a. Equal deflection of walls



b. Unequal deflection of walls due to torsion

FIGURE 4.19 Lateral distortions of buildings.

For safety, most buildings having rigid diaphragms should be designed considering at least 5 percent accidental torsional eccentricity to account for variances in materials and locations of walls and mass. ASCE 7 Section 12.8.4.2 requires this eccentricity to be added to the calculated eccentricity (see Figure 4.20). Therefore, the following relationships apply:

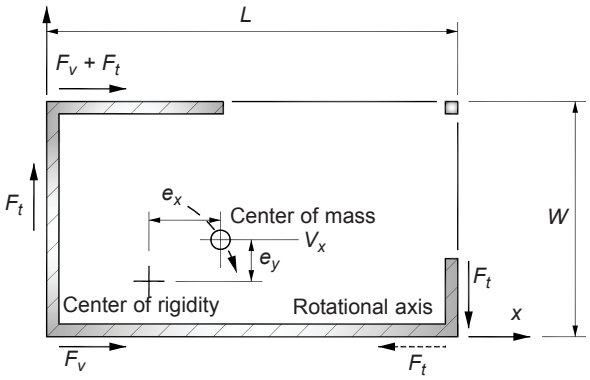


FIGURE 4.20 Plan of building showing location of center of mass and center of rigidity. Shear and torsional forces are shown.

Torsional moment = V_x (e_y)
= V_y (e_x)

Note:
ex = *ex* (calculated) ± 0.05 L
ey = *ey* (calculated) ± 0.05 W

4.7.2 TORSION CATEGORIES

ASCE divides torsion into the following categories:

- Inherent Torsion
- Accidental Torsion
- Amplification of Accidental Torsional Moment

4.7.2.1 INHERENT TORSION

The inherent torsional moment is caused by the eccentricity between the center of mass and center of rigidity. When diaphragms are non-flexible, the distribution of lateral forces must consider the consequence of inherent torsional moment.

Distribution of forces from horizontal to vertical elements requires consideration of the loads imposed and the distribution of the loads in a flexible diaphragm system.

4.7.2.2 ACCIDENTAL TORSION

Non-flexible diaphragms must take accidental torsion into consideration. There is an assumed displacement of the center of mass from actual location. This assumed displacement is 5% each way, two dimensionally, from the actual location. Accidental torsion is considered in addition to inherent torsion.

4.7.2.3 AMPLIFICATION OF THE ACCIDENTAL TORSION

Structures in SDC C and above that have Type 1a or 1b torsional irregularities as defined in ASCE 7 Table 12.3-1 must consider a torsional amplifier factor (A_x). The accidental torsional moment M_{ta} is multiplied by A_x. The maximum value of A_x is 3.0, with the most severe loading for each element considered in design.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 \qquad \text{(ASCE 7 Eq 12.8-14)}$$

where

δ_{max} = the maximum displacement at Level *x* (in. or mm) computed assuming A_x = 1

δ_{avg} = the average of the displacements at the extreme points of the structure at Level *x* computed assuming A_x = 1 (in. or mm)

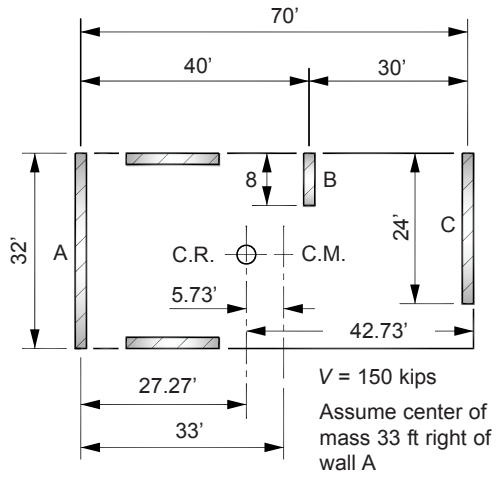
Note that the A_x term is usually used to amplify only the accidental torsion component and not the natural torsion component, and is not applied to amplifying both components together at the same time.

ASCE 7 Table 12.3-1 shows another category termed extreme torsional irregularity. These structures exist when story drift, including accidental torsion, at one end of the structure is more than 1.4 times the average of the story drifts at the two ends of the structure. These structures are subjected to the same design requirements as those with torsional irregularity, except that buildings having extreme torsional irregularity are not permitted in Seismic Design Categories E and F.

EXAMPLE 4-L Center of Rigidity.

Locate the center of rigidity for the *y* direction given the building shown below, and determine the

force distribution to each 16 ft high wall. Neglect accidental eccentricity in the *y* direction for simplicity of this problem. Neglect walls in *x* direction.



Solution 4-L

Locate the Center of Rigidity.

R_C from Table ASD-89

Wall	*h* (ft)	*d* (ft)	*h/d*	R_C	*X* (ft)	R_{cx}
A	16	32	0.50	5.000	0	0
B	16	8	2.00	0.263	40	10.52
C	16	24	0.67	3.112	70	217.84
				ΣR_C = 8.375		ΣR_{cx} = 228.36

Calculate the inherent eccentricity.

$$x_{CR} = \frac{228.36}{8.375} = 27.27 \text{ ft}$$

$$e_x = 33.0 - 27.27 = 5.73 \text{ ft}$$

Minimum e = (0.05 x 70) + 5.73

= 9.23 ft

Torsional moment = T = 150 kips x 9.23 ft

= 1384.5 ft kips

Using the polar moment of inertia, calculate the total force to each wall using calculated torsion plus 5%.

Wall	*R*	d_x	Rd_x	Rd_x^2
A	5.000	23.77	118.85	2825
B	0.263	16.23	4.27	69
C	3.112	46.23	143.87	6651
	ΣR = 8.375			ΣRd_x^2 = 9545

Force to wall = $F_v + F_T = V \times \frac{R}{\Sigma R} + T \frac{Rd_x}{\Sigma Rd_x^2}$

$$A = 150 \times \frac{5.000}{8.375} - 1384.5 \times \frac{118}{9545} = 89.6 - 17.1$$

= 72.5 kips

$$B = 150 \times \frac{0.263}{8.375} + 1384.5 \times \frac{4.3}{9545} = 4.7 + 0.6$$

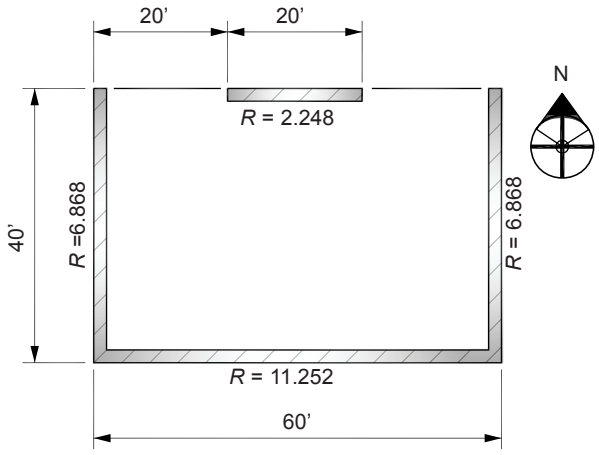
= 5.3 kips

$$C = 150 \times \frac{3.112}{8.375} + 1384.5 \times \frac{143}{9545} = 55.7 + 20.7$$

= 76.4 kips

EXAMPLE 4-M Forces to Walls, Rigid Diaphragm.

The figure below shows a plan view of a one-story masonry shear wall structure with a rigid diaphragm roof. The relative rigidity of each shear wall is given.



Determine
a. The center of mass and the center of rigidity
b. The minimum base shear and torsion values for both N-S and E-W lateral forces
c. The forces in each shear wall for a N-S earthquake

Given: Building is a one story box system;

All walls are a total of 19 ft high;

16 ft between supports with a 3 ft parapet.

Use C_S = 0.08 as the controlling value

Amplification factor, A_x = 1.0

Wall Weights:

Roof	=	75 psf
N Wall	=	75 psf
S Wall	=	100 psf
E, W Walls	=	75 psf

Solution 4-M Part a; Centers of Mass and Rigidity

Find the weight of each building component and determine the location of the center of mass.

Use h = 16/2 + 3 = 11

Thus, the weight of the E and W walls are:

75 x 11 x 40 = 33,000 lbs

Note that since the building is symmetrical with respect to the y axis, it is anticipated that x_{cm} = 60/2 = 30 ft. However, to show the methodology, calculate x_{cm}.

Item	Weight	*X* (ft)	*Y* (ft)	*Wx*	*Wy*
Roof	180 kips	30	20	5400	3600
W Wall	33 kips	0	20	0	660
E Wall	33 kips	60	20	1980	660
N Wall	16.5 kips	30	40	495	660
S Wall	66 kips	30	0	1980	0
	Σw = 328.5 kips			Σwx = 9855	Σwy = 5580

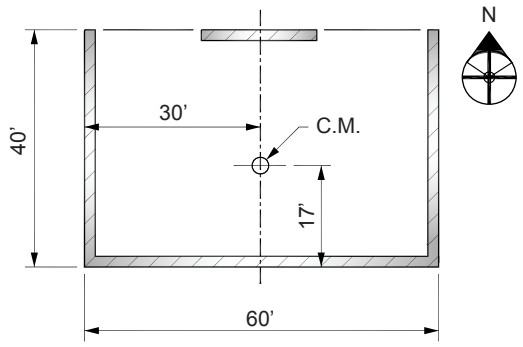
$$\bar{y}_{cm} = \frac{\Sigma Wy}{\Sigma W} = \frac{5580}{328.5}$$

= 17.0 ft north of the south wall

$$\bar{x}_{cm} = \frac{\Sigma W_x}{\Sigma W} = \frac{9855}{328.5}$$

= 30 ft to the east of the west wall

(This lies on the symmetrical centerline, as expected.)



Calculate the center of rigidity, use h = 16 ft-0 in. (neglect parapet)

Wall	*L* (ft)	*h/l*	R_{cy}	R_{cx}	*x*	*y*	yR_{cx}	xR_{cy}
N	20	0.80	–	2.248	–	40	89.9	–
S	60	0.27	–	11.252	–	0	0	–
E	40	0.40	6.868	–	60	–	–	412.1
W	40	0.40	6.868	–	0	–	–	0
			ΣR_{cy} = 13.736	ΣR_{cx} = 13.500			ΣyR_x = 89.9	ΣxR_{cy} = 412.1

C.R. y direction = $\dfrac{yR_{cx}}{\Sigma R_{cx}} = \dfrac{89.9}{13.5} = 6.7$ ft

C.R. x direction = $\dfrac{xR_{cy}}{\Sigma R_{cy}} = \dfrac{412.1}{13.736} = 30$ ft

Calculate positive torsional eccentricity

Eccentricity between center of mass and center of rigidity.

e_y = 17 - 6.7 = 10.3 ft

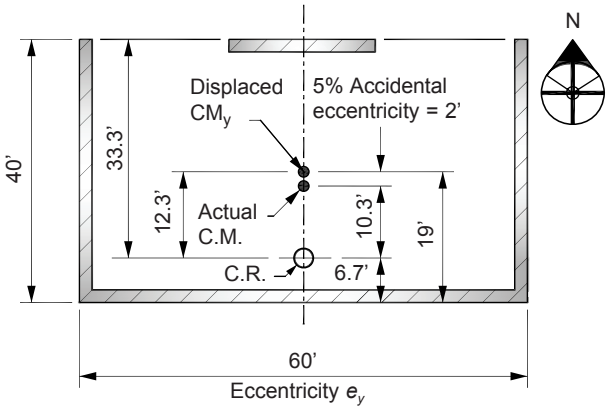
Add minimum 5% accidental eccentricity

0.05 x 40 = 2.0 ft

e_y = 10.3 + 2.0 = 12.3 ft

(Negative torsional eccentricity would be calculated using 10.3 - 2.0 = 8.3 ft)

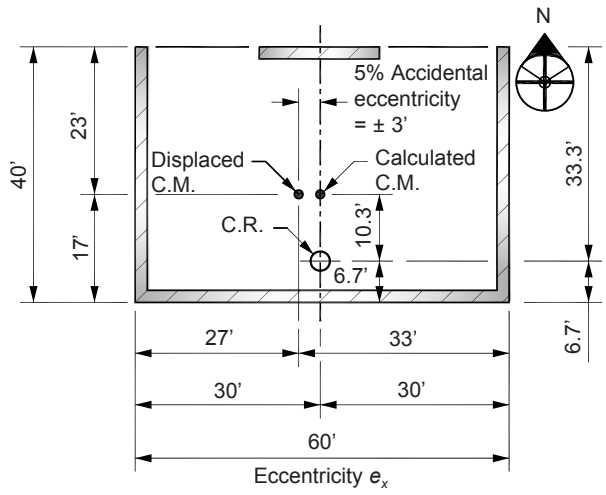
e_x = 30 - 30 = 0 ft



Include minimum 5% accidental eccentricity

0.05 x 60 = ± 3.0 ft

e_x = 0 ± 3.0 = 3 ft



Solution 4-M Part b; Base Shear

Calculate the seismic base shear

$V = C_s W$ (ASCE 7 Eq 12.8-1)

= 0.08 x 328.5 = 26.3 kips

Determine torsional moments

The torsional moments due to a N-S seismic force rotating about C.R. is:

$T = Ve_x$ = 26.3 x 3 ft

= ± 78.9 ft kips

Likewise the torsional moment due to an E-W seismic force

$T = Ve_y$ = 26.3 x 12.3 ft

= ± 323.5 ft kips

Solution 4-M Part c; Forces to Shear Walls

Determine the forces on each shear wall from a N-S earthquake, V = 26.3 kips; T = 78.9 ft kips

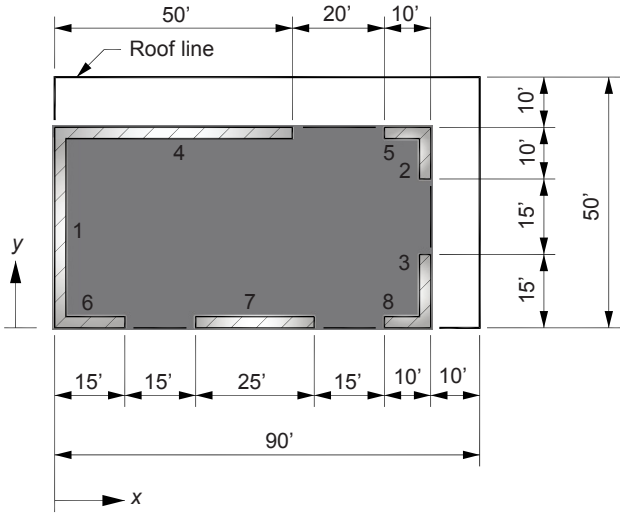
Force due to shear $F_v = V \dfrac{R_y}{\Sigma R_y}$

Forces due to torsion $F_t = T \dfrac{Rd}{\Sigma Rd^2}$

Where V = 26.3 kips, and T = 78.9 ft kips

EXAMPLE 4-N Center of Mass and Rigidity.

Locate the center of mass, C.M., and the center of rigidity, C.R., for the industrial structure shown. This is only an example of how to combine walls of different strengths and thicknesses. Usually, a consistent strength requirement and uniform thickness throughout the structure is suggested.



Distribution of forces for a seismic force in the N direction (Solution 4-M Part c)

Wall	R_y	R_x	d_x (ft)	d_y (ft)	Rd	Rd^2	Direct Force F_v (kips)	Torsional Force F_t (kips)	Total Force $F_v + F_t$ (kips)
N	–	2.25	–	33.3	74.9	2495.0	–	-0.7	-0.7
S	–	11.25	–	6.7	75.4	505.0	–	+0.7	+0.7
E	6.87	–	30	–	206.1	6183	13.2	+1.9	15.1*
W	6.87	–	30	–	206.1	6183	13.2	-1.9	11.3

$\Sigma R_y = \overline{13.7}$ $\Sigma R_x = \overline{13.5}$ $\Sigma Rd^2 = \overline{15{,}366}$ $\Sigma = \overline{26.4} = V$

* Since the East and West walls are symmetrical, use F = 15.1 kips for both walls (Earthquake force can act in either N or S direction).

All concrete block walls are 18 ft high. There are no openings, windows or doors in the walls. The roof is a rigid concrete slab 8 in. thick and weighs 70 psf.

Walls are cantilevered from the base.

Solution 4-N

The values for rigidity, R_c, from Table ASD-89 are based on t = 1 in. and E_m = 1,000,000 psi. Equate an 8 in. thickness to a base of 1 in and correct walls of other thicknesses by multiplying R_c by 1/8. Correct R_c for variations in the modulus of elasticity by multiplying R_c by E_m/1,000,000. However because $E_m = 900f'_m$, the value of R_c may be corrected by $900f'_m/1{,}000{,}000 = f'_m/1111$.

Location of center of rigidity

ΣR_c in the y direction = 12.38

ΣR_c in the x direction = 20.45

$$x = \frac{\Sigma xR_{cy}}{\Sigma R_{cy}} = \frac{420.59}{12.38} = 33.97 \text{ ft}$$

$$y = \frac{\Sigma yR_{cx}}{\Sigma R_{cx}} = \frac{455.23}{20.45} = 22.26 \text{ ft}$$

Location of center of mass of walls

$$\bar{x} = \frac{\Sigma xW}{\Sigma W} = \frac{11{,}197}{295.2} = 37.93 \text{ ft}$$

$$\bar{y} = \frac{\Sigma yW}{\Sigma W} = \frac{5{,}898.2}{295.2} = 19.98 \text{ ft}$$

Properties of Each Wall

Wall No. A	Thickness (inches) B	Thickness Correction $t/8$ C	f'_m (psi) D	E_m Correction $f'_m/1111$ E	Combined Correction for R_c C x E = F
1	8	1.00	1350	1.22	1.22
2	12	1.50	3000	2.70	4.05
3	12	1.50	3000	2.70	4.05
4	8	1.00	1500	1.35	1.35
5	12	1.50	1500	1.35	2.03
6	10	1.25	2000	1.80	2.25
7	10	1.25	2000	1.80	2.25
8	10	1.25	2000	1.80	2.25

Determination of Center of Rigidity h = 18'-0"

Wall No.	Direction	Length (ft)	h/l	R_c from Table ASD-89	Correction Coefficient	Corrected R_c	x (ft)	xR_{cy}	y (ft)	yR_{cx}
1	y	40	0.45	5.833	1.22	7.116	0.33	2.35		
2	y	10	1.80	0.348	4.05	1.409	79.50	112.05		
3	y	15	1.20	0.951	4.05	3.852	79.50	306.20		
4	x	50	0.36	7.895	1.35	10.658			39.67	422.81
5	x	10	1.80	0.348	2.03	0.706			39.50	27.90
6	x	15	1.20	0.951	2.25	2.140			0.42	0.90
7	x	25	0.72	2.738	2.25	6.161			0.52	3.20
8	x	10	1.80	0.348	2.25	0.783			0.52	0.41

$\Sigma R_{cy} = 12.38$ $\Sigma xR_C = 420.59$ $\Sigma yR_c = 455.23$

$\Sigma R_{cx} = 20.45$

Determination of Center Mass

Wall No.	*W* (psf)	Length (ft)	Area 18 x L	*W* (kips)	Direction	*x* (ft)	*xW*	*y* (ft)	*yW*
1	80	40	720	57.6	y	0.33	19.0	20.00	1,152.0
2	120	10	180	21.6	y	79.50	1,717.2	35.00	756.0
3	120	15	270	32.4	y	79.50	2,575.8	7.50	243.0
4	80	50	900	72.0	x	25.00	1,800.0	39.67	2,856.0
5	120	10	180	21.6	x	75.00	1,620.0	39.50	853.2
6	100	15	270	27.0	x	7.50	202.5	0.42	11.3
7	100	25	450	45.0	x	42.50	1,912.5	0.42	18.9
8	100	10	180	18.0	x	75.0	1,350.0	0.42	7.6

$\Sigma W = \overline{295.2}$ kips $\quad \Sigma xW = \overline{11{,}197.0} \quad \Sigma yW = \overline{5{,}898.2}$

Assume center of mass of roof coincides with geometric center of roof

$$\bar{x} = 45 \text{ ft} \qquad \bar{y} = 25 \text{ ft}$$

Weight of roof = 90 x 50 x 0.07 ksf = 315 kips

Combined center of mass

$$\bar{x} = \frac{\overset{\text{(walls)}}{295.2 \text{ x } 37.93} + \overset{\text{(roof)}}{315 \text{ x } 45}}{295.2 + 315} = 41.58 \text{ ft}$$

$$\bar{y} = \frac{\overset{\text{(walls)}}{295.2 \text{ x } 19.98} + \overset{\text{(roof)}}{315 \text{ x } 25}}{295.2 + 315} = 22.57 \text{ ft}$$

Eccentricity = C.M. - C.R.

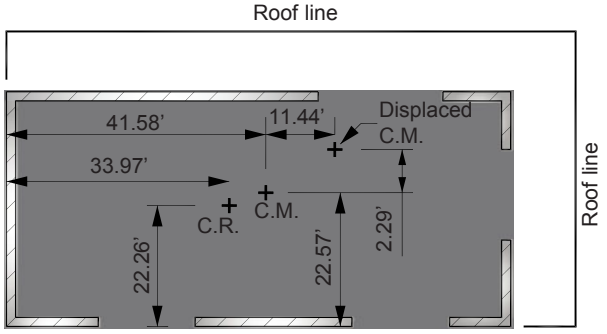
x direction = 41.58 - 34.14 = 7.44 ft

y direction = 22.57 - 22.28 = 0.29 ft

The design eccentricity is increased by 5% of the building dimension perpendicular to the direction of the seismic force (ASCE 7 Section 12.8.4.2).

x direction = 7.44 + 0.05 x 80 = 11.44 ft

y direction = 0.29 + 0.05 x 40 = 2.29 ft



4.8 BASE ISOLATION

4.8.1 GENERAL

Structures that are floated or isolated from seismic motions are not to be subjected to high earthquake forces. This technique of isolating the base of a structure is now an acceptable design and construction alternative and holds great promise for future structures.

Base isolators are a horizontally flexible and vertically rigid structural element which allows large lateral deformations due to seismic loads.

Essentially, base isolation greatly reduces the transmission of violent seismic shaking of the earth to the structure. In effect, it decouples the structure from the ground and changes the response of the building. This shift in response significantly reduces the buildings acceleration and interstory drift.

A variety of isolation systems can be used, depending on variables such as the structural system, availability of isolators, required isolator properties, and economy. The system should provide a significant change in the period of motion between the earth and the structure to adequately decouple the building from the ground. The period of the isolation system should be two to three times that of the building period.

A good example of the differential in period between the soil and a structure was shown dramatically in the October 17, 1985 Mexico City Earthquake. Frame buildings which had a long period of vibration built on a base of solid rock, or on alluvial soil having short periods of vibration, survived the shaking well. Similar buildings built on the deep soft

soil of the Mexico City lake bed were significantly damaged since the long period of vibration of the soil was close to the period of vibration of the tall frame buildings. Thus the vibrations magnified through the soft soils and into the buildings.

Likewise, stiff buildings with very short periods of vibration founded on the soft mud of Mexico City performed very well, while rigid buildings on rock or stiff soil were damaged.

The performance of these buildings in Mexico exemplifies the principle of seismic isolation in that there must be a large differential in soil/site period to the building period. Base isolators create such a differential above and below the isolation interface.

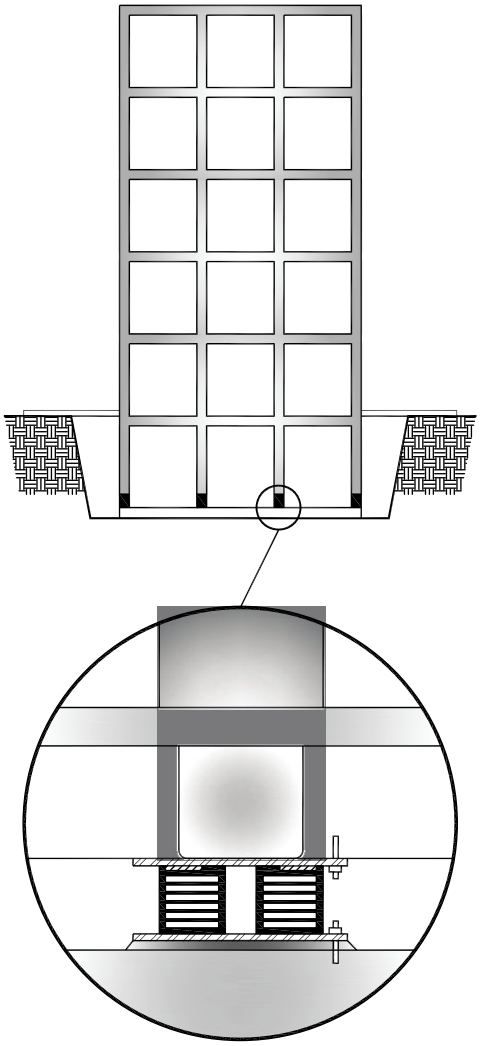


FIGURE 4.21 Building constructed on base isolators.

An isolation system should be flexible enough to lengthen the period of vibration, thereby reducing the response of the structure. At the same time, the structure must be sufficiently rigid to transmit wind loads without activating the isolation system. In this case the structure should be able to absorb more energy and be a more effective damper to control deflection. However, note that the structure must also be sufficiently rigid at low wind loads.

One type of base isolators are lead-filled elastomeric bearings, which provide the required flexibility, damping and low load rigidity. They have been used successfully on many structures and have been proven by performance in actual seismic events.

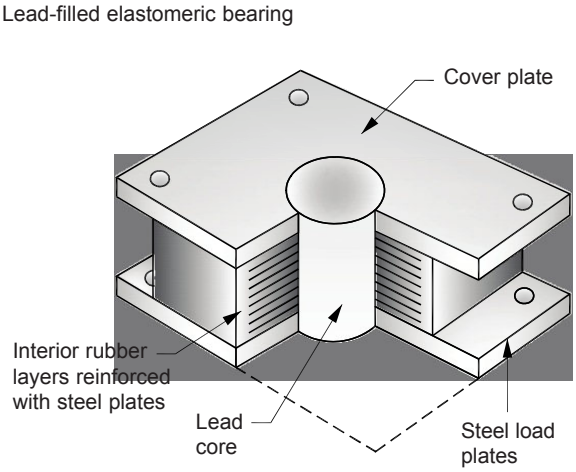


FIGURE 4.22 Steel, lead and rubber mechanical energy dissipating device.

4.8.2 PRINCIPLES OF SEISMIC REDUCTION

The principles of seismic isolation are represented in Figure 4.23. Figure 4.23 depicts the earthquake force imposed on the superstructure above the isolators as a function of the period of the superstructure.

Curve 1 plots the real force on a non-isolated structure that responds elastically to seismic action. Note that as the period increases, the seismic force is reduced.

Curve 2 plots the force on a non-isolated structure that is designed in accordance with the code to respond inelastically to seismic action. This indicates that the structure would reach plastic yielding and thus the period would be increased. However, the structure may suffer significant damage.

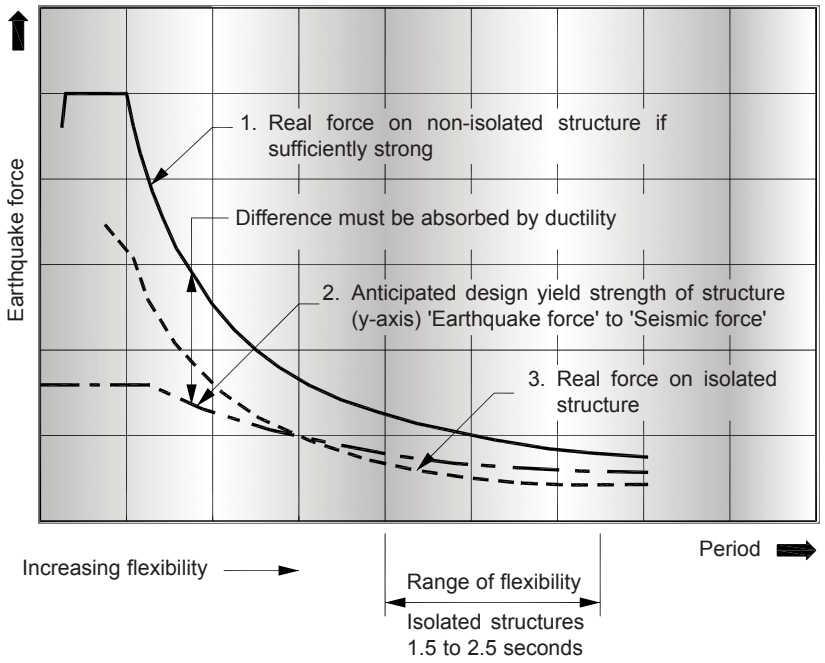


FIGURE 4.23 **Design principles of seismic isolation.**

Curve 3 plots the force on an isolated structure that responds elastically to seismic action. The curve is shifted downward from curve 1, due to the isolation system reducing the ground motion experienced by the superstructure. Isolated structures are required by code to respond nearly elastically. This is to avoid having inelastic actions reduce the stiffness of the structure, which would increase the period of the superstructure, moving it closer to the period of the isolation system.

Shear wall buildings are typically stiff and have very short periods. Accordingly, they are subjected to high seismic forces and must be designed for high force levels. By isolating a shear wall building from the seismic acceleration of the ground (decoupling it) the period is lengthened and the response and force levels are significantly reduced.

4.9 QUESTIONS AND PROBLEMS

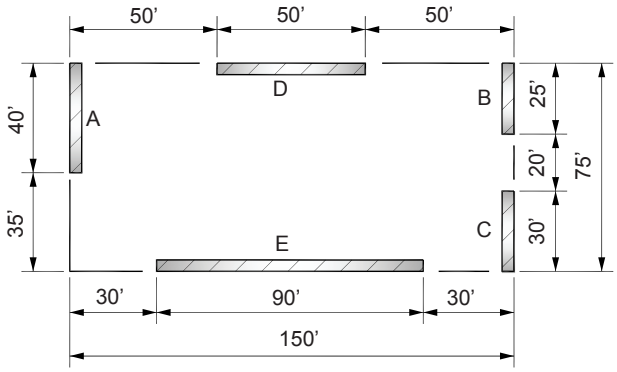
4-1 What is a horizontal diaphragm and how does it function to resist lateral forces on a building?

4-2 What are the requirements for diaphragm anchorage?

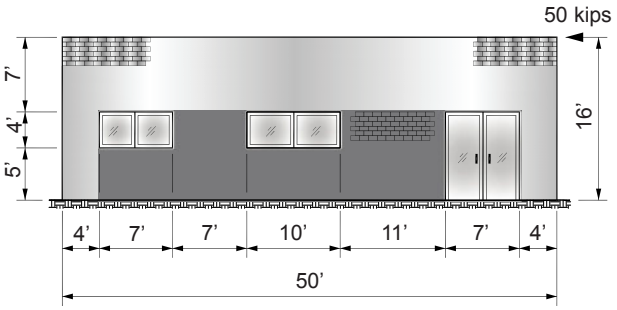
4-3 What are the effects of the deflection of a diaphragm on the load on a wall?

4-4 A building 60 ft by 180 ft with 9 in. thick brick walls (w = 90 psf), that are 18 ft high is located in Seismic Design Category C. Assume the roof dead load is 15 lbs per square foot and the live load is 20 lbs per square foot. What is the shear force per linear foot which the roof diaphragm delivers to the side walls? Specify the anchor bolts required for a 4 x 12 ledger on side walls and longitudinal walls.

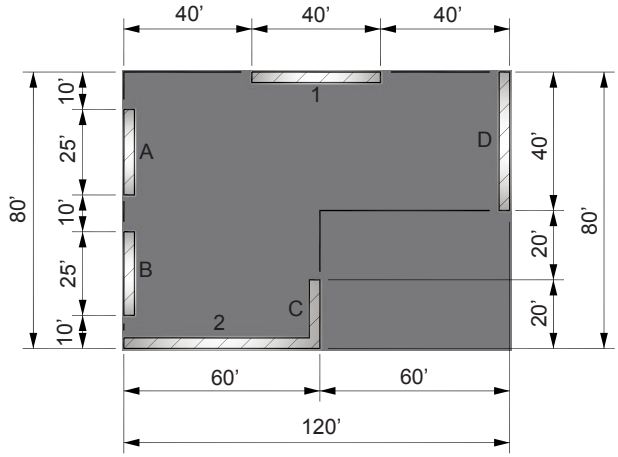
4-5 What are flexible and rigid diaphragms? Given the following plan, what is the force to each of the walls A, B, C, D and E if a flexible diaphragm is used? What are the forces in these walls if a rigid diaphragm is used? Assume the walls are cantilevered from the foundation and are 20 ft high. The lateral force is 750 plf.



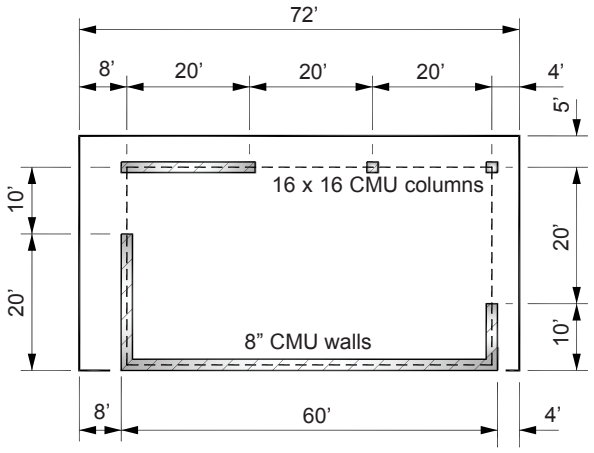
4-6 Compare the following wall, shown with openings to a similar wall without any openings? Determine the rigidity of the wall in each case. If a lateral force on the wall with openings is 50 kips, what is the shear force in each of the wall elements?



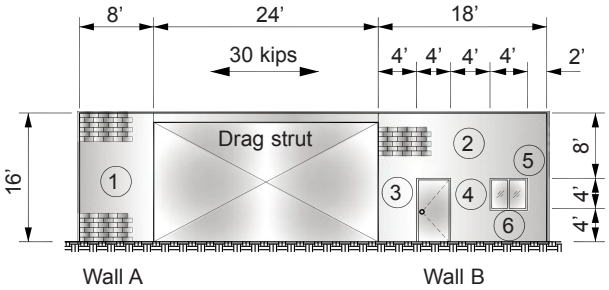
4-7 Locate center of mass and center of rigidity for the plan shown. Assume roof is a rigid diaphragm that is 4 in. of concrete on a metal deck (w = 55 psf). What are the forces to each wall shown if there is a lateral force on the wall of 700 lbs per linear foot? Assume all walls are 24 ft high and cantilevered from the base.



4-8 Determine the centers of mass and rigidity of the building shown. The walls cantilevered from the foundation are 24 ft high. Assume rigid concrete roof weighs 65 lbs per square foot, walls weigh 78 lbs per square foot and the columns weigh 270 lbs per linear foot. The base shear is 150 kips in either direction. Determine the force in each of the walls.



4-9 The 8 in. interior shear wall shown is solid grouted concrete masonry with f'_m = 1500 psi, f_y = 60,000 psi and no special inspection (use half stresses). The seismic load from the flexible roof diaphragm is 30 kips applied at the top wall.



Determine the lateral load in piers 1, 2, 3, 4, 5 and 6 due to the 30 kips load, neglecting the weight of the walls for seismic effects. Also determine the maximum anchorage load from the drags struts to the walls. Assume pin ends and no axial deformation of the strut. If the load at the top of wall B is 25 kips what will be the axial load in pier Number 5?

4-10 How are torsional shear forces distributed in a building? What is the minimum eccentricity that must be used in the calculations for torsion in a building. Are negative torsional shears deducted from the direct force shear?

4-11 What is base isolation and how does it function? Is it advantageous to use base isolation in resisting wind loads? Is base isolation beneficial if (a) there is a soft soil and a flexible building? (b) if the soil is rock and the building stiff? (c) if the soil is soft and the building is rigid?

CHAPTER 5

DESIGN OF STRUCTURAL MEMBERS BY ALLOWABLE STRESS DESIGN (ASD)

5.1 HISTORY

Prior to the 1933 Long Beach, California earthquake, masonry structures were generally unreinforced and designed by empirical procedures based on the past performance of similar structures. Since reinforcing steel was not utilized, early masonry structures tended to be massive in order to effectively resist lateral as well as vertical loads. Although this empirical procedure is still permitted to be used in lower Seismic Design Categories, the Long Beach earthquake showed engineers that a more defined and logical procedure was necessary to design structures that would effectively withstand higher seismic forces.

During this time, elastic working stress design procedures were being used to design reinforced concrete structures. Based on this elastic design approach, engineers began reinforcing masonry so that the steel could resist tensile forces while the masonry carried compressive forces.

By 1937, the Uniform Building Code included working stress design procedures for masonry which allowed engineers to size masonry members by ensuring that anticipated service loads did not exceed allowable design stresses.

With the working stress design method, engineers have designed masonry structures throughout much of the 20th century.

Allowable stress design continues to evolve as masonry design enters the 21st century. As an example, the UBC made a distinction for allowable design stresses based on whether or not masonry was continuously inspected. The current requirements, based on the IBC and MSJC Code, do not consider a stress adjustment due to inspection, but require an appropriate inspection level and allow full design stresses.

5.2 PRINCIPLES OF ALLOWABLE STRESS DESIGN

5.2.1 GENERAL, FLEXURAL STRESS

The design and analysis of reinforced masonry structural systems have traditionally been by the straight line, elastic working stress method. This procedure assumes the masonry resists compressive forces and reinforcing steel resists tensile forces.

In Allowable Stress Design (ASD), the limits of allowable stress (Tables ASD-3 and ASD-4) for the materials are established based on the properties of each material. The actual or code live loads and dead loads must not cause stresses in the structural section that exceed these allowable values.

The procedure presented is based on the working stress or straight line assumptions where all stresses are in the elastic range and:

1. Plane sections before bending remain plane during and after bending.
2. Stress is proportional to strain which is proportional to distance from the neutral axis.
3. Modulus of elasticity is constant throughout the member.

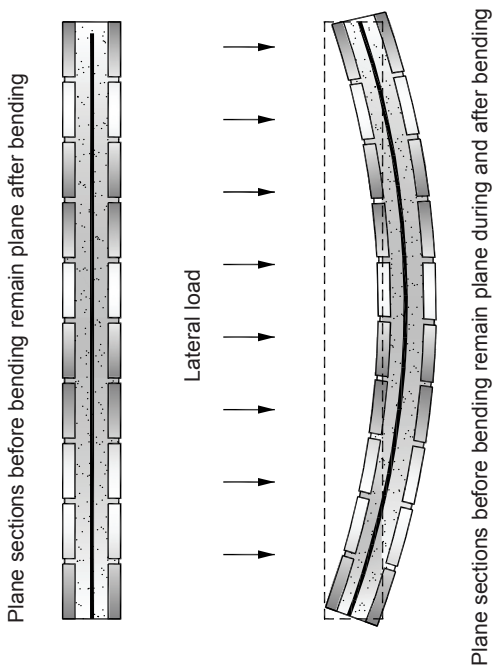


FIGURE 5.1 Wall in flexure.

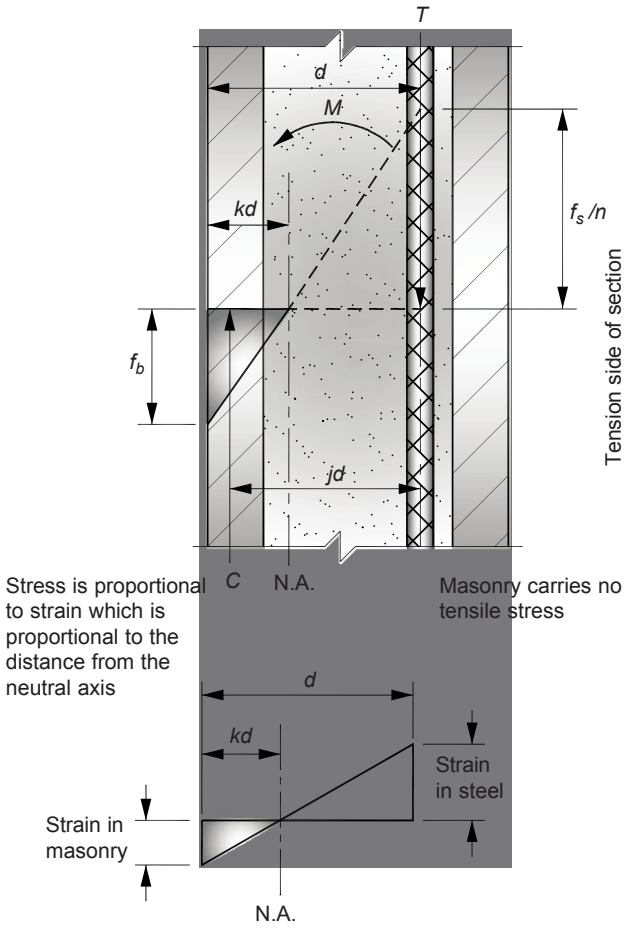


FIGURE 5.2 Stresses and strains in wall due to out of plane lateral loads, perpendicular to the plane of the wall.

4. Masonry carries no tensile stresses.
5. Span of the member is large compared to the depth (except possibly shear walls).
6. Masonry elements combine to form a homogeneous and isotropic member.
7. External and internal moments and forces are in equilibrium.
8. Steel is stressed about the center of gravity of the bars equally.
9. The member is straight and of uniform cross-section.

These assumptions are in keeping with homogeneous elastic materials. For heterogeneous materials, such as reinforced masonry, these assumptions are satisfactory for normal working stress levels. For high stress levels many of the assumptions may not be applicable, particularly Items 2 and 5, since stress may not be proportional to strain.

In evaluating a design, one should understand whether a design is tension or compression controlled, which is the most basic way to understand how hard a section is working. For example, once past the balanced condition adding significant reinforcement may not significantly increase the capacity. The examples in this section should help to understand the process.

5.3 DERIVATION OF FLEXURAL FORMULAS

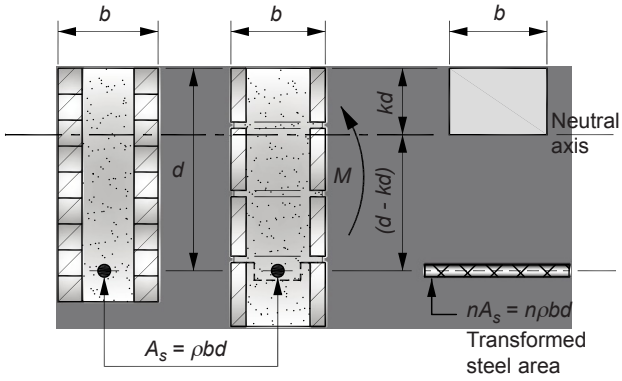
The basis of the flexural equations for Allowable Stress Design (ASD) techniques of heterogeneous systems in which one material resists compression and the other material with different physical properties resists tension is the concept of modular ratio. The modular ratio, n, is the ratio of the modulus of elasticity of steel, E_s, to the modulus of elasticity of masonry, E_m.

$$n = \frac{E_s}{E_m}$$

By use of the modular ratio, n, the steel area can be transformed into an equivalent masonry area. The strain is in proportion to the distance from the neutral axis and the strain of steel can be converted to stress in the steel. In order to establish the ratio of stresses and strains between the materials, the location of the neutral axis must be located.

5.3.1 LOCATION OF NEUTRAL AXIS

Location of the neutral axis is defined by the dimension, *kd*, which is dependent on the modular ratio, *n*, and the reinforcing steel ratio, $\rho = A_s/bd$. For a given modular ratio, *n*, the neutral axis will raise by decreasing the amount of steel (reducing ρ) or will lower by increasing the amount of steel (increasing ρ).



Note: The amount of masonry below the neutral axis does not affect transformed properties.

FIGURE 5.3 Location of neutral axis for a beam.

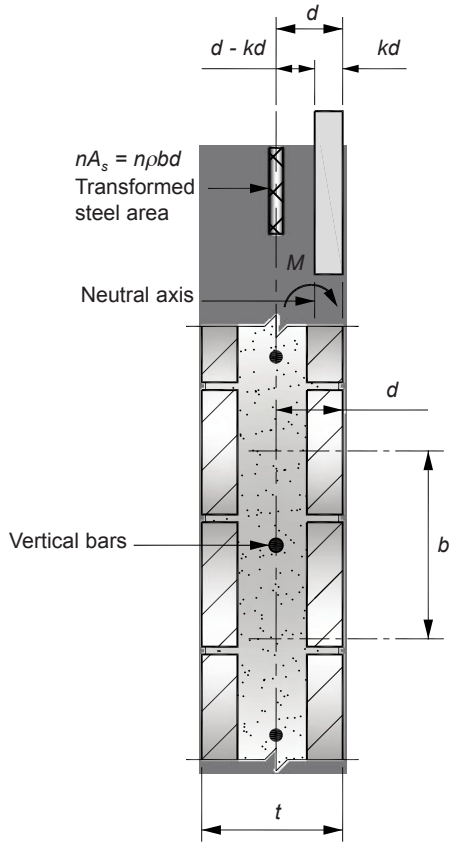


FIGURE 5.4 Location of neutral axis for a wall.

Take moments of the stress areas about the neutral axis.

Compressive stress area	x	Moment arm	=	Tensile stress area	x	Moment arm
(*bkd*)	x	($^1/_2$)(*kd*)	=	($n\rho db$)	x	(*d* - *kd*)

$$\frac{1}{2}bd^2k^2 = n\rho\left(bd^2 - kbd^2\right)$$

$$\frac{1}{2}bd^2k^2 - n\rho bd^2(1-k) = 0$$

Divide by bd^2 and multiply by 2

$$k^2 - 2n\rho\,(1 - k) = 0$$

Solving for *k*

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho$$

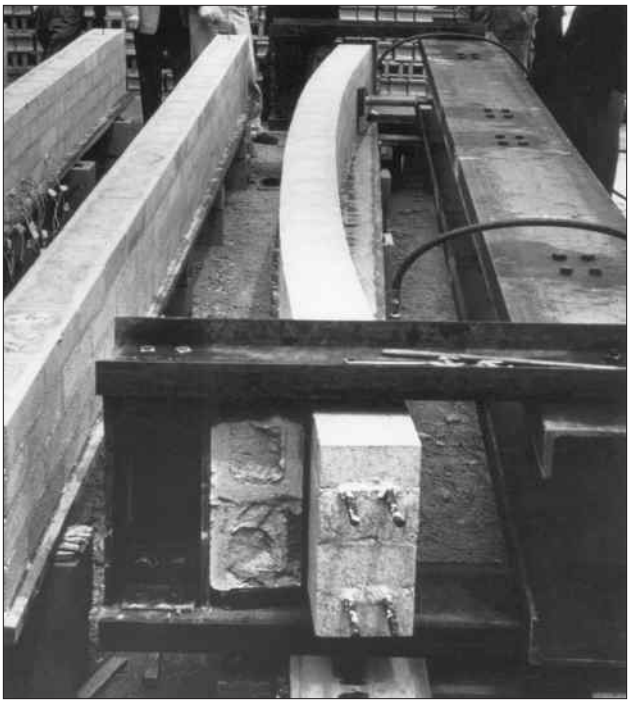


FIGURE 5.5 Reinforced masonry beam subjected to lateral forces.

5.3.2 VARIATION OF COEFFICIENTS *k*, *j* AND FLEXURAL COEFFICIENT K_f.

The coefficient *k* defines the depth of the compression area, *kd*, and is the location of the neutral axis for the section. The neutral axis is

determined by the modular ratio and the steel ratio. For under-reinforced sections where the reinforcing steel is stressed to its allowable value, the coefficient *k* will increase as the amount of steel increases. Accordingly, the depth of the compression area will also increase until the stress in masonry increases up to the allowable compressive stress. When the maximum allowable masonry stress is attained, the section is considered in a balanced stress condition, since the steel stress is already at its maximum allowable value. If the area of steel is increased, and the masonry stress is held at its maximum value, the stress in the steel decreases and the compression stress block deepens, increasing the coefficient *k*, which is determined by the equation:

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho$$

The coefficient *j* defines the distance between the centroid of the compression area and the centroid of the tensile steel area. The lever arm, *jd*, is used to compute the internal resistance moment. This lever arm, *jd*, decreases from a maximum value to a minimum value as the depth of the compressive stress block increases and is determined by the equation:

$$j = 1 - \frac{k}{3}$$

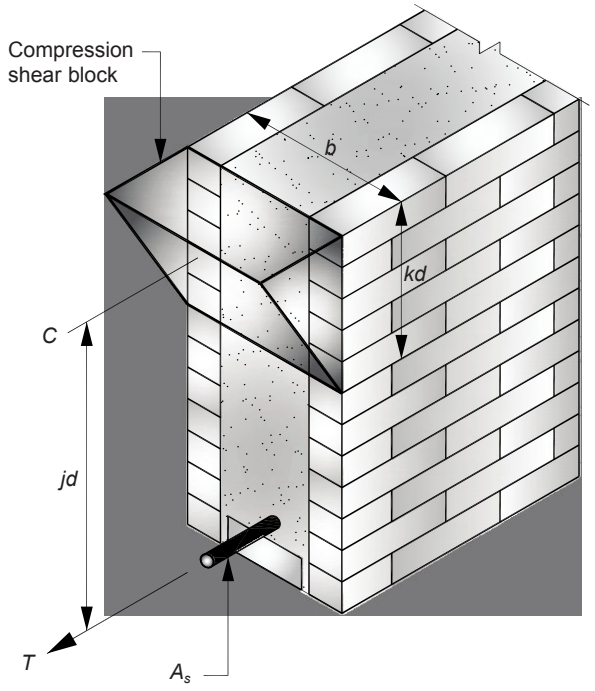


FIGURE 5.6 Compression stress block for a beam in flexure.

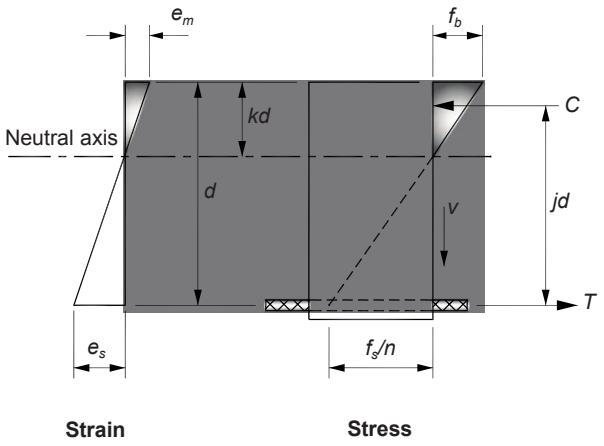


FIGURE 5.7 Stress and strain representation for a beam flexure.

The flexural coefficient K_f is a combination of values that defines the moment or flexural capacity of a section.

K_f = $^1/_2$ $f_b jk$ for flexural computations, psi.
= $f_s \rho j$ for flexural computations, psi

If steel tensile stress is maintained at its maximum allowable stress, the value of K_f will vary from minimum to maximum as the masonry compressive stress f_b increases. The value of K_f also increases as the steel tensile stress is reduced while the compressive stress in masonry is maintained at its maximum allowable stress. Tables ASD-23 through ASD-33 may be used to find K_f values easily. Alternately, K_f may be determined based on steel stress as:

$$K_f = f_s \rho j$$

or based on masonry stress as:

$$K_f = \frac{kj}{2} f_b$$

See Diagrams ASD-23 through ASD-33 for variation of K_f vs ρ for different stresses in masonry and steel.

5.3.3 MOMENT CAPACITY OF A SECTION

The moment capacity of a reinforced structural masonry wall or beam can be limited by the allowable masonry stress, (over-reinforced), allowable steel stress, (under-reinforced), or both, in which case it would be a balanced design condition.

When a member is designed for specified loads and the masonry and reinforcing steel are stressed to their maximum allowable stresses, the design is said to be a "balanced" design. This balanced design is different than the balanced design for strength design method. (See Section 6.4.1.1). For working stresses, balanced design occurs when masonry is stressed to its maximum allowable compressive stress and steel is stressed to its maximum allowable tensile stress.

However, in many cases, the "balanced" design does not satisfy the conditions for the materials available or for the predetermined member size or the economy of the project. There may be advantages to under-stress (under-reinforce) the masonry or under-stress (over-reinforce) the steel so that the size of the member can be maintained.

The moment capability of a section based on the steel stress is defined as:

M_s = force multiplied by the moment arm

Where:

Force in the steel, $T = A_s f_s = \rho bdf_s$

Moment Arm = jd

$M_s = T\,jd = A_s\,f_s\,jd$

$M_s = \rho bdf_s\,jd = f_s\rho jbd^2$

Also, since $K_f = f_s\rho j$,

$M_s = K_f\,bd^2$

The moment capability of a section based on the masonry stress is defined as:

M_m = force multiplied by the moment arm

Where:

Force in the masonry, $C = \frac{1}{2} f_b(kd)b = \frac{1}{2} f_b kbd$

Moment Arm = jd

$$M_m = C(jd) = \frac{1}{2} f_b kbdjd$$

$$M_m = \frac{1}{2} f_b kjbd^2$$

Since $K_f = \frac{1}{2} f_b kj$,

$$M_m = K_f bd^2$$

5.3.4 SUMMARY

The above shows general derivations for moment on a section for any stress level within the elastic straight line stress range. It assumes the section has only tensile reinforcement steel.

The primary ASD formulas for design or analysis are:

$$M = K_f bd^2 \quad \text{(in.-lbs)}$$

or $$M_m = \frac{1}{2} f_b kjbd^2 \quad \text{(in.-lbs)}$$

or $$M_s = f_s \rho jbd^2 \quad \text{(in.-lbs)}$$

and $$K_f = \frac{M}{bd^2}$$

Where:

M is the moment on the member, or moment per unit width in in.-lbs or in.-lbs/ft.

b is the width of the member in inches.

d is the depth from the outer compression fiber to the centroid of tension reinforcing steel in inches.

K_f is the flexural coefficient determined by the formulas above and is $K_f = f_s\rho j$ or $K_f = {}^1/_2 f_b kj$ psi.

Moment = stress multiplied by the section modulus

$M = f\,S$

For a solid rectangular section:

$$S = \frac{bd^2}{6}$$

$$\text{Thus, Stress} = \frac{M}{S} = \frac{6M}{bd^2}$$

This is similar to:

$$f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right)$$

and

$$f_s = \frac{M}{bd^2}\left(\frac{1}{\rho j}\right)$$

A reinforced masonry section is not symmetrical about the neutral axis. The value of *c*, which is the distance from the neutral axis to the extreme tension or compression fiber, is different for the stress in the masonry and the steel. Therefore, the section modulus, *I/c* will be different when determining the stress in the masonry or the steel.

$$S = \frac{bd^2}{\left(\frac{2}{jk}\right)}$$ for masonry, and

$S = bd^2 \rho j$ for steel

EXAMPLE 5-A Determination of Moment Capacity of a Wall.

A partially grouted 8 in. concrete masonry wall is reinforced with #6 bars at 24 in. o.c. The reinforcement is 5.3 in. from the compression face and is Grade 60. If f'_m = 2500 psi, what is the moment capacity of the wall?

Solution 5-A

For f'_m = 2500 psi

$$F_b = \frac{1}{3} f'_m = 833 \text{ psi}$$ (MSJC Code Section 2.3.3.2.2)

$E_m = 900 f'_m$ = 2,250,000 psi (MSJC Code Section 1.8.2.2.1)

Also for f_y = 60,000 psi

F_s = 24,000 psi (MSJC Code Section 2.3.2.1)

E_s = 29,000,000 psi (MSJC Code Section 1.8.2.1)

Steel ratio, $\rho = \frac{A_s}{bd} = \frac{0.44}{(24)(5.3)} = 0.0035$

Modular ratio, $n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{(900)(2500)} = 12.9$

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho$$

$$= \sqrt{[(12.9)(0.0035)]^2 + 2(12.9)(0.0035)}$$

-(12.9 x 0.0035)

= 0.259

$$j = 1 - \frac{k}{3} = 1 - \frac{0.259}{3} = 0.914$$

Determine the limiting compressive force in masonry;

$$M_m = \frac{f_b kjbd^2}{2}$$

$$= \frac{(833)(0.259)(0.914)(12)(5.3)^2}{2}$$

= 33,235 in.-lbs/ft

= 2.77 ft-k/ft

Determine the limiting tensile force in reinforcement;

$M_s = f_s \rho j bd^2$

= 24,000(0.0035)(0.914)(12)(5.3)^2

= 25,880 in.-lbs/ft

= 2.16 ft-k/ft ⟵ Controls

Alternately,

From Table ASD-26b for ρ = 0.0035 find

K_f = 76.8 f_b = 650 psi f_s = 24,000 psi

k = 0.259 j = 0.914 $\frac{2}{jk} = 8.46$

Moment capacity = $K_f bd^2$

= (76.8)(12)(5.3)^2

= 25,888 in.-lbs/ft

= 2.16 ft-k/ft (same as above)

5.3.4.1 STRAIN COMPATIBILITY

Two basic assumptions of Allowable Stress Design are that plane sections before bending remain plane during and after bending and that stress is proportional to strain which is proportional to the distance from neutral axis.

The above assumptions provide the basis for straight line values for stress and strain on the cross-section of a member subjected to moment and are illustrated by Figures 5.1, 5.2 and 5.8.

The location of the neutral axis is explained in Section 5.3.1 and is denoted as a distance, *kd*, from the compression face.

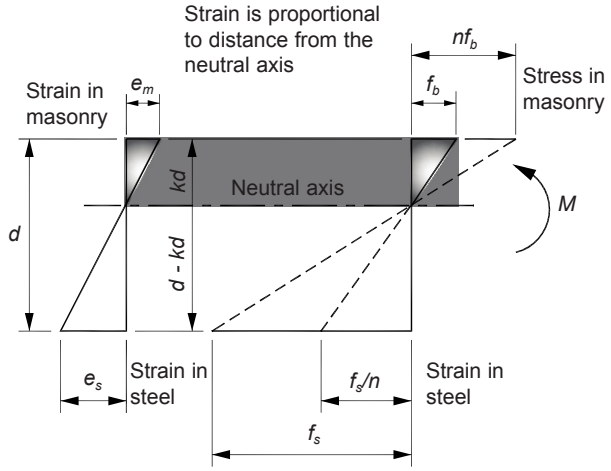


FIGURE 5.8 Relationship of stress and strain.

Stress in masonry: $f_b = e_m E_m$

Stress in steel: $f_s = e_s E_s$

Ratio of strains: $\frac{e_m}{e_s} = \frac{(kd)}{(d - kd)}$ straight line variation

Ratio of stresses: $\frac{f_b}{f_s} = \frac{e_m E_m}{e_s E_s} = \frac{e_m}{e_s}\left(\frac{1}{n}\right)$

$$\frac{f_b}{f_s} = \frac{(kd)}{(d - kd)}\left(\frac{1}{n}\right)$$

$$f_b = \frac{(kd)}{(d - kd)}\left(\frac{f_s}{n}\right)$$

This shows straight line variation of stresses when f_s is divided by modular ratio *n*.

EXAMPLE 5-B Flexural Design – Tension Reinforcement.

Determine the tension reinforcement required for a 14 ft long, simply supported, clay masonry beam using both the IBC and the MSJC Code. The beam is 9 in. wide by 24 in. deep with an effective depth, *d*, of 20 in. A superimposed live load of 1400 plf is carried by the beam as well as its own weight.

Assume

f'_m = 2500 psi

F_s = 24,000 psi

Type S mortar

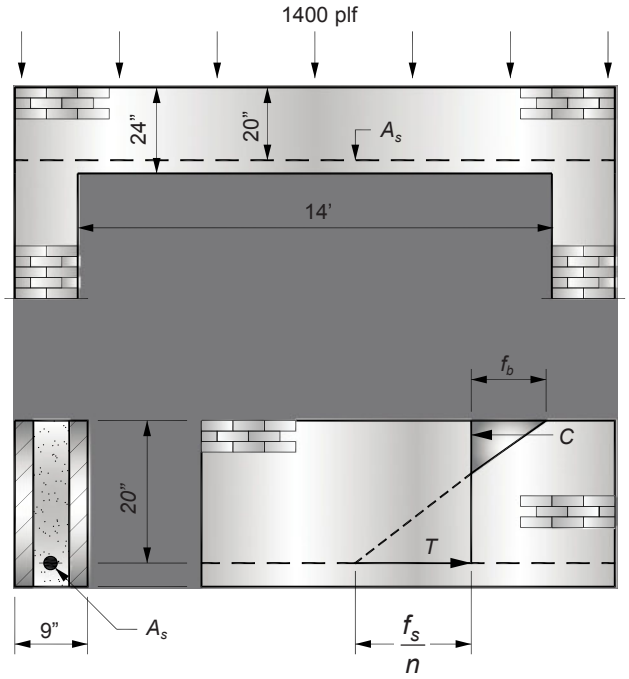


FIGURE 5.9 Beam in flexure.

Solution 5-B

Design by IBC and MSJC Code

Find the self weight of the beam from Table GN-3c as 90 psf.

$$DL = 90\left(\frac{24}{12}\right) = 180 \text{ plf}$$

$$LL = 1400 \text{ plf}$$

Total w = 1580 plf

Calculate the simple beam moment

$$M = \frac{wl^2}{8} = \frac{1580(14)^2}{8}$$

= 38,710 ft-lbs

Determine the K_f factor

$$K_f = \frac{M}{bd^2} = \frac{(38{,}710)(12)}{9(20)^2} = 129$$

Enter Table ASD-26a for clay masonry with f'_m = 2500 psi with K_f = 129:

estimate ρ = 0.0061

Therefore, the required area of steel is:

A_s = (0.0061)(9)(20) = 1.10 sq in.

From Table GN-20a, select 4 - #5 bars (A_s = 1.24 sq in.) or 2 - #7 (A_s = 1.20 sq in.)

EXAMPLE 5-C Stresses in Masonry and Reinforcing Steel.

A 10 in. thick reinforced double-wythe clay masonry wall was constructed with #7 bars at 24 in. o.c. in the center of the wall. After construction, the designer discovered that a lower moment than the required design moment of 2.5 ft-kips/ft was used. Check the masonry and steel stresses to verify the wall is not overstressed.

Use f'_m = 2000 psi and F_y = 60,000 psi

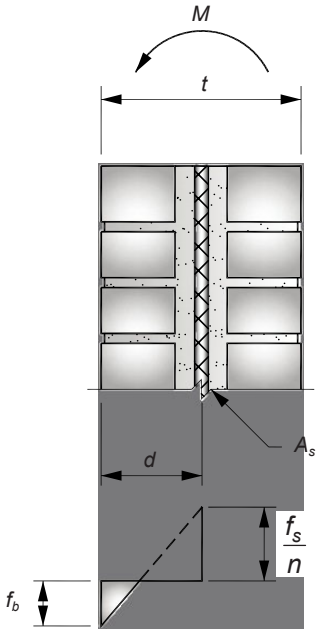


FIGURE 5.10 Stresses in wall.

Solution 5-C

(1) From Table ASD-3 and ASD-4 the allowable stresses are:

F_b = 667 psi and F_s = 24,000 psi

(2) Compute the flexural coefficient, K_f, using d = 10/2 = 5 in.

$$K_f = \frac{M}{bd^2} = \frac{(2.5)(12{,}000)}{(12)(5)^2} = 100$$

(3) Compute the reinforcement ratio, ρ

$\rho = \frac{A_s}{bd}$ or from Table GN-23c for #7 bars @ 24 in.

with d = 5 in., ρ = 0.0050

(4) Plot K_f = 100 and ρ = 0.0050 in Diagram ASD-25a to determine the actual stresses:

f_b = 650 psi and f_s = 23,000 psi

Both stresses are below the allowable values and the wall will be sufficient to withstand the increased loading.

5.3.4.2 VARIATION IN STRESS LEVELS OF THE MATERIALS

The following outlines the conditions of variable stress for the materials, masonry and reinforcing steel in which:

1) The reinforcing steel is at the maximum allowable tension stress, (the section is under-reinforced), while the masonry stress is variable from a low value up to its maximum allowable compressive stress.

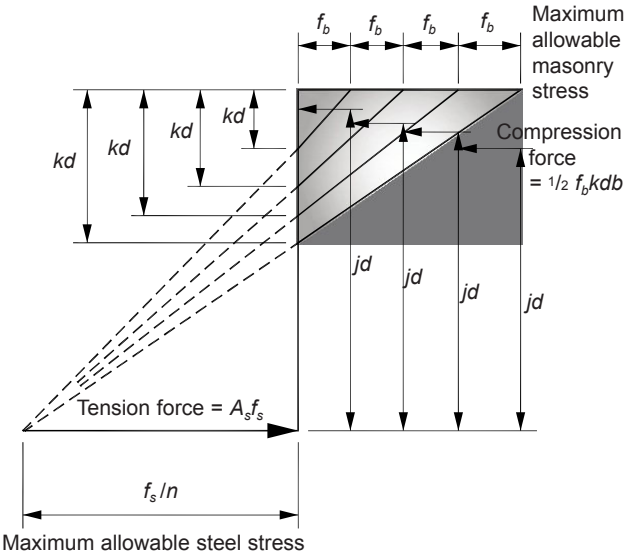
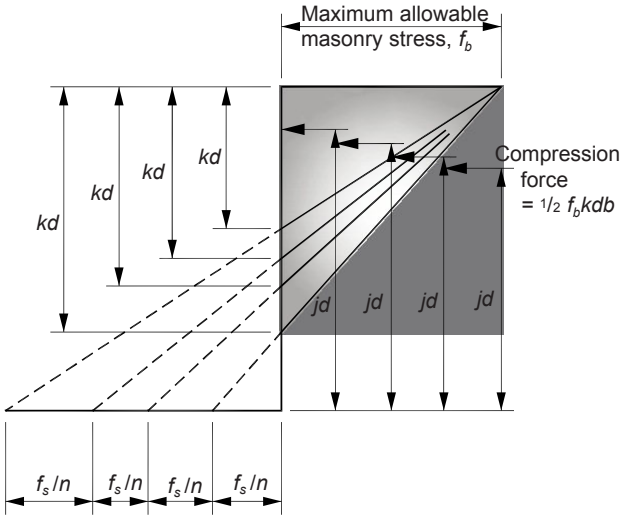


FIGURE 5.11 Maximum tensile stress and variable compression stress, under-reinforced.

2) The masonry is at the maximum allowable compression stress, (the section is over-reinforced), while the stress in the reinforcing steel is variable from a low value to the maximum allowable tension stress.



Maximum allowable steel stress, F_s

FIGURE 5.12 Maximum compressive stress with variable steel stress, over-reinforced.

EXAMPLE 5-D Flexural Design; Determination of Beam Depth and Reinforcing Steel.

For balanced working stress design conditions, find the minimum lintel depth and the required area of reinforcement.

Design Data:

Clay masonry lintel constructed with Type S mortar.

M = 40 ft-k

b = 9 in.

f'_m = 1500 psi

F_s = 24,000 psi

Neglect weight of lintel beam

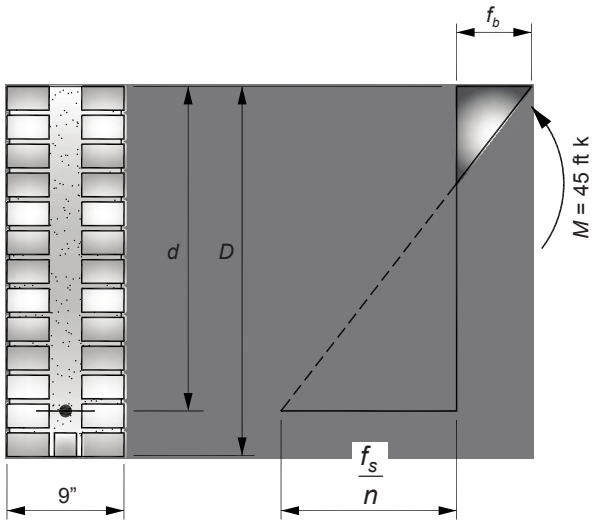


FIGURE 5.13 Moment on a beam in flexure.

Solution 5-D

Balanced design conditions occur when the maximum allowable masonry and steel stresses occur simultaneously.

(1) In Table ASD-24a, find K_{bal} = 80.2, ρ_{bal} = 0.0038

(2) $d_{min} = \sqrt{\frac{(40)(12{,}000)}{(80.2)(9)}} = 25.8$ in. Use 28 in.

Use total depth D = 26 + 6 = 32 in.

(3) $A_s = \rho bd$ = 0.0038(9)(26) = 0.89 sq in.

(4) From Table GN-20c choose 2 - #6 bars (A_s = 0.88 sq in.)

EXAMPLE 5-E Moment Capacity of Beam

Determine the moment capacity of the lintel beam shown in Figure 5.14.

Given:

b = 10 in.

total depth = 36 in.

d to steel = 30 in.

A_s = 2 - #7 bars

f'_m = 2000 psi

F_s = 24,000 psi

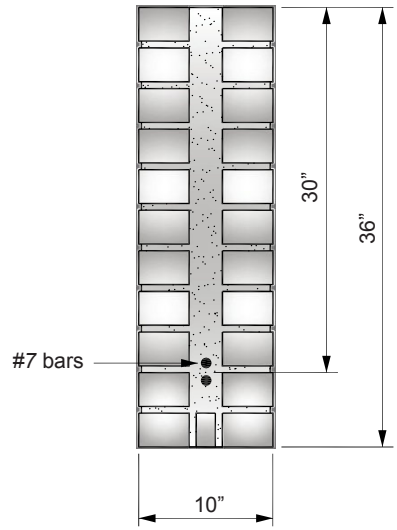


FIGURE 5.14 Beam cross section.

Solution 5-E

(1) From Table ASD-3, Allowable Stresses,
f_b = 667 psi maximum

(2) From Table GN-20a, Area of Steel
for 2 - #7 bars A_s = 1.20 sq in.

$$\text{steel ratio} = \frac{1.20}{(10)(30)} = 0.0040$$

(3) Enter Diagram ASD-25a with ρ = 0.0040
and f_b = 667 psi maximum
and f_s = 24,000 psi maximum

Proceed vertically up the ρ = 0.0040 line until either the limiting f_b line or f_s line is intersected.

The f_s = 24,000 psi is intersected first at the ordinate K_f = 85. Also read f_b = 580 psi.

(4) Moment capacity,

$$M = K_f b d^2 = 85(10)(30)^2$$
$$= 765{,}000 \text{ in.-lbs}$$
$$= 63.8 \text{ ft-kips}$$

5.3.4.3 MAXIMUM AMOUNT OF REINFORCEMENT

Chapter 2 of the MSJC Code remains silent on the issue of maximum amount of flexural reinforcement for force resistance calculations. General reinforcement provisions in MSJC Code, Table 1.16.1 Footnote 4, however, contain a limitation of 6% for the amount of vertical reinforcement permitted in the cell area. Notwithstanding, the designer must first consider the provisions in the IBC.

The 2006 edition of the IBC contains the following additional requirement for the maximum reinforcement ratio:

IBC Section 2107, Allowable Stress Design

2107.8 ACI 530/ASCE 5/TMS 402, Section 2.3.7, maximum reinforcement percentage. Add the following text to Chapter 2:

2.3.7 Maximum reinforcement percentage. Special reinforced masonry shear walls having a shear span ratio, M/V_d, equal to or greater than 1.0 and having an axial load, P, greater than $0.05 f'_m A_n$ that are subjected to in-plane forces shall have a maximum reinforcement ratio, ρ_{max}, not greater than that computed as follows:

$$\rho_{max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'_m}\right)} \qquad \textbf{(Equation 21-3)}$$

The maximum reinforcement ratio does not apply in the out-of-plane direction.

5.3.5 DESIGN USING $n\rho j$ AND $2/jk$ VALUES

The tables provided in this handbook are based on commonly used values for E_m and n. The designer may, however, encounter materials with other values of E_m and n.

Therefore, a technique of design has been developed that is applicable to any material, modulus of elasticity, E_m, modular ratio, n, or stress value. It is called the Universal Elastic Flexural Design Technique in which values for $2/jk$ and $n\rho j$ are obtained and then values of $n\rho$, j, k and ρ are determined. Table ASD-34 provides the data to determine $n\rho$, $2/jk$, $n\rho j$, j and k.

Since the moment based on allowable flexural compressive masonry stress, F_b is:

$$M = bd^2\left(\frac{jk}{2}\right)F_b$$

A value for $2/jk$ can be found by rearranging the equation as follows:

$$\frac{2}{jk} = bd^2\frac{F_b}{M}$$

Similarly, since the moment based on the allowable tensile steel stress F_s, is:

$$M = bd^2 (\rho j) F_s$$

A value of $n\rho j$ can be found by multiplying both sides by n and solving for $n\rho j$:

$$n\rho j = \frac{nM}{bd^2 F_s}$$

With the values of $2/jk$ and $n\rho j$, $n\rho$ values can be obtained from Table ASD-34 and the required steel ratio is calculated using the actual modular ratio:

$$\rho = n\rho/n$$

The area of steel can then be determined:

$$A_s = \rho bd$$

Therefore for design, given the moment on the section, the effective depth, d, the width, b, the specified strength of the masonry, f'_m, the allowable stress of the steel, F_s, and calculating the modular ratio, n, the values, $2/jk$ and $n\rho j$ can be calculated and the required steel can be determined.

EXAMPLE 5-F Determination of Area of Steel.

Given an out-of-plane moment requirement of 2150 ft-lbs/ft, determine the reinforcing steel required for an 8 in. nominal CMU if f'_m = 2500 psi, f_s = 24,000 psi and d = 5.3 in.

Solution 5-F

$$F_b = \frac{1}{3} f'_m = \frac{1}{3}(2500) = 833 \text{ psi}$$

$$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{900 f'_m} = 12.9$$

Determine $2/jk$ and $n\rho j$ to find $n\rho$ from Table ASD-34. Use the maximum value to obtain the required steel ratio.

$$\frac{2}{jk} = bd^2 \frac{F_b}{M}$$

$$= (12)(5.3)^2 \left(\frac{833}{(2150)(12)} \right) = 10.883$$

= 10.883

From Table ASD-34 for $2/jk$ = 10.883:

$$n\rho = 0.024$$

$$n\rho j = \frac{nM}{bd^2 F_S}$$

$$= \frac{(12.9)(2150)(12)}{(12)(5.3)^2 (24{,}000)} = 0.0411$$

From Table ASD-34 for $n\rho j$ = 0.0411

$$n\rho = 0.045$$

Steel stress governs since $n\rho$ is larger.

$$\rho = \frac{n\rho}{n} = \frac{0.045}{12.9} = 0.0035$$

$A_s = \rho bd$ = 0.0035(12)(5.3) = 0.22 sq in./ft

Use #6 at 24 in. o.c.

For analysis, the physical properties and the moment are given or calculated and the stress in the masonry and steel can then be determined as:

$$f_b = \frac{M}{bd^2} \left(\frac{2}{jk} \right)$$

$$f_s = \frac{M}{bd^2} \left(\frac{1}{pj} \right)$$

Where $\rho = A_s/b_d$ and $n = E_s/E_m$ the values and $2/jk$ and j are easily obtained from Table ASD-34 based on the calculated $n\rho$ value.

5.3.6 PARTIALLY GROUTED WALLS

In order to reduce the weight of a wall and to minimize the amount of grout used, only cells containing reinforcing steel are grouted in partially grouted hollow unit walls. This reduces the cross-sectional area of the wall and consideration should be given to reduced vertical load capacity, reduced shear capacity parallel to the wall and flexural capacity for out of plane forces.

Walls grouted only at the cells containing reinforcing steel develop a rectangular or a tee stress block when they are subjected to lateral forces perpendicular to the wall. If the compression area or kd distance to the neutral axis is within the face shells, the wall would be analyzed as a rectangular section.

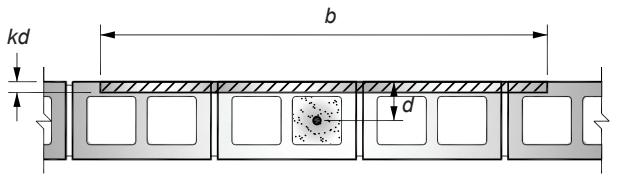


FIGURE 5.15 Partially grouted wall, rectangular stress block.

If the neutral axis, *kd*, is below the face shell the section would have a Tee section stress block.

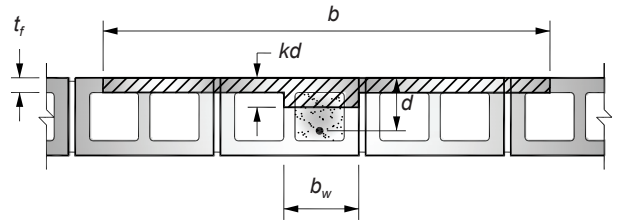


FIGURE 5.16 Partially grouted wall, Tee stress block.

For an 8 in. hollow unit wall the neutral axis will typically be within the face shell and the wall can be designed or analyzed as a rectangular section.

For larger units where the reinforcing steel is placed at a maximum *d* distance, a Tee section stress block may develop. The compression force, *C*, is resisted by both the face shell flange and part of the web.

Compression on flange

$$C_f = \frac{1}{2} f_b \left(1 + \frac{kd - t_f}{kd}\right) b t_f$$

Compression on web

$$C_w = \frac{1}{2} f_b \left(\frac{kd - t_f}{kd}\right) b_w (kd - t_f)$$

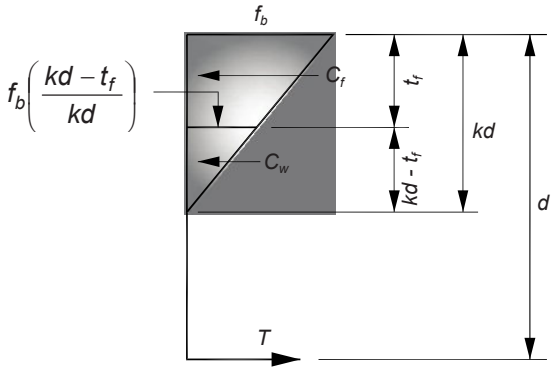


FIGURE 5.17 Stress diagram for Tee section.

Determination of the depth of the stress block, *kd*, is based on the modular ratio, *n*, steel ratio ρ, thickness of the face shell, t_f, and depth to the steel, *d*.

$$k = \frac{n\rho + \frac{1}{2}\left(\frac{t_f}{d}\right)^2}{n\rho + \left(\frac{t_f}{d}\right)}$$

The moment resistance for the Tee section becomes

$$M = C_f j_f d + C_w j_w d$$

The compression force on the web is usually small and generally can be ignored. The evaluation of the $j_f d$ value becomes complex and can be reasonably estimated by conservatively assuming the lever arm $jd = (d - t_f/2)$.

The value of the compression force can be determined by using the face shell area only and the average stress on it.

The compression force is

$$C = \frac{1}{2} f_b \left(1 + \frac{kd - t_f}{kd}\right) b t_f$$

The moment based on masonry stress is

$$M_m = \frac{1}{2} f_b \left(1 + \frac{kd - t_f}{kd}\right) b t_f \left(d - \frac{t_f}{2}\right)$$

The moment based on steel stress is

$$M_s = A_s f_s \rho b d f_s \left(d - \frac{t_f}{2}\right)$$

EXAMPLE 5-G Design of a Partially Grouted Wall.

Determine the reinforcing steel required for a nominal 10 in. CMU wall, 20 ft high and subjected to a lateral wind force of 20 psf. The wall is located in Seismic Design Category D.

Assume f'_m = 1500 psi, n = 21.48, f_s = 24,000 psi, $d = t/2$ = 4.81.

The wall is to be partially grouted at the vertical reinforcing steel bars spaced at 48 in. o.c.

Solution 5-G

$$\text{Moment} = \frac{wh^2}{8} \quad \text{(pinned each end)}$$

$$= \frac{(20)(20)^2}{8}(4)$$

Moment per bar = 4000 ft-lbs/bar

For estimating reinforcing steel (using the one-third stress increase for wind)

$$\frac{4}{3}K = \frac{M}{bd^2} = \frac{(4000)(12)}{(48)(4.81)^2} = 43.2$$

From Table ASD-24b for $\frac{4}{3}K_f = 43.2$

$\rho = 0.0014$ $A_s = \rho bd = 0.0014\ (48)(4.81)$

$= 0.323$ sq in.

Use 1 - #6 bar $A_s = 0.44$ sq in.

Check minimum area of steel for SDC D (MSJC Code 1.14.6.3)

Use $0.0007bt$ min. for horizontal (temperature and shrinkage) steel and $0.0013bt$ min. for vertical steel.

Minimum $A_s = 0.0013(48)(9.63)$

= 0.60 sq in., therefore, provide 1 - #7 bar for minimum A_s.

Determine location of neutral axis to see if it is outside the shell face.

$$\rho = \frac{A_s}{bd} = \frac{0.60}{(48)(4.81)} = 0.0026$$

$n\rho = 21.48(0.0026) = 0.056$

$$\frac{t_f}{d} = \frac{1.25}{4.81} = 0.26$$

$$k = \frac{n\rho + \frac{1}{2}\left(\frac{t_f}{d}\right)^2}{n\rho + \left(\frac{t_f}{d}\right)}$$

$$= \frac{0.056 + 05(0.26)^2}{0.056 + (0.26)}$$

= 0.284

$kd = 0.284(4.81) = 1.37 > t_f = 1.25$ in.

Therefore, the stress block is a Tee section.

Allowable masonry stress

$$F_b = \left(\frac{1}{3}\right)\left(\frac{4}{3}\right)(1500) = 667 \text{ psi}$$

Masonry stress

$$f_b = \frac{2M}{\left(1 + \frac{kd - t_f}{kd}\right)\left(d - \frac{t_f}{2}\right)bt_f}$$

$$= \frac{2(4000)(12)}{\left(1 + \frac{1.37 - 1.25}{1.37}\right)\left(4.81 - \frac{1.25}{2}\right)(48)(1.25)}$$

= 352 psi < 667 psi O.K.

Allowable steel stress

$$F_s = \left(\frac{4}{3}\right)(0.4)f_y = 32{,}000 \text{ psi}$$

Steel stress

$$f_s = \frac{M}{A_s\left(d - \frac{t_f}{2}\right)} = \frac{(4000)(12)}{0.60\left(4.81 - \frac{1.25}{2}\right)}$$

= 19,116 psi < 32,000 psi O.K.

Horizontal steel; use minimum A_s

$A_s = 0.0007bt$

= 0.0007(48)(9.63)

= 0.32 sq in. per 48 in.

Use #5 at 48 in. o.c.

Note: For walls that are taller or have a greater lateral load on them, two curtains of steel with a distance of 7.25 in. may be preferred.

5.3.7 COMPRESSION REINFORCEMENT

Masonry elements seldom require compression steel to obtain the required moment capacity since masonry sections are generally large and deep. However, in order not to overstress the masonry, in some cases compression steel may be beneficial. In walls and piers subjected to overturning moments, jamb steel at each end acts in both tension and compression and increases the moment capacity of the wall or pier. Of course, in column sections where

both compression and tension reinforcement exist there would be an advantage to consider the compression reinforcement in the traditional sense of a "doubly-reinforced" flexural capacity.

The use of compression reinforcement in masonry increases the moment capacity of the section by increasing the compression capacity of the masonry. It increases the moment arm distance, *jd*, producing an increase in flexural moment capacity.

5.3.7.1 COMPRESSION STEEL – MODULAR RATIO

Even though not currently required by the code, (although it was required by UBC) conventional practice is that the area of compression steel is multiplied by 2*n* to obtain the transformed area in flexural members reinforced with compression steel. This 2*n* is to account for creep in compression as stress is transferred from the surrounding masonry to compression reinforcement. This use of 2*n* will be shown in conjunction with Example 5-U. MSJC Code Section 1.8.6 provides for creep coefficients with the following values:

- Clay masonry creep coefficient, k_c = 0.7 x 10^{-7} per psi, and
- Concrete masonry creep coefficient, k_c = 2.5 x 10^{-7} per psi.

Thus, the higher creep coefficient would present evidence that the 2*n* transfer of stress is probably more appropriate for concrete masonry systems. However, since convention has used the 2*n* value, that value is retained for the examples. The stress in compression steel must not exceed the allowable tensile stress.

Based on the working stress, elastic design theory, strain between masonry and steel is assumed to be the same, therefore, the sharing of load between the masonry and compression steel would be in direct relation to modular ratio so that the stress in the steel would be as shown in calculations based upon using an "*n*" value.

As the stress strain curve for masonry is not linear and the strain increases in a non-linear fashion, strain in the steel is increased thus more load is taken by the steel than is initially calculated.

In addition, there is plastic flow and creep that takes place in masonry. The masonry is still capable of taking its share of the load but there is an increased strain in masonry and with this increase in strain, a similar strain is introduced into the steel subjecting the steel to a greater load or stress condition. Accordingly, the value for design and calculations of 2*n* is more in keeping with the actual stresses in the member with compression steel. This condition also utilizes, to a much more efficient degree, the use of steel by the introduction of the 2*n* value in keeping with the traditional ACI and concrete standards.

Tables ASD-73 to ASD-83 and Diagrams ASD-73 to ASD-83 are provided for the design and analysis of walls and beams using compression reinforcement.

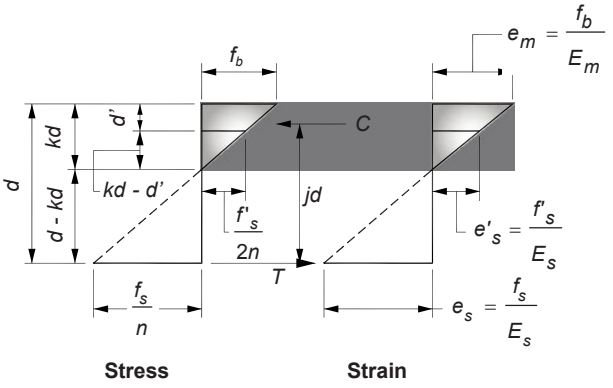


FIGURE 5.18 Stress and strain compatibility in flexural member with compression steel.

These Tables and Diagrams are based on a value for the transformed area of steel in compression which is doubled, i.e., transformed compression steel area = $2nA'_s = 2n\rho'bd$. In computing the location of the neutral axis, it is easier to maintain the compression area of masonry as *kdb* and to account for the area displaced by steel by $(2n - 1)A'_s$.

Maximum stress of compression steel at the maximum allowable masonry stress is calculated as follows:

$$\frac{f_b}{kd} = \frac{f'_s}{2n(kd - d')}$$

$$f'_s = 2nf_b\left(\frac{kd - d'}{kd}\right)$$

EXAMPLE 5-H Compression Steel Stress.

Determine the stress in the compression steel for a section with:

f'_m = 1500 psi

$F_b = \frac{1}{3} f'_m = 500$ psi

n = 21.48

d = 40 in.; d' = 4 in.

k = 0.30

Solution 5-H

$$f'_s = 2nf_b\left(\frac{kd - d'}{kd}\right)$$

$$= 2(21.48)(500)\left(\frac{(0.30)(40) - 4}{(0.30)(40)}\right)$$

= 14,320 psi

Stress in reinforcing steel is limited by the allowable stress in masonry and the d' distance. Although the compression steel is not stressed to its maximum allowable stress ($f'_s = 0.4f_y$ max. or 24,000 psi), it still improves the compression and moment capacity of the section.

Compression steel is effective only if d' is less than kd.

EXAMPLE 5-I Flexural Design, Tension and Compression Reinforcement.

A clay brick masonry beam is subjected to bending moment, M. Determine the reinforcing steel required:

(a) with tension steel, A_s, only

(b) with tension steel, A_s, and compression steel, A'_s.

Given:

Moment M = 55 ft-kips

f'_m = 1500 psi

f_y = 60,000 psi

Solution 5-I

From Table ASD-3 and ASD-4:

F_b = 500 psi

n = 27.6

F_s = 24,000 psi (tension steel)

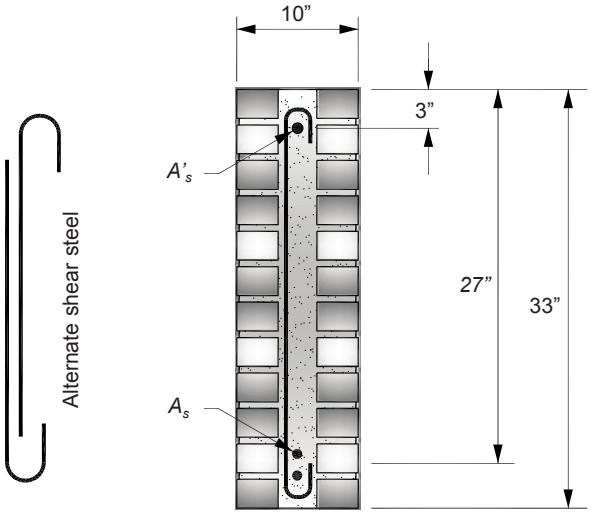


FIGURE 5.19 Beam with tension and compression steel.

F_{sc} = 24,000 psi (compression steel)

b = 10 in., d = 27 in., d' = 3 in.

Part (a) Tension steel, A_s, only

(1) Determine the flexural coefficient, K_f

$$K_f = \frac{M}{bd^2} = \frac{55 \text{ x } 12{,}000}{10 \text{ x } 27^2} = 90.5$$

This is greater than the balanced K_f, which is K_b = 80.2 as given in Table ASD-24a. Either

(a) over-reinforce the beam

(b) use compression steel, or

(c) increase the size of the member

(2) Determine the steel area required when reinforced for tension only.

From Table ASD-24a,

For K_f = 90.5: ρ = 0.0055

Area of steel $A_s = \rho bd$

= 0.0055 (10) (27)

= 1.49 sq in.

From Table GN-20a, selection of size and amount of steel.

Use 2 - #8 bars (A_s = 1.58 sq in.)

Part (b) Tension steel, A_s, and compression steel, A'_s

(3) Using Table ASD-74a, Coefficients for Tension and Compression Steel, or Diagram ASD-74a, ρ vs K_b.

$$\text{for } \frac{d'}{d} = \frac{3}{27} = 0.11 \text{ and } K_b = 90.5$$

read tension steel ratio $\rho = 0.0043$

compression steel ratio $\rho' = 0.0003$

Tension steel $A_s = \rho bd = 0.0043(10)(27)$

$= 1.16$ sq in.

Use 2 - #7 bars ($A_s = 1.20$ sq in.)

Compression steel $A'_s = \rho' bd$

$= (0.0003)(10)(27)$

$= 0.08$ sq in.

Use 1 - #3 bar ($A'_s = 0.11$ sq in.)

Total area of steel:

1.20 + 0.11 = 1.31 sq in.

5.4 SHEAR

5.4.1 GENERAL

Structural elements such as walls, piers and beams are subjected to shear forces as well as flexural stresses. The unit shear stress is computed based on the formula:

$$f_v = \frac{V}{bjd} = \frac{V}{bd} \text{ or } \frac{V}{bl}$$

Deletion of the *j* coefficient is usually not significant as the actual shear stress distribution is not fully understood and the refinement of *jd* is unwarranted. In fact, the *j* coefficient is not included in the calculation of the shear stress for concrete or in MSJC Code Equation 2-19. Therefore, *j* is now ignored for shear design in reinforced masonry design.

Shear design analysis and criteria have been based on tests and experience and the limiting allowable stresses are conservative. The MSJC Code provides for the shear provisions:

MSJC Code Section 2.3.5

2.3.5 *Shear*

2.3.5.1 Members that are not subjected to flexural tension shall be designed in accordance with the requirements of Section 2.2.5 or shall be designed in accordance with the following:

2.3.5.1.1 Reinforcement shall be provided in accordance with the requirements of Section 2.3.5.3.

2.3.5.1.2 The calculated shear stress, f_v, shall not exceed F_v, where F_v is determined in accordance with Section 2.3.5.2.3.

2.3.5.2 Members subjected to flexural tension shall be reinforced to resist the tension and shall be designed in accordance with the following:

2.3.5.2.1 Calculated shear stress in the masonry shall be determined by the relationship:

$$f_v = \frac{V}{bd} \quad (2\text{-}19)$$

2.3.5.2.2 Where reinforcement is not provided to resist all of the calculated shear, f_v shall not exceed F_v, where:

(a) for flexural members

$$F_v = \sqrt{f'_m} \quad (2\text{-}20)$$

but shall not exceed 50 psi (345 kPa).

(b) for shear walls,

where, $M/Vd < 1$,

$$F_v = \left(\tfrac{1}{3}\right)\left[4 - \left(M/Vd\right)\right]\sqrt{f'_m} \quad (2\text{-}21)$$

but shall not exceed 80 - 45(M/Vd) psi

where, $M/Vd \geq 1$,

$$F_v = \sqrt{f'_m} \quad (2\text{-}22)$$

but shall not exceed 35 psi (241 kPa).

2.3.5.2.3 Where shear reinforcement is provided in accordance with Section 2.3.5.3 to resist all of the calculated shear, f_v shall not exceed F_v, where:

(a) for flexural members:

$$F_v = 3.0\sqrt{f'_m} \quad (2\text{-}23)$$

but shall not exceed 150 psi (1034 kPa).

(b) for shear walls:

where, $M/Vd < 1$,

$$F_v = \left(\tfrac{1}{2}\right)\left[4 - \left(M/Vd\right)\right]\sqrt{f'_m} \quad (2\text{-}24)$$

but shall not exceed 120 - 45(M/Vd) psi

where $M/Vd \geq 1$,

$$F_v = 1.5\sqrt{f'_m} \quad (2\text{-}25)$$

but shall not exceed 75 psi (517 kPa).

2.3.5.2.4 The ratio *M/Vd* shall always be taken as a positive number.

2.3.5.3 Minimum area of shear reinforcement required by Section 2.3.5.1 or 2.3.5.2.3 shall be determined by the following:

$$A_v = \frac{Vs}{F_s d} \qquad (2\text{-}26)$$

2.3.5.3.1 Shear reinforcement shall be provided parallel to the direction of applied shear force. Spacing of shear reinforcement shall not exceed the lesser of *d*/2 or 48 in. (1219 mm).

2.3.5.3.2 Reinforcement shall be provided perpendicular to the shear reinforcement and shall be at least equal to one-third A_v. The reinforcement shall be uniformly distributed and shall not exceed a spacing of 8 ft (2.44 m).

2.3.5.4 In composite masonry walls, shear stresses developed in the planes of interfaces between wythes and filled collar joints or between wythes and headers shall meet the requirements of Section 2.1.5.2.2.

2.3.5.5 In cantilever beams, the maximum shear shall be used. In noncantilever beams, the maximum shear shall be used except that sections located within a distance *d*/2 from the face of support shall be designed for the same shear as that computed at a distance *d*/2 from the face of support when the following conditions are met:

(a) support reaction, in direction of applied shear force, introduces compression into the end regions of member, and

(b) no concentrated load occurs between face of support and a distance *d*/2 from face.

If the unit shear stress does not exceed the allowable shear stress for masonry as listed in Tables ASD-3 and ASD-5, no shear reinforcement is required. If the unit shear exceeds the listed allowable shear stress for masonry, shear reinforcing steel must be provided to resist all the shear forces. Tables and Diagrams ASD-6 and ASD-54 through ASD-66 can be used to size the shear reinforcing steel.

If the unit shear stress exceeds the maximum allowable shear stress for the reinforcing steel, the section must be increased in size and/or higher strength masonry must be specified.

5.4.2 BEAM SHEAR

When masonry flexural members are designed to resist shear forces without the use of reinforcing steel, the calculated shear stress may not exceed $1.0\sqrt{f'_m}$ nor 50 psi. Should the unit shear stress exceed the allowable masonry shear stress, all shear stress must be resisted by reinforcing steel. For flexural members with reinforcing steel resisting all the shear force, the maximum allowable shear stress is $3.0\sqrt{f'_m}$ with 150 psi as a maximum.

The principle of shear reinforcement is to provide steel to resist the diagonal tension stresses developed in a member. Figures 5.20 and 5.21 demonstrate the diagonal tension principle.

Diagonal tension stresses are due to the combined vertical and horizontal shear, and although reinforcing steel in either direction will resist the diagonal tension stresses, the shear reinforcement should be parallel to the direction of external applied loads or shear forces. Therefore, shear reinforcement is vertical in a beam and horizontal in a wall.

Steel resists the shear by tension and it must be anchored in the compression zone of the beam or the wall.

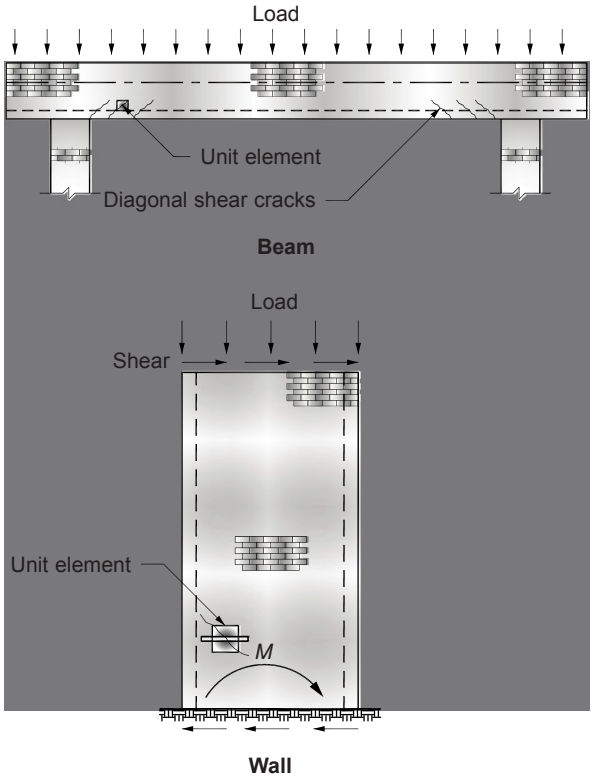


FIGURE 5.20 Diagonal tension cracks in a flexural member.

Unit shear, f_v, is used to determine the shear steel spacing based on the formula:

$$\text{Spacing, } s = \frac{A_v F_s}{f_v b}$$

Diagrams ASD-54 through ASD-66 can be used to quickly find the required shear reinforcement size and spacing. Likewise Tables ASD-54 through ASD-66 give the allowable shear stress capacity, F_v, which can be found for a given size and spacing of steel.

$$F_v = \frac{A_v F_s}{bs}$$

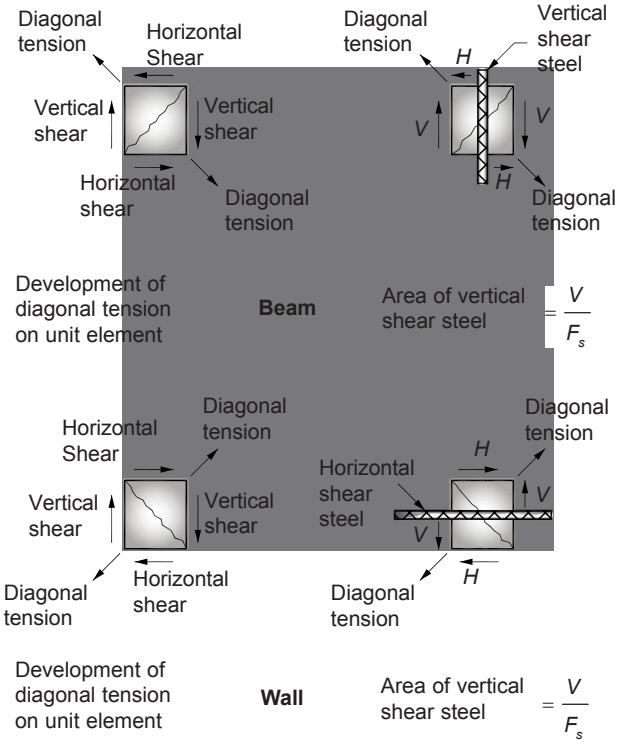


FIGURE 5.21 Development of shear in unit element.

For continuous or fixed beams, the value used to determine the shear steel spacing may be taken at a distance *d* from the face of the support. The maximum spacing of shear steel may not exceed *d*/2 or 48 inches. The first shear reinforcing bar should be located at half the calculated spacing but no more than *d*/4 from the face of support.

The thickness of a member or wall for shear calculations may be influenced by the treatment of the joints. Masonry with flush or concave tooled joints would have the total thickness effective. However, if joints are raked, consideration should be given to the reduction in the width of the wall caused by raking.

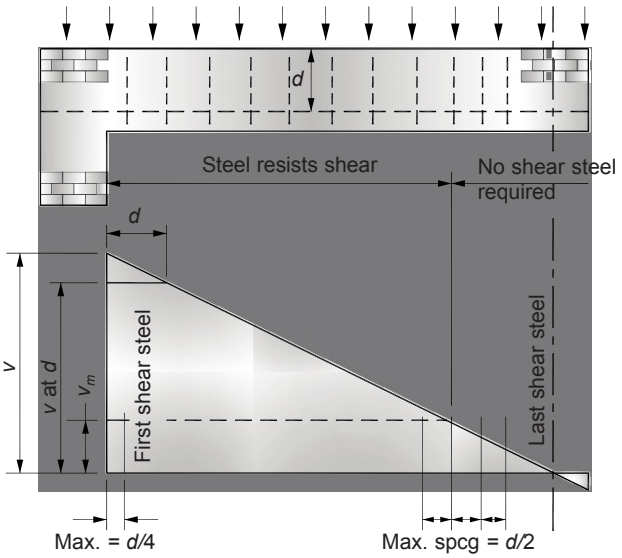


FIGURE 5.22 Spacing of shear reinforcement in a beam.

EXAMPLE 5-J Flexural Design – Unit Shear Stress.

Determine the unit shear stress for the following continuous masonry beam:

DL = 150 plf
LL = 400 plf
Span = 14 ft (continuous span)
d = 20 in.
b = 9 in.
f'_m = 1500 psi

Solution 5-J

(1) Total load = 400 + 150 = 550 lb/ft

(2) Total shear $V = \frac{1}{2}(550)(14) = 3850$ lbs

The MSJC Code does not stipulate computing the shear at a distance *d* from the support for those members that are not reinforced for shear, but does stipulate a distance of *d*/2 for those reinforced for shear. Thus, for this part, no reduction in shear is made.

(3) Calculate the shear stress:

$$\frac{V}{bd} = \frac{3850}{(9)(20)} = 21.4 \text{ psi}$$

From Table ASD-3, the allowable flexural shear stress with no shear reinforcement is:

$$F'_v = \sqrt{1500} = 39 \text{ psi} > 21.4 \text{ psi}$$

No shear reinforcement is required.

EXAMPLE 5-K Beam – Shear Reinforcement.

A concrete masonry spandrel beam is subjected to a shear force

V = 13 kips

Design the shear reinforcement for the simply supported beam if:

Nominal b = 8 in., Actual b = 7.625 in., d = 36 in.

F_s = 24,000 psi, f'_m = 1500 psi

Solution 5-K

(1) From Table ASD-3, the allowable flexural shear stress with shear reinforcement is

$$F_v = 3\sqrt{1500} = 116.2 \text{ psi;}$$

and the maximum without shear reinforcement is

$$F_v = \sqrt{1500} = 38.7 \text{ psi}$$

$$\text{Shear stress, } f_v = \frac{V}{bd} = \frac{13{,}000}{(7.625)(36)}$$

= 47.4 psi < 116.2 psi,

but > 38.7 psi; therefore, must be reinforced.

(2) From Diagram ASD-58, spacing of shear steel for b = 7.625 in. and f_v = 47.4 psi

#5 at 20 in. o.c. (Shear capacity, F_v = 49 psi) satisfies requirement, however, masonry uses 8 in. modules. Use #5 @ 16 in. o.c. Spacing at 16 in. o.c. also keeps spacing of reinforcing steel less than d/2.

EXAMPLE 5-L Beam Shear Reinforcing Size and Spacing.

Determine the shear reinforcement required in the 8 in. solid grouted concrete masonry beam shown in Figure 5.23.

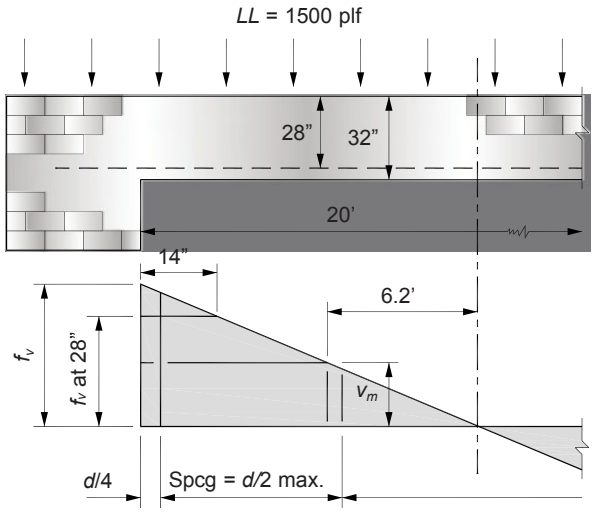


FIGURE 5.23 Shear reinforcement in beam.

Design data:

f'_m = 2500 psi; f_y = 60,000 psi; d = 28 in.

Solution 5-L

(1) Conservatively, assume the beam is constructed of normal weight concrete block and grout. Thus, from Table GN-3a, the weight of solid grouted hollow concrete block = 84 psf

$$DL = \frac{(84)(32)}{12} = 224 \text{ plf}$$

LL = 1500 plf

$TL = w$ = 1724 plf

$$\text{Total shear } V = \frac{wl}{2} = \frac{(1724)(20)}{2}$$

= 17,240 lbs

(2) Calculate the shear stress. For 8 in. concrete masonry units, b = 7.63 in.

$$f_v = \frac{V}{bd} = \frac{17{,}240}{(7.63)(28)}$$

= 80.7 psi

(3) Check the capacity of the masonry without shear reinforcement.

From Table ASD-3 for 2500 psi masonry;

F_v = 50 psi < f_v of 80.7 psi; therefore, beam must have shear reinforcement

(4) Find where shear reinforcement is required.

$$V = F_v b d = 50(7.63)(28)$$
$$= 10{,}682 \text{ lbs}$$

Distance from center of beam where no shear reinforcement is required.

$$\text{Distance } S = \frac{V}{w} = \frac{10{,}682}{1724} = 6.2 \text{ ft}$$

(5) Calculate the shear at a distance *d*/2 from the support and determine the size and spacing of the reinforcing steel.

$$V = 17{,}240 - 1724\,\frac{28}{(12)(2)}$$

= 15,229 lbs

$$\text{Unit shear, } f_v = \frac{V}{bd} = \frac{15{,}229}{7.63(28)}$$

= 71.3 psi

From Diagram ASD-58 for b = 7.63 in. and f_v = 71.3 psi, try either #4 @ 8 in. or #6 @ 16 in.

Maximum spacing of shear reinforcement is limited to $d/2$ = 28/2 = 14 in.

Therefore use #4 @ 8 in.

Place the first bar at $s/2$ = 8/2 = 4 in.

Continue the reinforcement past the point where shear reinforcement is no longer required.

$$\text{Number of spaces} = \frac{[(10)(12) - (6.2)(12)] - 4}{8} = 5.2$$

Say 6 spaces or 7 bar locations

Use at least 6 spaces @ 8 in. = 4 ft - 0 in.

5.4.3 SHEAR PARALLEL TO WALL

Walls which resist lateral forces, particularly forces due to wind or earthquake, are called shear walls. These walls may be load bearing or non-load bearing. Shear walls may also resist lateral forces due to earth or water.

The allowable shear stress for walls, based on *M/Vd* is given in MSJC Code Section 2.3.5.2 and Tables ASD-5 and ASD-6 of this book.

When *M/Vd* is less than one, the maximum allowable shear stress in the masonry is determined by the equation:

$$F_v = \frac{1}{3}\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m} \quad \text{(MSJC Code Eq 2-21)}$$

with a maximum value of

$$F_{v(\max)} = \left(80 - 45\,\frac{M}{Vd}\right)(\text{psi})$$

When *M/Vd* is one or greater, the maximum allowable masonry shear stress is:

$$F_v = 1.0\sqrt{f'_m};\ 35 \text{ psi maximum} \quad \text{(MSJC Code Eq 2-22)}$$

When the shear stress, f_v, exceeds the allowable masonry shear stress given above, reinforcing steel must be provided to resist all the shear.

The allowable shear stress for reinforced walls when *M/Vd* is less than one:

$$F_v = \frac{1}{2}\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m} \quad \text{(MSJC Code Eq 2-24)}$$

with a maximum value of:

$$F_{v(\max)} = \left(120 - 45\,\frac{M}{Vd}\right)(\text{psi})$$

When *M/Vd* is one or greater, the maximum allowable shear stress is:

$$F_v = 1.5\sqrt{f'_m};\ 75 \text{ psi maximum} \quad \text{(MSJC Code Eq 2-25)}$$

The reduction in allowable shear stress based on the *M/Vd* ratio is related to the decreased shear capability from a pure shear condition, i.e., *M/Vd* = 0; to a flexural shear condition in which the wall element is acting as a flexural beam element as well as a shear resisting wall.

Allowable stresses may be increased by one third when the lateral force is due to wind or seismic loads, as explained in ASCE 7-05, C2.4.1.

MSJC Code Section 2.1.2.3

2.1.2.3 The allowable stresses and allowable loads in Chapters 2 and 4 shall be permitted to be increased by one-third when considering Load Combination (c), (d), or (e) of Section 2.1.2.1, and as permitted by the legally adopted building code.

(c) $D + L +$ (W or E)

(d) $D + W$

(e) $0.9\,D + E$

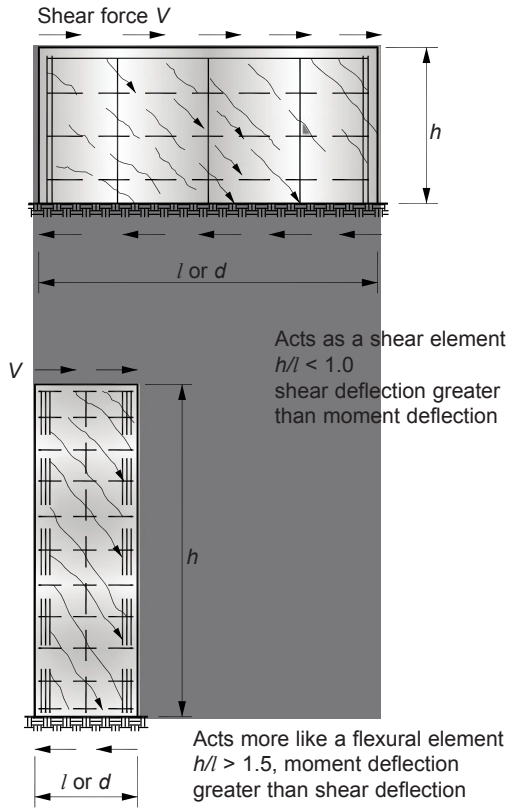


FIGURE 5.24 Shear walls.

The requirement that reinforcing steel be designed to resist all shear is conservative since masonry has some shear capacity, which is ignored. IBC Section 2106 provides seismic requirements for masonry in addition to the requirements of MSJC Code Section 1.14. Shear wall types for lateral force-resisting systems are designated by the following names:

- Ordinary plain (unreinforced) masonry shear walls,
- Detailed plain (unreinforced) masonry shear walls,
- Ordinary reinforced masonry shear walls,
- Intermediate reinforced masonry shear walls, and
- Special reinforced masonry shear walls

IBC Code Section 2106

2106.1 Seismic design requirements for masonry. Masonry structures and components shall comply with the requirements in Section 1.14.2.2 and Section 1.14.3, 1.14.4, 1.14.5, 1.14.6 or 1.14.7 of ACI 530/ASCE 5/TMS 402 depending on the structure's seismic design category as determined in Section 1613. All masonry walls, unless isolated on three edges from in-plane motion of the basic structural systems, shall be considered to be part of the seismic-force-resisting system. In addition, the following requirements shall be met.

2106.2 Anchorage of masonry walls. Masonry walls shall be anchored to the roof and floors that provide lateral support for the wall in accordance with Section 1604.8.2.

2106.3 Seismic Design Category B. Structures assigned to Seismic Design Category B shall conform to the requirements of Section 1.14.4 of ACI 530/ASCE 5/TMS 402 and to the additional requirements of this section.

> **2106.3.1 Masonry walls not part of the lateral-force-resisting system.** Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that the vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.

2106.4 Additional requirements for structures in Seismic Design Category C. Structures assigned to Seismic Design Category C shall conform to the requirements of Section 2106.3, Section 1.14.5 of ACI 530/ASCE 5/TMS 402 and the additional requirements of this section.

> **2106.4.1 Design of discontinuous members that are part of the lateral-force-resisting system.** Columns and pilasters that are part of the lateral-force-resisting system and that support reactions from discontinuous stiff members such as walls shall be provided with transverse reinforcement spaced at no more than one-fourth of the least nominal dimension of the column or pilaster. The minimum transverse reinforcement ratio shall be 0.0015. Beams supporting reactions from discontinuous walls or frames shall be provided with transverse reinforcement spaced at no more than one-half of the nominal depth of the beam. The minimum transverse reinforcement ratio shall be 0.0015.

2106.5 Additional requirements for structures in Seismic Design Category D. Structures assigned to Seismic Design Category D shall conform to the requirements of Section 2106.4, Section 1.14.6 of ACI 530/ASCE 5/TMS 402 and the additional requirements of this section.

> **2106.5.1 Loads for shear walls designed by the working stress design method.** When calculating in-plane shear or diagonal tension stresses by the working stress design method, shear walls that resist seismic forces shall be designed to resist 1.5 times the seismic forces required by Chapter 16. The 1.5 multiplier need not be applied to the overturning moment.

2106.5.2 Shear wall shear strength. For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.

For all cross sections within a region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall, the nominal shear strength shall be determined by Equation 21-1.

$$V_n = A_n \rho_n f_y \qquad \textbf{(Equation 21-1)}$$

The required shear strength for this region shall be calculated at a distance $L_w/2$ above the base of the shear wall, but not to exceed one-half story height.

For the other region, the nominal shear strength of the shear wall shall be determined from Section 2108.

2106.6 Additional requirements for structures in Seismic Design Category E or F. Structures assigned to Seismic Design Category E or F shall conform to the requirements of Section 2106.5 and Section 1.14.7 of ACI 530/ASCE 5/TMS 402.

Shear wall types are the same in the IBC and the MSJC Code. Prescriptive reinforcement requirements for these walls is given in the MSJC Code Section 1.14.2 and the additional seismic prescriptive requirements for each associated Seismic Design Category are given in MSJC Code Sections 1.14.3 through 1.14.7.

Note that the requirements have been divided not only by Seismic Design Categories, but also by the shear wall types. Shear wall types listed below are from the lowest ductility and least detailing requirements to those with the highest ductility and most stringent detailing requirements.

The Ordinary Plain (Unreinforced) Masonry Shear Walls are designed in accordance with the unreinforced masonry requirements. This type of wall is listed for information only and does not merit design discussion in this handbook.

The Detailed Plain (Unreinforced) Masonry Shear Walls are designed in accordance with MSJC Code Section 2.2 or Section 3.2 and must comply with Sections 1.14.2.2.2.1 and 1.14.2.2.2.2. Also an unreinforced wall listed for information only.

MSJC Code Section 1.14.2.2.2.1

1.14.2.2.2.1 *Minimum reinforcement requirements* — Vertical reinforcement of at least 0.2 in.^{2} (129 mm^2) in cross-sectional area shall be provided at corners, within 16 in. (406 mm) of each side of openings, within 8 in. (203 mm) of each side of movement joints, within 8 in. (203 mm) of the ends of walls, and at a maximum spacing of 120 in. (3048 mm) on center.

Reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm) in either the horizontal or vertical direction, unless the spacing of distributed reinforcement is interrupted by such openings.

Horizontal joint reinforcement shall consist of at least two wires of W1.7 (MW11) spaced not more than 16 in. (406 mm) on center, or bond beam reinforcement shall be provided of at least 0.2 in.^{2} (129 mm^2) in cross-sectional area spaced not more than 120 in. (3048 mm) on center. Horizontal reinforcement shall also be provided at the bottom and top of wall openings and shall extend not less than 24 in. (610 mm) nor less than 40 bar diameters past the opening, continuously at structurally connected roof and floor levels, and within 16 in. (406 mm) of the top of walls.

1.14.2.2.2.2 *Connections* — Connectors shall be provided to transfer forces between masonry walls and horizontal elements in accordance with the requirements of Section 2.1.8. Connectors shall be designed to transfer horizontal design forces acting either perpendicular or parallel to the wall, but not less than 200 lb per lineal ft (2919 N per lineal m) of wall. The maximum spacing between connectors shall be 4 ft (1.22 m).

Ordinary Reinforced Masonry Shear Walls must comply with reinforced masonry requirements given in MSJC Code Section 2.3 for ASD or 3.3 for SD, and Sections 1.14.2.2.2.1 and 1.14.2.2.2.2.

Intermediate Reinforced Masonry Shear Walls follow the same prescriptive requirements as Ordinary Reinforced Masonry Shear Walls with 48 in. maximum spacing of vertical reinforcement.

Special Reinforced Masonry Shear Walls must comply with the provisions of MSJC Code Section 2.3 or Section 3.3. Design must also comply with the requirements of Sections 1.14.2.2.2.1, 1.14.2.2.2.2, 1.14.6.3, and the following:

(a) The maximum spacing of vertical and horizontal reinforcement shall be the smaller of one-third the length of the shear wall, one-third the height of the shear wall, or 48 in.

(b) The minimum cross-sectional area of vertical reinforcement shall be one-third of the required shear reinforcement.

(c) Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.

Next, the shear wall categories above must comply with the various Seismic Design Categories (SDC) as given in ASCE 7-02 (or IBC Section 1613).

These are summarized below for each SDC. These seismic provisions apply to masonry shear walls as well as other elements of masonry systems to resist lateral loads due to earthquakes.

SDC A:

Masonry structures located in Seismic Design Category A shall comply with the drift limits and anchorage requirements as shown in MSJC Code Sections 1.14.3.2 and 1.14.3.3:

MSJC Code Section 1.14.3.2

1.14.3.2 *Drift limits* — The calculated story drift of masonry structures due to the combination of design seismic forces and gravity loads shall not exceed 0.007 times the story height.

1.14.3.3 *Anchorage of masonry walls* — Masonry walls shall be anchored to the roof and all floors that provide lateral support for the walls. The anchorage shall provide a direct connection between the walls and the floor or roof construction. The connections shall be capable of resisting the greater of a seismic lateral force induced by the wall or 1000 times the effective peak velocity-related acceleration, lb per lineal ft of wall (14,590 times, N/m).

SDC B:

Masonry structures located in Seismic Design Category B must comply with the provisions of SDC A with additional requirements as shown below for MSJC Code Section 1.14.4.2:

MSJC Code Section 1.14.4.2

1.14.4.2 *Design of elements that are part of the lateral force-resisting system* — The lateral force-resisting system shall be designed to comply with the requirements of Chapter 2, 3, or 4. Masonry shear walls shall comply with the requirements of ordinary plain (unreinforced) masonry shear walls, detailed plain (unreinforced) masonry shear walls, ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, or special reinforced masonry shear walls.

SDC C:

Masonry structures located in Seismic Design Category C must comply with the requirements of SDC A and B, in addition to the requirements of MSJC Code Sections 1.14.5.2. and 1.14.5.3. The design for SDC C is divided into elements that are and are not part of the lateral force-resisting system. The following requirements apply to elements that are not part of the lateral force-resisting system:

MSJC Code Section 1.14.5.2

1.14.5.2.1 Load-bearing frames or columns that are not part of the lateral force-resisting system shall be analyzed as to their effect on the response of the system. Such frames or columns shall be adequate for vertical load carrying capacity and induced moment due to the design story drift.

1.14.5.2.2 Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.

1.14.5.2.3 *Reinforcement requirements* — Masonry elements listed in Section 1.14.5.2.2, except AAC masonry elements, shall be reinforced in either the horizontal or vertical direction in accordance with the following:

(a) *Horizontal reinforcement* — Horizontal joint reinforcement shall consist of at least two longitudinal W1.7 (MW11) wires spaced not more than 16 in. (406 mm) for walls greater than 4 in. (102 mm) in width and at least one longitudinal W1.7 (MW11) wire spaced not more 16 in. (406 mm) for walls not exceeding 4 in. (102 mm) in width; or at least one No. 4 (M #13) bar spaced not more than 48 in. (1219 mm). Where two longitudinal wires of joint reinforcement are used, the space between these wires shall be the widest that the mortar joint will accommodate. Horizontal reinforcement shall be provided within 16 in. (406 mm) of the top and bottom of these masonry walls.

(b) *Vertical reinforcement* — Vertical reinforcement shall consist of at least one No. 4 (M #13) bar spaced not more than 120 in. (3048 mm) for Seismic Design Category C and not more than 48 in. (1219 mm) for Seismic Design Category D, E, and F. Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.

The design of elements that are a part of the lateral force-resisting system are designed according to MSJC Code Section 1.14.5.3 as follows:

MSJC Code Section 1.14.5.3

1.14.5.3.1 *Connections to masonry columns* — Connectors shall be provided to transfer forces between masonry columns and horizontal elements in accordance with the requirements of Section 2.1.8. Where anchor bolts are used to connect horizontal elements to the tops of columns, anchor bolts shall be placed within lateral ties. Lateral ties shall enclose both the vertical bars in the column and the anchor bolts. There shall be a minimum of two No. 4 (M #13) lateral ties provided in the top 5 in. (127 mm) of the column.

1.14.5.3.2 *Masonry shear walls* — Masonry shear walls shall comply with the requirements for ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, or special reinforced masonry shear walls.

SDC D:

Masonry structures located in Seismic Design Category D must comply with the requirements of SDC A, B, and C, in addition to the requirements of MSJC Code Section 1.14.6:

MSJC Code Section 1.14.6

1.14.6.2 *Design requirements* — Masonry elements, other than those covered by Section 1.14.5.2.2, shall be designed in accordance with the requirements of Sections 2.1 and 2.3, Chapter 3, Chapter 4 or Appendix A.

1.14.6.3 *Minimum reinforcement requirements for masonry walls* — Masonry walls other than those covered by Section 1.14.5.2.2, and other than AAC masonry, shall be reinforced in both the vertical and horizontal direction. The sum of the cross-sectional area of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall, and the minimum cross-sectional area in each direction shall be not less than 0.0007 times the gross cross-sectional area of the wall, using specified dimensions. Reinforcement shall be uniformly distributed. The maximum spacing of reinforcement shall be 48 in. (1219 mm), except for stack bond masonry. Wythes of stack bond masonry shall be constructed of fully grouted hollow open-end units, fully grouted hollow units laid with full head joints, or solid units. Maximum spacing of reinforcement for walls with stack bond masonry shall be 24 in. (610 mm).

1.14.6.4 *Masonry shear walls* — Masonry shear walls shall comply with the requirements for special reinforced masonry shear walls.

1.14.6.5 *Minimum reinforcement for masonry columns* — Lateral ties in masonry columns shall be spaced not more than 8 in. (203 mm) on center and shall be at least 3/8 in. (9.5 mm) diameter. Lateral ties shall be embedded in grout.

1.14.6.6 *Material requirements* — Neither Type N mortar nor masonry cement shall be used as part of the lateral force-resisting system.

1.14.6.7 *Lateral tie anchorage* — Standard hooks for lateral tie anchorage shall be either a 135-degree standard hook or a 180-degree standard hook.

SDC E and F:

Masonry structures located in Seismic Design Categories E or F must be designed by the requirements of SDC A, B, C, D, and the requirements given in MSJC Code Section 1.14.7:

MSJC Code Section 1.14.7

1.14.7.2 *Minimum reinforcement for stack bond elements that are not part of the lateral force-resisting system* — Stack bond masonry that is not part of the lateral force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0015 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 24 in. (610 mm). These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

1.14.7.3 *Minimum reinforcement for stack bond elements that are part of the lateral force-resisting system* — Stack bond masonry that is part of the lateral force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0025 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 16 in. (406 mm). These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

Additionally, 2006 IBC Section 2106.5.1 provides for a 1.5 multiplier on seismic forces applied to shear walls. Also, other adjustments in the requirements for seismic design applied to the masonry shear walls and elements are contained in IBC Section 2106:

EXAMPLE 5-M Shear Design, Wall Pier.

Design the horizontal shear reinforcement in a clay masonry pier for a lateral seismic force, V, for 19.2 kips if:

f'_m = 1500 psi; f_y = 60,000 psi;

w = 48 in.; d = 42 in.; t = 10 in.

Solution 5-M

(1) Calculate the actual shear stress

Use IBC Section 2106.5.1. Increase the design shear force by 1.5 times the applied force.

$$f_v = \frac{1.5V}{bd} = \frac{1.5(19{,}200)}{10(42)} = 69 \text{ psi}$$

(2) Find the allowable shear stress by calculating

$$\frac{M}{Vd}$$

For a fixed pier subjected to a deflection, Δ:

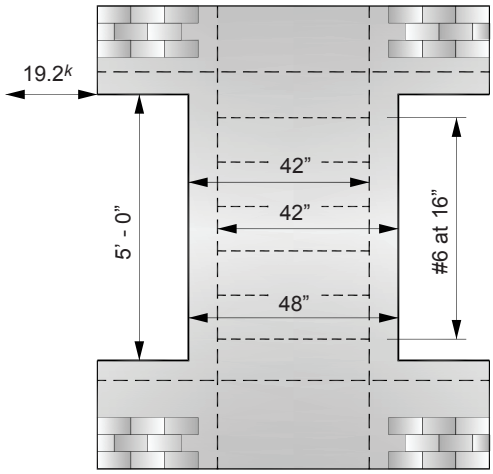


FIGURE 5.25 Pier with shear reinforcement.

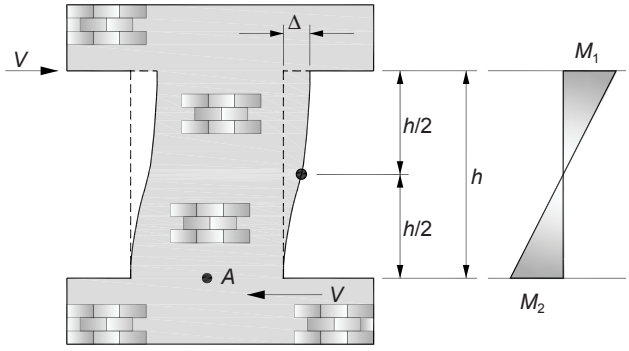


FIGURE 5.26 Fixed pier subjected to displacement, Δ.

$$\Sigma M_A = 0$$

$$0 = M_1 + M_2 - Vh$$

$$M = \frac{Vh}{2}$$

$$\frac{M}{Vd} = \frac{Vh/2}{Vd} = \frac{h}{2d}$$

Therefore for this pier,

$$\frac{M}{Vd} = \frac{h}{2d} = \frac{5 \text{ x } 12 \text{ in./ft}}{2(42)}$$

$$= 0.71$$

From Table ASD-5 for M/Vd = 0.71 and f'_m = 1500 psi, the allowable shear stress for the masonry is:

$$F_v = \frac{4}{3}(43 \text{ psi}) = 57 \text{ psi} < 69 \text{ psi} \qquad \text{N.G.}$$

However, note that if the one-third stress increase were not allowed then the wall must be reinforced by a larger amount.

Reinforcing steel must be provided and designed to carry all the shear load. From Table ASD-6,

$$F_v = \frac{4}{3}(64 \text{ psi}) = 85 \text{ psi} > 69 \text{ psi} \qquad \text{O.K.}$$

Size the shear steel from Diagram ASD-60 for F_s = 32,000 psi, t = 10 in., f'_m = 1500 psi and f_v = 69 psi.

#6 bars spaced vertically @ 20 in. o.c. satisfies the requirement, however, masonry steel should be spaced at 8 in. modules. Space #6 bars at 16 in. o.c.

EXAMPLE 5-N Shear Design, Wall Pier.

Determine the reinforcement for an interior shear wall with limited surcharge loading, but wind loading at each of the floors as indicated in the diagram. The absence of a surcharge load is often a more critical condition, since surcharge adds to the shear resistance of a structure. The wind loads include the omega factor of 1.3 from 2006 IBC. Determine the different options for the shear reinforcement. Include the flexural reinforcement and both the vertical and horizontal shear reinforcement. Use 6 in. concrete masonry with face-shell bedding for a four-story building wall as shown below (with each story height of 10 ft):

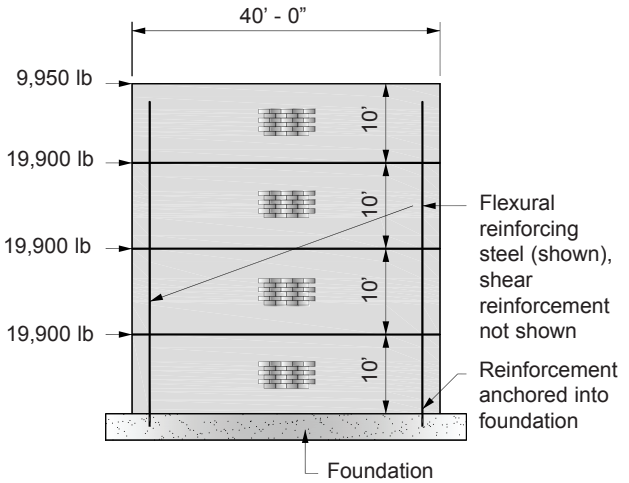


FIGURE 5.27 Pier with flexural reinforcement.

Material properties:

f'_m = 1500 psi

face shell thickness = 1 in.

wall weight = 30 lb/ft^2

Solution 5-N

Assume that two bars will be placed in each end for the flexural reinforcement by grouting the end units and with one bar placed in each grouted cell. Thus,

d = 40(12) - 8 = 472 in. or 39.33 ft

Flexural A_s is calculated:

Overturning moment about the base is:

M = (19,900)(10 + 20 + 30) + 9,950(40)

= 1,592,000 ft-lbs = 19,104,000 in.-lbs

Assume that these lateral loads are from wind, and that the one-third stress increase applies. Assume j = 0.9, which is a good estimate for searching for the neutral axis when the surcharge is small, and refine, if needed – this refinement is verified after the reinforcement selection, below.

$$A_s = \frac{19{,}104{,}000}{0.9(39.33)(12)(1.33)(24{,}000)}$$

= 1.41 in.^{2}

Using 2 - #8 bars at each end of the wall, by grouting the end units and placing one bar in each grouted cell, provides 1.58 in.^{2} > 1.41 in.^{2} OK

Note that the refined calculated j for this problem shows j = 0.92, which is close enough and results in the same steel selection.

Shear

The actual masonry shear stress (using b = 2(1) for two face shells)

$$f_v = \frac{69{,}650}{2(39.33)(12)} = 74 \text{ psi}$$

Determine whether the masonry alone can be counted on to resist the shear forces:

V = 19,900(3) + 9950 = 69,650 lbs

$$\frac{M}{Vd} = \frac{19{,}104{,}000}{(69{,}650)(472)} = 0.58 < 1.0$$

(MSJC Equation 2-21 applies)

$$F_v = \frac{1}{3}\left[4 - \left(\frac{M}{Vd}\right)\right]\sqrt{f'_m} = \frac{1}{3}[4 - (0.58)]\sqrt{1500} = 44.15 \text{ psi}$$

and with the increase by 1/3 for wind:

$$F_v = \left(\frac{4}{3}\right)(44.15) = 58.9 \text{ psi} < 74 \text{ psi (No Good)}$$

Thus, shear reinforcement will be required and shear reinforcement will be required to resist all the shear.

Design the shear reinforcement (MSJC Code Equation 2-24 applies). The "allowable" masonry shear stress (increase by 1/3 for wind):

$$F_v = \left(\frac{4}{3}\right)0.5\left[4 - \frac{M}{Vd}\right]\sqrt{f'_m} = 88 \text{ psi} > 74 \text{ psi OK}$$

Therefore, shear reinforcement is designed to carry the ENTIRE shear load, using MSJC Code Eq. 2-26:

$$A_v = \frac{Vs}{F_s d}$$

One alternative is to place the shear steel in bond beams at 48 in., that is, s = 48 in., gives,

$$A_v = \frac{69{,}650(48)}{(1.33)(24{,}000)(39.33)(12)} = 0.22 \text{ in.}^2$$

Thus, one solution would be to use 1 - #5 bar in bond beams spaced 4 ft-0 in. on centers.

Another solution is to consider the use of joint reinforcement at a 16 in. spacing (every other course). Note that the allowable stress for joint reinforcement is 30,000 psi, rather than the 24,000 psi allowed for deformed bars. Thus, the required steel area is:

$$A_s = 0.22\left(\frac{24}{30}\right)\left(\frac{16}{48}\right) = 0.059 \text{ in.}^2$$

Using the heavier truss style joint reinforcement with 3/16 in. side rods and No. 9 gage cross rods provides an area of 0.071 in.^{2} > 0.059 OK.

Yet, another solution is to consider if every course is reinforced with standard No. 9 gage ladder style joint reinforcement (an 8 in. spacing), the steel area provided in each course is 0.034 in.^{2}, while around 0.03 in.^{2} is required.

According to MSJC Code Section 2.3.5.3.2, orthogonal shear reinforcement is also required in an amount equal to 1/3 of the main shear reinforcement.

Thus, the total amount of vertical shear reinforcement required

$$A_s = 0.22\left(\frac{(40)(12)}{(48)(3)}\right) = 0.733 \text{ in.}^2$$

Using 7 - #3 bars at a 5 ft-0 in. spacing, gives a total steel area of 0.77 in.^{2}

Note that in this particular shear wall example the designer has three different options of the types of shear wall horizontal reinforcement, i.e. bond beams at 4 ft., heavy duty joint reinforcement at 16 in. o.c. or regular joint reinforcement at each bed joint at 8 in. o.c. Thus, the designer has the options of choosing the better economy between materials and labor for the reinforcement.

5.4.4 SHEAR PERPENDICULAR TO WALL

To compute the unit shear stress perpendicular to a masonry wall, the dimension *d* to the steel reinforcement could be used.

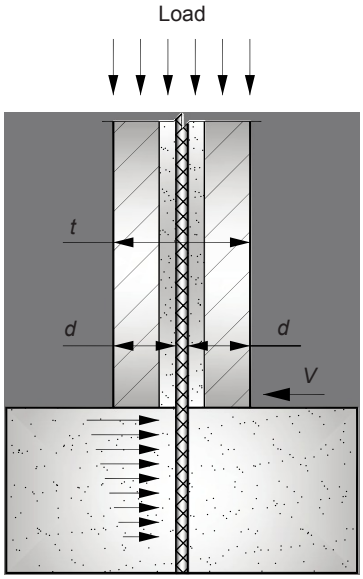


FIGURE 5.28 Shear resistance at floor joint.

To determine the unit shear at the base of the wall, it would be satisfactory to determine the unit shear stress $f_v = V/bt$ as unreinforced masonry or $f_v = V/bd$ as reinforced masonry.

The shear capacity of a masonry wall is influenced by vertical forces or loads on the wall. Vertical loads on a wall will increase its shear capacity by the added frictional resistance between the wall and the concrete footing or floor. The range and normal applicable coefficients for static friction are listed in Table 5.1.

TABLE 5.1 Coefficient of Static Friction

Materials	Range	Normal[1]
Masonry and masonry	0.65 – 0.75	0.70
Masonry and concrete	0.65 – 0.75	0.70
Masonry and dry earth	0.30 – 0.50	0.35
Masonry and metal	0.30 – 0.50	0.40
Concrete and dry earth	0.30 – 0.50	0.35
Masonry and wood	0.50 – 0.60	0.50

1. The normal coefficient values are reasonable to use to consider lateral frictional shear resistance.

Shear resistance of reinforcing steel at the floor joint can be conservatively assumed as the same as for anchor bolts. Values are given in Table ASD-8a. The connection between the floor, roof diaphragms and the walls must be capable of resisting a lateral force in any direction of at least 200 plf (MSJC Code Section 1.14.2.2.2.2).

EXAMPLE 5-O Determination of Shear Stresses for a Partially Grouted Wall.

Calculate the shear stress for an 8 in. hollow unit masonry wall shown below with steel grouted at 32 in. o.c. and a shear force of 200 plf.

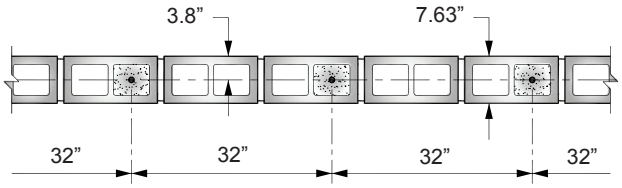
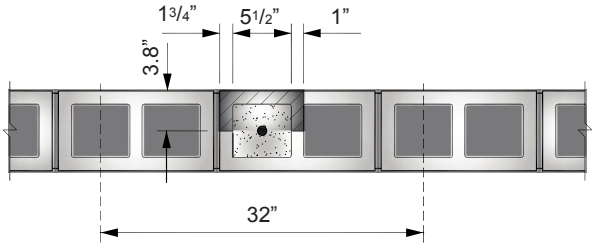


FIGURE 5.29 Plan section of hollow unit masonry wall.

Solution 5-O

Shear perpendicular to wall.

a. Minimum shear area; grouted cell + web + end wall



Min. Shear width per 32 in.

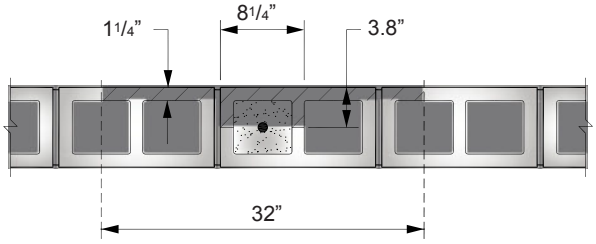
b = cell width + web + end web

$$= 5\frac{1}{2} + 1 + 1\frac{3}{4}$$

= 8.25 in. per 32 in.

$$f_v = \frac{V}{bd} = \frac{(200)(2.67)}{(8.25)(3.8)} = 17.0 \text{ psi}$$

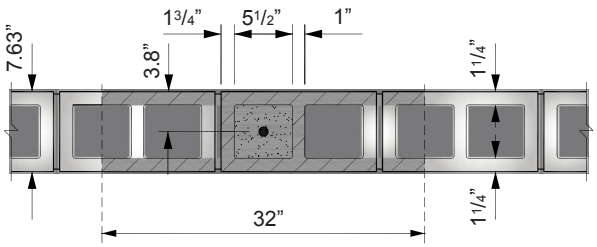
b. Shear area using grouted cell, web, end wall and one mortared face shell.



Shear area = (8.25)3.8
+ (32 - 8.25)1.25 = 61.0 sq in.

$$f_v = \frac{V}{\text{Shear area}} = \frac{(200)(2.67)}{61} = 8.8 \text{ psi}$$

c. Shear area for walls with no net tension stress use grouted cell, web, wall and both mortared face shells.



Shear area = (8.25)7.63 + (32 - 8.25)(1.25)(2)
= 62.9 + 59.4
= 122.3 sq in.

$$f_v = \frac{V}{\text{Shear area}} = \frac{(200)(2.67)}{122.3} = 4.4 \text{ psi}$$

Shear area parallel to wall (same as part c)

A_v = 122.3 sq in. per 32 in.

$$f_v = \frac{V}{bd} = \frac{(200)(2.67)}{122.3} = 4.4 \text{ psi};$$

shear parallel to wall

5.5 BOND

5.5.1 BOND IN MASONRY

Properly designed and placed mortar and grout will develop sufficient bond strength with the masonry units which will result in a homogeneous mass for design considerations within the range of allowable stresses. High bond strength results when the clay masonry units are saturated surface dry (*s.s.d.*) and have a suction or initial rate of absorption between 5 and 20 grams of water at time of being laid (See Section 1.2.1.3.3). Mortar Type M or S, which is workable and mixed with maximum amount of water produces the strongest bond strength.

5.5.2 BOND BETWEEN GROUT AND STEEL

Bond between mortar or grout and reinforcing steel is vital and necessary to insure that stresses will be transferred between the steel, the grout and the masonry units. The bond strength is developed by the adhesion of the portland cement paste and the mechanical interlock with the deformation of reinforcing steel. Older traditional means of determining bond is given in Table 5.2, from the UBC. However, today's IBC and MSJC Code do not use this procedure to determine bond stress. Instead, development length is used to design for bond.

In the report, *Bond and Splices in Reinforced Masonry*, by Soric and Tulin, 1987, the allowable bond stress could be 400 psi based on an experimental minimum test result of 1000 psi, before failure, with a factor of safety of 2.5 applied.

TABLE 5.2 Allowable Bond Stress, psi

	No Special Inspection	Special Inspection
Plain Bars	30	60
Deformed Bars 1988 UBC	70	140
Deformed Bars 1994/1997 UBC	100	200

EXAMPLE 5-P Determination of Bond Stress.

Calculate the bond stress, *u*, for a masonry beam reinforced with (a) two #6 bars, and (b) one #7 bar.

Given:

Span = 14 ft;

DL = 90 plf; *LL* = 200 plf

d = 20 in.; *b* = 9 in.

F_s = 24,000 psi; f'_m = 2500 psi

Solution 5-P

1) From Footnote 9 of Table ASD-3, the allowable bond stress in the masonry, *u* = 100 psi

2) Total shear, *V* = (90 + 200) (14) = 4060 lbs

3) From Table GN-19a the perimeter of the steel bars are given and the total perimeter may be computed as:

   a) Two #6 bars, Σ_o = 4.7 in.

   b) One #7 bar, Σ_o = 2.7 in.

4) Calculate bond stress

$$u = \frac{V}{\Sigma_o jd}$$

Assume *j* = 0.88

a) Two #6 bars

$$u = \frac{4060}{4.7(0.88)(20)} = 49 \text{ psi} < 100 \text{ psi} \quad \text{O.K.}$$

b) One #7 bar

$$u = \frac{4060}{2.7(0.88)(20)} = 85 \text{ psi} < 100 \text{ psi} \quad \text{O.K.}$$

5) Note: Since the reinforcing bars are embedded in at least 2000 psi concrete grout, it would also be reasonable to use the allowable bond stress for concrete.

Adequate bond between reinforcing steel and mortar or grout is assured by providing a sufficient length of bar to fully develop the stress within the bar. In general, this development must occur on the tension or compression development of reinforcement must occur on each side or direction of the section requiring the strength of the reinforcement. This development can be accomplished by straight development lengths, hooks, mechanical devices or a combination thereof. Hooks cannot be used to develop bars in compression. The development lengths are different for wires than for bars. The development length of bars in tension or compression is given in MSJC Code Section 2.1.10.3, as shown below:

MSJC Code Section 2.1.10.3

2.1.10.3 *Development of bars in tension and compression* — The required development length of reinforcing bars shall be determined by Eq. (2-9), but shall not be less than 12 in. (305 mm).

$$l_d = \frac{0.13 d_b^2 f_y \gamma}{K\sqrt{f'_m}} \qquad (2\text{-}9)$$

K shall not exceed the lesser of the masonry cover, clear spacing between adjacent reinforcement, nor 5 times d_b.

γ = 1.0 for No. 3 (M#10) through No. 5 (M#16) bars;

γ = 1.3 for No. 6 (M#19) through No. 7 (M#22) bars; and

γ = 1.5 for No. 8 (M#25) through No. 11 (M#36) bars.

When epoxy-coated reinforcing bars are used, development length determined by Eq. (2-9) shall be increased by 50 percent.

However, the criteria for development of wire reinforcement is contained in MSJC Code Section 2.1.10.2, as shown below:

MSJC Code Section 2.1.10.2

2.1.10.2 *Development of wires in tension* — The development length of wire shall be determined by Eq. (2-8), but shall not be less than 6 in. (152 mm).

$$l_d = 0.0015\, d_b F_s \qquad (2\text{-}8)$$

When epoxy-coated wires are used, development length determined by Eq. (2-8) shall be increased by 50 percent.

The embedment of flexural reinforcement typically follows similar criteria to ACI 318 that has been used by designers for many years. The criteria requires that tension reinforcement be fully developed at critical sections where maximum reinforcement stress is reached. These sections typically occur at points of maximum moment or where adjacent reinforcing steel is terminated or bent. In addition, the reinforcement must extend beyond the point where it is no longer required. This is a distance equal to the effective depth, d, of the member or twelve bar diameters, whichever is greater, except at the free end of a cantilever or at the supports of simple spans. The continuing reinforcement must extend a development distance, l_d, beyond where bent or terminated reinforcement is no longer required for flexure.

The flexural reinforcement cannot be terminated in a tension zone unless one of the following three criteria is met:

1. Shear at the cutoff point does not exceed 2/3 of the allowable shear at the section.
2. Stirrup area along the terminated bar is provided in excess of that provided for shear for a distance of 3/4 of the effective depth, d. This excess stirrup area cannot be less than $60bs/f_y$ and the spacing of the stirrups cannot exceed $d/(8\beta_b)$.
3. Continuous reinforcement must provide twice the area required for flexure and the shear cannot exceed 3/4 of the allowable shear at the section being considered.

Anchorage for tension reinforcement in corbels, deep flexural members, variable depth arches, and in cases where the reinforcement is not parallel to the compression face follow MSJC Code Section 2.1.10.4.1.6:

MSJC Code Section 2.1.10.4.1.6

2.1.10.4.1.6 Anchorage complying with Section 2.1.10.2 or 2.1.10.3 shall be provided for tension reinforcement in corbels, deep flexural members, variable-depth arches, members where flexural reinforcement is not parallel with the compression face, and in other cases where the stress in flexural reinforcement does not vary linearly in proportion to the moment.

For development of the reinforcement in a positive bending moment region, follow MSJC Code Section 2.1.10.4.2:

MSJC Code Section 2.1.10.4.2

2.1.10.4.2 *Development of positive moment reinforcement* — When a wall or other flexural member is part of a primary lateral resisting system, at least 25 percent of the positive moment reinforcement shall extend into the support and be anchored to develop a stress equal to the F_s in tension.

However, the development of reinforcement in a negative bending moment region must follow MSJC Code Section 2.1.10.4.3:

MSJC Code Section 2.1.10.4.3

2.1.10.4.3 *Development of negative moment reinforcement*

2.1.10.4.3.1 Negative moment reinforcement in a continuous, restrained, or cantilever member shall be anchored in or through the supporting member in accordance with the provisions of Section 2.1.10.1.

2.1.10.4.3.2 At least one-third of the total reinforcement provided for moment at a support shall extend beyond the point of inflection the greater distance of the effective depth of the member or one-sixteenth of the span.

The development of hooks is simplified from the traditional ACI 318 criteria. The MSJC Code Section for hooks is very short and consists of the following language:

MSJC Code Section 2.1.10.5

2.1.10.5 *Hooks*

2.1.10.5.1 Standard hooks in tension shall be considered to develop an equivalent embedment length, l_e, equal to 11.25 d_b.

The development of the shear reinforcement includes criteria for both the wire and bar reinforcement in accordance with MSJC Code Section 2.1.10.6, as shown:

MSJC Code Section 2.1.10.6

2.1.10.6 *Development of shear reinforcement*

2.1.10.6.1 *Bar and wire reinforcement*

2.1.10.6.1.1 Shear reinforcement shall extend to a distance d from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its calculated stress.

2.1.10.6.1.2 The ends of single leg or U-stirrups shall be anchored by one of the following means:

(a) A standard hook plus an effective embedment of 0.5 l_d. The effective embedment of a stirrup leg shall be taken as the distance between the middepth of the member, $d/2$, and the start of the hook (point of tangency).

(b) For No. 5 bar (M #16) and D31 (MD200) wire and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of 0.33 l_d. The 0.33 l_d embedment of a stirrup leg shall be taken as the distance between middepth of member, $d/2$, and start of hook (point of tangency).

2.1.10.6.1.3 Between the anchored ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

2.1.10.6.1.4 Longitudinal bars bent to act as shear reinforcement, where extended into a region of tension, shall be continuous with longitudinal reinforcement and, where extended into a region of compression, shall be developed beyond middepth of the member, $d/2$.

2.1.10.6.1.5 Pairs of U-stirrups or ties placed to form a closed unit shall be considered properly spliced when length of laps are 1.7 l_d. In grout at least 18 in. (457 mm) deep, such splices with $A_v f_y$ not more than 9,000 lb (40 032 N) per leg shall be permitted to be considered adequate if legs extend the full available depth of grout.

2.1.10.6.2 *Welded wire fabric*

2.1.10.6.2.1 For each leg of welded wire fabric forming simple U-stirrups, there shall be either:

(a) Two longitudinal wires at a 2-in. (50.8-mm) spacing along the member at the top of the U, or

(b) One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 2 in. (50.8 mm) from the first wire. The second wire shall be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than $8d_b$.

2.1.10.6.2.2 For each end of a single leg stirrup of welded smooth or deformed wire fabric, there shall be two longitudinal wires spaced a minimum of 2 in. (50.8 mm) with the inner wire placed at a distance at least $d/4$ or 2 in. (50.8 mm) from middepth of member, $d/2$. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

Splicing of the reinforcement can be accomplished by lap splices, welded splices or mechanical connections. The welding must conform to that of AWS D1.4. The welded splices must develop at least 125 percent of the specified yield strength of the bar. Likewise, the mechanical splice connections must develop 125 percent of the specified yield strength of the bar. End-bearing splices follow MSJC Code Section 2.1.10.7.4, as follows:

MSJC Code Section 2.1.10.7.4

2.1.10.7.4 *End-bearing splices*

2.1.10.7.4.1 In bars required for compression only, the transmission of compressive stress by bearing of square cut ends held in concentric contact by a suitable device is permitted.

2.1.10.7.4.2 Bar ends shall terminate in flat surfaces within 1 1/2 degree of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

2.1.10.7.4.3 End-bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

IBC Section 2107.6

2107.6 ACI 530/ASCE 5/TMS 402, Section 2.1.10.7, splices of reinforcement. Modify Section 2.1.10.7 as follows:

2.1.10.7 Splices of reinforcement. Lap splices, welded splices or mechanical splices are permitted in accordance with the provisions of this section. All welding shall conform to AWS D1.4. Reinforcement larger than No. 9 (M #29) shall be spliced using mechanical connections in accordance with Section 2.1.10.7.3.

IBC Section 2701.5 provides the lap splice criteria requirements and modifies MSJC Code Section 2.1.10.7.1.1:

IBC Section 2107.5

2107.5 ACI 530/ASCE 5/TMS 402, Section 2.1.10.7.1.1, lap splices. Modify Section 2.1.10.7.1.1 as follows:

2.1.10.7.1.1 The minimum length of lap splices for reinforcing bars in tension or compression, l_d, shall be

$$l_d = 0.002 d_b f_s \qquad \textbf{(Equation 21-2)}$$

For SI: $l_d = 0.29 d_b f_s$

but not less than 12 inches (305 mm). In no case shall the length of the lapped splice be less than 40 bar diameters.

where:

d_b = Diameter of reinforcement, inches (mm).

f_s = Computed stress in reinforcement due to design loads, psi (MPa).

In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tension stress, F_s, the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase shall be permitted.

Where epoxy coated bars are used, lap length shall be increased by 50 percent.

MSJC Code Section 2.1.10.7.1

2.1.10.7.1 *Lap splices*

~~**2.1.10.7.1.1** The minimum length of lap for bars in tension or compression shall be determined by Eq. (2-9), but not less than 12 in. (305 mm).~~

2.1.10.7.1.2 Bars spliced by noncontact lap splices shall not be spaced transversely farther apart than one-fifth the required length of lap nor more than 8 in. (203 mm).

EXAMPLE 5-Q Development Length.

Given #5 reinforcing bar Grade 60, F_s = 24,000 psi, f'_m = 1,500 psi, determine the following for Allowable Stress Design:

a) Development length, straight bar

b) Lap splice length

c) Development length provided by hooks

Solution 5-Q

The development length for deformed reinforcing steel in tension is calculated as follows:

a) Development length:

K = 5(0.625) = 3.125 in. ≤ Cover distance

γ = 1.0 for #5 bars

$$l_d = \frac{0.13 d_b^2 f_y \gamma}{K\sqrt{f'_m}} \qquad \text{(MSJC Code Eq 2-9)}$$

$$= \frac{0.13(0.625)^2(60{,}000)(1.0)}{(3.125)\sqrt{1500}}$$

= 25.17 in. > 12 in. (Minimum development length, MSJC Code Section 2.1.10.3)

b) Splice length for lap splices:

l_d = 0.002$d_b f_s$ (IBC Eq 21-2)

but not less than 12 inches. In regions of moment where the tensile stresses are greater than 80% of the allowable steel tension stress, F_s, the lap length of splices shall be increased by at least 50%.

l_d = 0.002(0.625)(24,000) = 30 in.

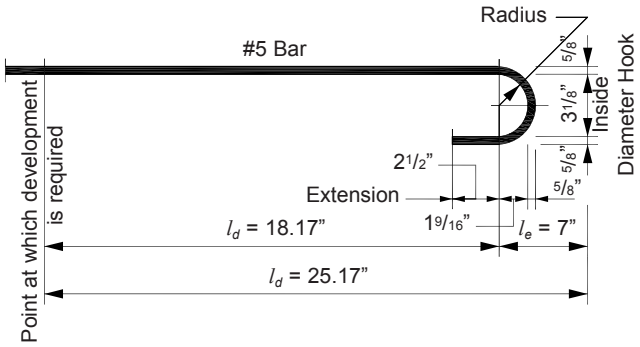
c) Development length provided by hooks:

l_e = 11.25 d_b (MSJC Code Section 2.1.10.5.1)

= 11.25(0.625) = 7 in.

Thus, the remaining development length required for a hooked bar is:

$l_{d\ balance\ required}$ = l_d - l_e = 25.17 - 7 ≅ 18 in.
(development length in addition to the hook)



See Table ASD-22 for development length provided by a hook and for detail dimensions. Inside hook diameter is five bar diameters (5d_b) for a #5 bar (MSJC Code Section 1.13.6). An extension of four bar diameters (4d_b) is required on the hook (2 1/2 in. minimum per MSJC Code Section 1.13.5a).

5.6 COMPRESSION IN WALLS AND COLUMNS

5.6.1 WALLS

MSJC Code Section 1.6

Wall — A vertical element with a horizontal length to thickness ratio greater than 3, used to enclose space.

5.6.1.1 GENERAL

Load bearing reinforced masonry walls are limited to an axial load of:

$$P = F_a A_e$$

Where:

$$F_a = \frac{1}{4} f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ for } \frac{h}{r} \le 99$$

(MSJC Code Eq 2-12)

or

$$F_a = 0.25 f'_m \left(\frac{70r}{h}\right)^2 \text{ for } \frac{h}{r} > 99 \text{ (MSJC Code Eq 2-13)}$$

A_e = effective cross-sectional area of masonry which includes grouted and mortared areas.

For cavity walls consider only the loaded wythes. If mortar joints are raked, reduce the effective area accordingly. At the *h/r* transition point of 99, the F_a values calculate the same for either reduction factor.

Any vertical wall reinforcement is not considered effective in carrying vertical loads since it is not confined by ties. Thus the reinforcing steel is considered effective only for resisting lateral loads parallel and perpendicular to the wall. The allowable load bearing wall stress, F_a, is the same for both reinforced and unreinforced masonry.

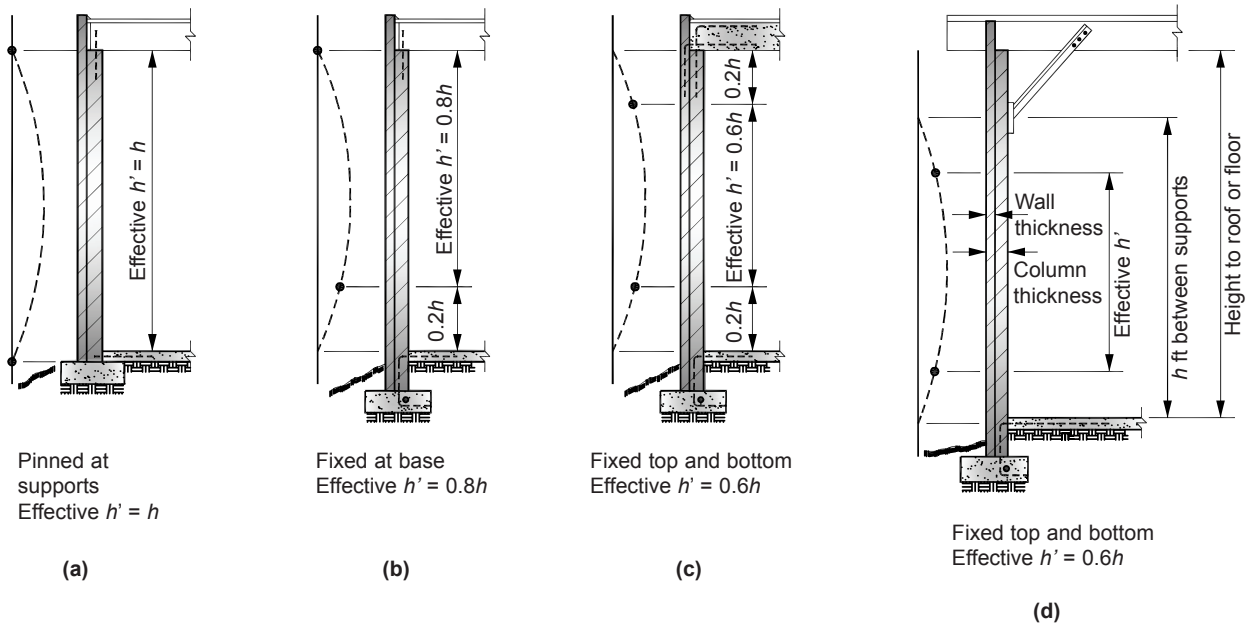
5.6.1.2 STRESS REDUCTION AND EFFECTIVE HEIGHT

The stress reduction factor is based on the radius of gyration of the section. Tables GN-4 to GN-16 contain values for the radius of gyration, *r*, which can also be calculated as

$$\sqrt{\frac{I}{A}}$$

Figure 5.30 shows conditions that describe the effective height of a wall. For members not supported at the top normal to the plane of the wall, the effective height, *h*', is considered twice the height of the member above the base.

If a wall spans horizontally, the wall can be considered to be continuous over vertical supports such as pilasters or intersecting walls. Such a continuous wall would have inflection points at approximately the quarter points although they are often conservatively assumed to be 0.2*l* from the supports (See Figure 5.31). The effective length (or *h*') of the wall is the distance between points of inflection or 0.6*l*.



Pinned at supports
Effective *h*' = *h*

(a)

Fixed at base
Effective *h*' = 0.8*h*

(b)

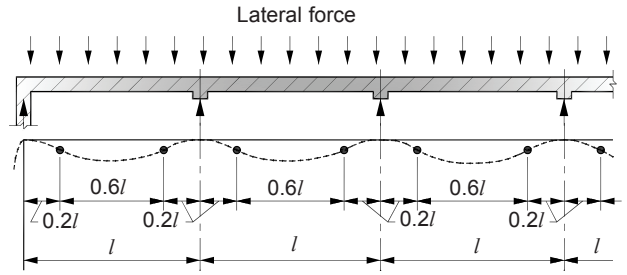
Fixed top and bottom
Effective *h*' = 0.6*h*

(c)

Fixed top and bottom
Effective *h*' = 0.6*h*

(d)

FIGURE 5.30 Conditions of effective height *h'*.



Effective l = 0.6l

FIGURE 5.31 Longitudinal continuity of wall.

5.6.1.3 EFFECTIVE WIDTH

The effective width of a flexural wall member may be either horizontal or vertical depending on the way the wall spans. There should be consideration as to whether the wall is laid up in running bond or stack bond and whether the units are solid grouted, or open ended units.

For running bond, the effective width used in computing flexural stresses must not be greater than six times the wall thickness nor the center to center distance between the reinforcing bars, nor 72 inches.

MSJC Code Section 2.3.3.3

2.3.3.3 *Effective compressive width per bar*

2.3.3.3.1 In running bond masonry, and masonry in other than running bond with bond beams spaced not more than 48 in. (1219 mm) center-to-center, the width of the compression area used in stress calculations shall not exceed the least of:

(a) Center-to-center bar spacing.
(b) Six times the nominal wall thickness.
(c) 72 in. (1829 mm).

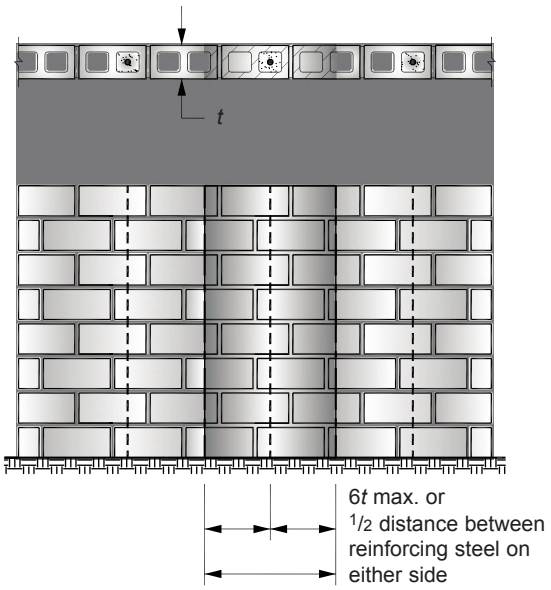
2.3.3.3.2 In masonry in other than running bond, with bond beams spaced more than 48 in. (1219 mm) center-to-center, the width of the compression area used in stress calculations shall not exceed the length of the masonry unit.

MSJC Code Section 1.6, Definitions, states that a wall is considered to be laid in running bond or stack bond by the following two definitions:

MSJC Code Section 1.6

Running bond — The placement of masonry units such that head joints in successive courses are horizontally offset at least one-quarter the unit length.

Stack bond — For the purpose of this Code, stack bond is other than running bond. Usually the placement of units is such that the head joints in successive courses are vertically aligned.



* Maximum spacing of steel up to 8 ft. has been shown to be effective based on an Effective 'b' research program by the Masonry Institute of America.

FIGURE 5.32 Effective width of flexural member, running or common bond.

Where stack bond is used, buildings in Seismic Design Categories D and higher must be reinforced with a minimum A_s of 0.0007bt distributed uniformly with joint reinforcement or reinforcing steel spaced at a maximum of two feet on centers. Additional restrictions apply to stack bond walls that are part of the lateral force-resisting system.

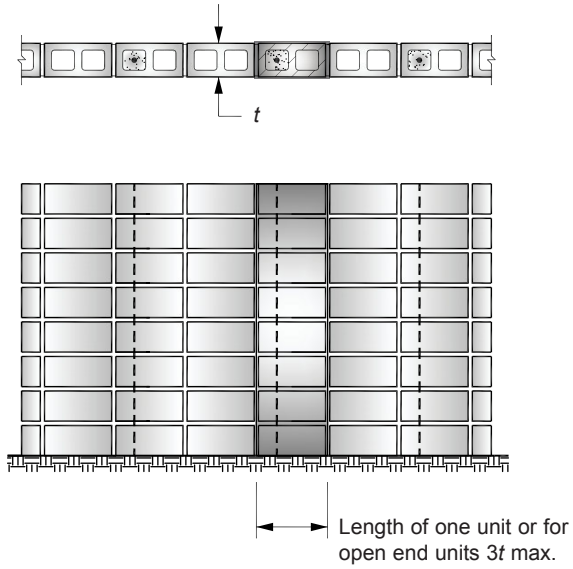


FIGURE 5.33 Effective width of flexural member, stack bond.

EXAMPLE 5-R Lateral Wind Force on Wall, Flexural Design.

Determine the required flexural reinforcement for a 16 ft 0 in. high, 8 in. concrete masonry wall subjected to a 45 psf lateral wind load.

Given:

d = 3.75 in.,

f'_m = 1500 psi, and

F_s = 24,000 psi

Solution 5-R

F_b = 500 psi, n = 21.5 (Table ASD-3)

$\frac{4}{3}F_b = 667$ psi (one third increase allowed for wind load by MSJC Code Section 2.1.2.3)

$\frac{4}{3}F_s = 32{,}000$ psi

(1) Assume pin connections at top and bottom of wall:

$$M = \frac{wl^2}{8} = \frac{45(16)^2}{8}(12) = 17{,}280 \text{ in. - lbs/ft}$$

(2) Flexural coefficient:

$$K_f = \frac{M}{bd^2} = \frac{17{,}280}{(12)(3.75)^2} = 102.4$$

(3) Enter Diagram ASD-34 with the flexural coefficient, K_f = 102.4 and F_b = 667 psi.

Read $n\rho$ = 0.09.

(4) Steel ratio $\rho = \frac{n\rho}{n} = \frac{0.09}{21.5} = 0.0042$

(5) From Table GN-23b for d = 3.75 in. and ρ = 0.0042, choose #6 at 28 in. o.c. (A_s = 0.44 sq in./ft) or rather #6 at 24 in. for CMU cell spacing.

Note that $A_s = \rho bd$ = 0.0042(12)(3.75) = 0.189 and 0.44(12/24) = 0.22 O.K.

Alternate method, use Table ASD-24b:

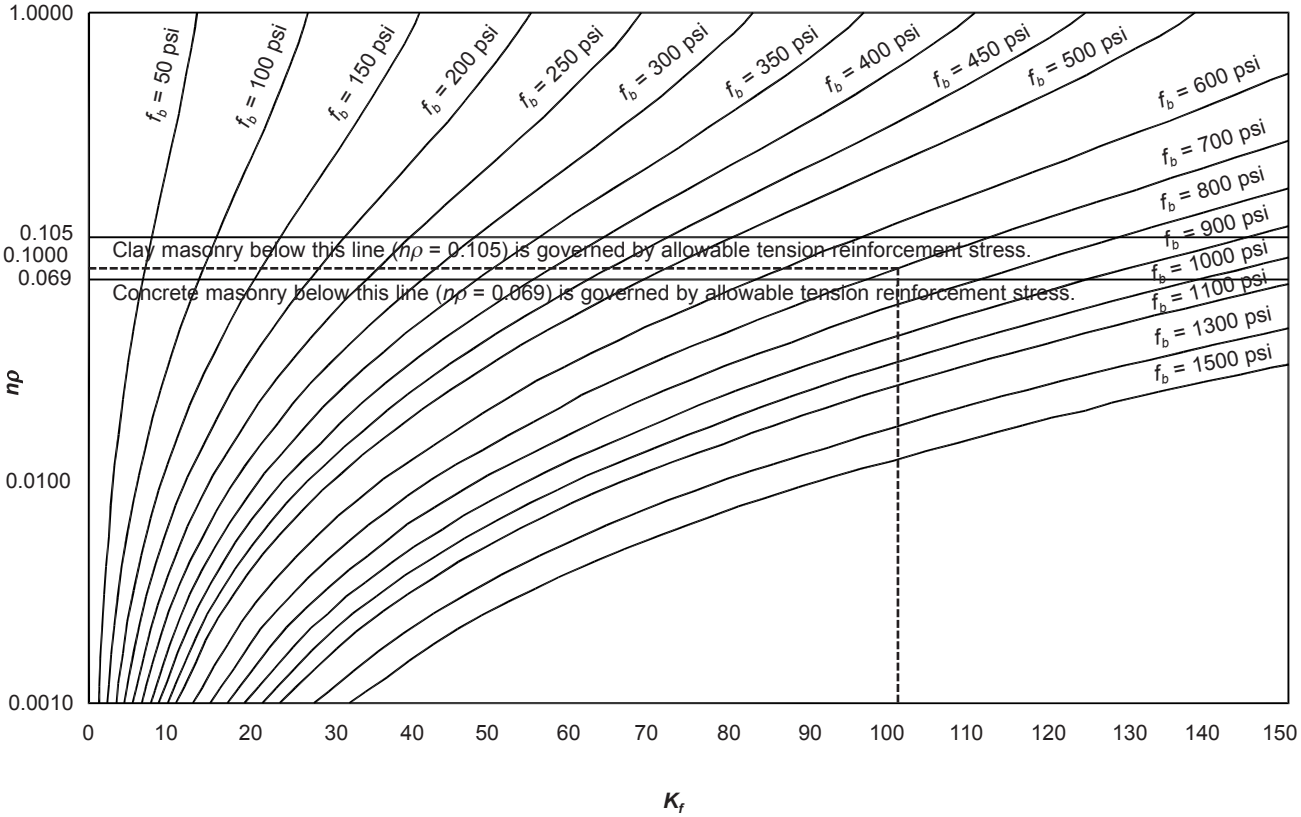
(6) From Table ASD-24b for $\frac{4}{3}K_f = 102.4$

f_b = 667 psi f_s = 26,950 psi

ρ = 0.00431 A_s = 0.00431(12)(3.75) = 0.194 sq in. < 0.22 sq in. O.K.

(7) Again choose #6 at 24 in.

Diagram ASD-34 K_f versus $n\rho$ for Various Masonry Stresses f_b



EXAMPLE 5-S Minimum Wall Thickness.

For a solid grouted clay masonry, non-load bearing exterior wall subjected to a lateral wind force, determine the minimum wall thickness when the steel is located in the center of the wall.

Given:

Height of wall = 25 ft

Wind load = 30 psf

f'_m = 2000 psi

f_y = 60,000 psi

ρ = 0.0013

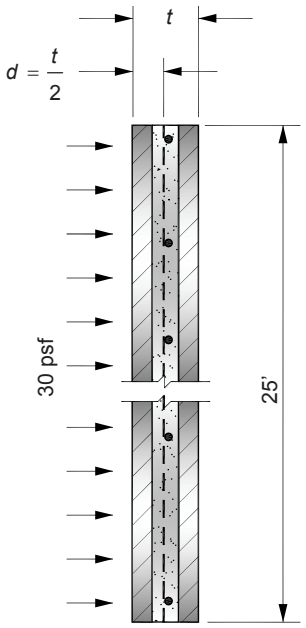


FIGURE 5.34 Lateral load on wall.

Solution 5-S

(1) From Tables ASD-3 and ASD-4, find the allowable stresses. These allowable stresses may be increased by one-third since load is due to wind (MSJC Code Section 2.1.2.3).

$$F_b = \frac{4}{3}(667) = 890 \text{ psi}$$

$$F_s = \frac{4}{3}(24{,}000) = 32{,}000 \text{ psi}$$

(2) Enter Diagram ASD-24a and proceed vertically up the ρ = 0.0013 line until it intersects with F_s, or F_b. Find minimum K_f by proceeding to the left of the lowest intersection. Read $K_{min} \approx 38$.

(3) $K_f = \dfrac{M}{bd^2}$ Calculate M by assuming the wall is pinned at the top and bottom.

$$M = \frac{wl^2}{8} = \frac{30(25)^2}{8} = 2344 \text{ ft - lbs/ft}$$

$$= 28{,}125 \text{ in.-lbs/ft}$$

Since b = 12 in./ft, the above equation can be solved for d.

$$(d_{\min})^2 = \frac{M}{bK}$$

$$d_{\min} = \sqrt{\frac{28{,}125}{12(38)}} = 7.86 \text{ in.}$$

Since $d = t/2$, use a 16 in. thick wall.

(4) Check stresses with d = 8 in.

$$K = \frac{M}{bd^2} = \frac{28{,}125}{12(8)^2} = 36.6$$

Enter Diagram ASD-25a with K_f = 36.6 and ρ = 0.0013 read:

$f_b \approx 400 \text{ psi} < 890 \text{ psi}$ O.K.

$f_s \approx 31{,}000 \text{ psi} < 32{,}000 \text{ psi}$ O.K.

EXAMPLE 5-T Moment Capacity of Wall Reinforced with Minimum Reinforcement.

Determine the moment capacity of a grouted clay masonry wall which spans vertically and is reinforced with the minimum area of steel. Also, find the allowable uniform pressure, in Figure 5.35, the wall can support if it spans 15 ft vertically.

Assume:

f'_m = 3000 psi

f_y = 60,000 psi and F_s = 24,000 psi

t = 9 in.

Vertical steel, $A_s = 0.0013bt$

Horizontal steel, $A_s = 0.0007bt$

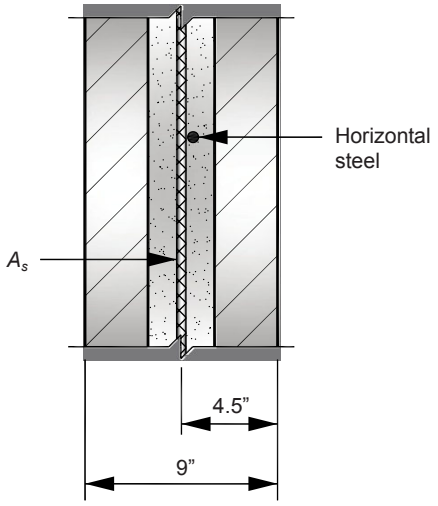


FIGURE 5.35 Location of steel in wall.

Solution 5-T

Part (a) Moment Capacity

From Table ASD-47a with A_s = 0.0013*bt*, *d* = 4.5 in., f'_m = 3000 psi and F_s = 24,000 psi:

M_m = 2.19 ft-k/ft

M_s = 1.16 ft-k/ft

Therefore, M_s controls the design and the

Moment capacity of wall = 1.16 ft-k/ft

= 1160 ft-lbs/ft

Part (b) Lateral Load

Assume the wall is simply supported at the top and bottom. Thus, the maximum lateral load the wall can safely support is:

$$M = \frac{wL^2}{8} \text{ or } w = \frac{8M}{L^2}$$

$$w = \frac{8(1160)}{15^2} = 41\text{psf}$$

5.6.2 COLUMNS

MSJC Code Section 1.6

Column — An isolated vertical member whose horizontal dimension measured at right angles to its thickness does not exceed 3 times its thickness and whose height is greater than 4 times its thickness.

5.6.2.1 GENERAL

A masonry column is a vertical structural member designed primarily to support vertical and axial loads. In a reinforced column the masonry and reinforcing steel share in supporting imposed vertical loads and any overturning moment. The reinforcing steel is secured with horizontal ties or other suitable means to properly locate the steel and provide confinement.

The area of vertical reinforcement in a masonry column may not be less than 0.25% or more than 4% of the effective cross-sectional area of the column. At least four vertical reinforcing bars must be provided in all columns, except for code-defined lightly loaded columns which may be reinforced with a single vertical bar.

Details of reinforcement and ties are shown in Chapter 7.

The maximum allowable axial load on a reinforced masonry column is:

for $\frac{h}{r} \le 99$

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

(MSJC Code Eq 2-17)

for $\frac{h}{r} > 99$

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left(\frac{70r}{h}\right)^2$$

(MSJC Code Eq 2-18)

The maximum allowable unit axial stress is:

$$f_a = \frac{P_a}{A_n}$$

The reduction factor based on the *h/r* ratio is the same for reinforced columns as for walls. The same consideration is made for the determination of the effective height, *h*', which is used in the *h/r* ratio.

The effective thickness, *t*, is the specified thickness in the direction considered. For non-rectangular columns the effective thickness is the thickness of a square column with the same moment of inertia.

MSJC Code Section 2.1.6 provides requirements for columns as follows:

MSJC Code Section 2.1.6

2.1.6 *Columns*

Design of columns shall meet the general requirements of this section.

2.1.6.1 Minimum side dimension shall be 8 in. (203 mm) nominal.

2.1.6.2 The ratio between the effective height and least nominal dimension shall not exceed 25.

2.1.6.3 Columns shall be designed to resist applied loads. As a minimum, columns shall be designed to resist loads with an eccentricity equal to 0.1 times each side dimension. Consider each axis independently.

2.1.6.4 Vertical column reinforcement shall not be less than $0.0025A_n$ nor exceed $0.04A_n$. The minimum number of bars shall be four.

2.1.6.5 *Lateral ties* — Lateral ties shall conform to the following:

(a) Longitudinal reinforcement shall be enclosed by lateral ties at least 1/4 in. (6.4 mm) in diameter.

(b) Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 lateral tie bar or wire diameters, or least cross-sectional dimension of the member.

(c) Lateral ties shall be arranged so that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135 degrees. No bar shall be farther than 6 in. (152 mm) clear on each side along the lateral tie from such a laterally supported bar. Lateral ties shall be placed in either a mortar joint or in grout. Where longitudinal bars are located around the perimeter of a circle, a complete circular lateral tie is permitted. Lap length for circular ties shall be 48 tie diameters.

(d) Lateral ties shall be located vertically not more than one-half lateral tie spacing above the top of footing or slab in any story, and shall be spaced not more than one-half a lateral tie spacing below the lowest horizontal reinforcement in beam, girder, slab, or drop panel above.

(e) Where beams or brackets frame into a column from four directions, lateral ties shall be permitted to be terminated not more than 3 in. (76.2 mm) below the lowest reinforcement in the shallowest of such beams or brackets.

IBC Section 2107.4 provides some additions to MSJC Code Section 2.1.6 to light-frame construction column provisions as follows:

IBC Section 2107.4

2107.4 ACI 530/ASCE 5/TMS 402, Section 2.1.6, columns. Add the following text to Section 2.1.6:

2.1.6.6 Light-frame construction. Masonry columns used only to support light-frame roofs of carports, porches, sheds or similar structures with a maximum area of 450 square feet (41.8 m^2) assigned to Seismic Design Category A, B or C are permitted to be designed and constructed as follows:

1. Concrete masonry materials shall be in accordance with Section 2103.1 of the *International Building Code*. Clay or shale masonry units shall be in accordance with Section 2103.2 of the *International Building Code*.
2. The nominal cross-sectional dimension of columns shall not be less than 8 inches (203 mm).
3. Columns shall be reinforced with not less than one No. 4 bar centered in each cell of the column.
4. Columns shall be grouted solid.
5. Columns shall not exceed 12 feet (3658 mm) in height.
6. Roofs shall be anchored to the columns. Such anchorage shall be capable of resisting the design loads specified in Chapter 16 of the *International Building Code.*
7. Where such columns are required to resist uplift loads, the columns shall be anchored to their footings with two No. 4 bars extending a minimum of 24 inches (610 mm) into the columns and bent horizontally a minimum of 15 inches (381 mm) in opposite directions into the footings. One of these bars is permitted to be the reinforcing bar specified in Item 3 above.

   The total weight of a column and its footing shall not be less than 1.5 times the design uplift load.

EXAMPLE 5-U Column Capacity.

A CMU column located in SDC B is shown in Figure 5.36.

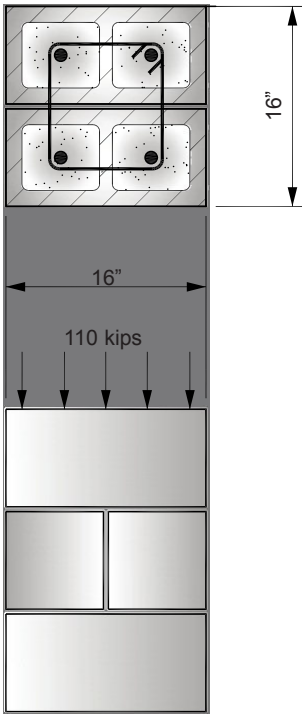


FIGURE 5.36 CMU column.

Given:

P = 110 kips

Effective height of column, h' = 10 ft-0 in. = 120 in.

f'_m = 1500 psi

f_y = 60,000 psi

Type S portland cement/lime mortar

Determine the required reinforcement. Try a nominal 16 x 16 in. square concrete masonry column consisting of two nominal 8 x 16 in. CMU sections with a unit compressive strength of 1900 psi. Assume pinned ends.

Solution 5-U

f'_m = 1500 psi

Area = (15.625)(15.625) = 244.1 in.^{2}

$$\text{Section modulus, } S = \frac{bt^2}{6} = \frac{15.625(15.625)^2}{6}$$

$$= 635.8 \text{ in.}^3$$

Minimum eccentricity, e, per MSJC Code Section 2.1.6.3 is 0.1 times each side dimension

e = 0.1(15.625) = 1.563 in.

Thus, $M = (P)(e)$ = (110,000)(1.563) = 171,930 in.-lb

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{bt^3/12}{A}}$$

$$= \sqrt{\frac{(15.625)(15.625)^3/12}{(15.625)(15.625)}}$$

$$= 4.51 \text{ in.}$$

$$\frac{h}{r} = \frac{120}{4.51} = 26.6 < 99$$

Thus, from MSJC Code Equation 2-12:

$$F_a = 0.25f'_m\left[1-\left(\frac{h}{140r}\right)^2\right] \text{ for } \frac{h}{r} \le 99$$

$$F_a = 0.25(1500)\left[1-\left(\frac{26.6}{140}\right)^2\right] = 361 \text{ psi}$$

$F_b = \frac{1}{3}f'_m$ from MSJC Code Equation 2-14

F_b = 1500/3 = 500 psi

$$f_a = \frac{P}{A} = \frac{110{,}000}{244.1} = 450.6 \text{ psi} > 361 \text{ psi}$$

NG, needs reinforcement

$$f_b = \frac{M}{S} = \frac{171{,}930}{635.8} = 270.4 \text{ psi} < 500 \text{ psi}$$

Since the computed axial stress exceeds the allowable stress, reinforcement is needed and can be determined from the MSJC Code Equation 2-17, using F_s = 24,000 psi and P_a = 110,000 lbs.

$$\text{Thus, } P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left[1-\left(\frac{h}{140r}\right)^2\right]$$

$$110{,}000 = [(0.25)(1500)(244.1 - A_{st})$$

$$+ 0.65A_{st}(24{,}000)]\left[1-\left(\frac{26.6}{140}\right)^2\right]$$

which calculates minimum A_{st} = 1.48 in.^{2}

Use 4 - #6 bars for A_{st} = 1.76 in.^{2}

Next, check the masonry compressive stress using MSJC Code Section 2.3.3.2.2. In this case a transformed area of steel in compression will be taken as $2n$ as by convention from reinforced concrete over many years, although this is not a code provision.

Thus, $2n = 2\dfrac{E_s}{E_m}$

$E_m = 900f'_m = 900(1500) = 1{,}350{,}000$ psi

$$2n = 2\left(\frac{29{,}000{,}000}{1{,}350{,}000}\right) = 43.0$$

Thus, the transformed area is

$= (244.1 - A_{st}) + 2n\,A_{st}$

$= (244.1 - 1.76) + 43(1.76)$

$= 318.0$ in.^{2}

$f_a = 110{,}000/318 = 345.9$ psi < 361 psi

Also, from MSJC Code Section 2.3.3.2.2, check $f_a + f_b < f'_m/3$. In order to compute the bending stress from the eccentric loading, the section modulus for the reinforced section is needed. The moment of inertia of the transformed section will be found, assuming that the bars are located in the center of each cell, 7.3 in. apart. Since the entire section is under compressive stress, the steel will be transformed by $2n$ (as above).

$$I = \frac{bt^3}{12} + 2(2n-1)\left(\frac{A_{st}}{2}\right)\left(\frac{7.3}{2}\right)^2$$

$$= \frac{(15.625)(15.625)^3}{12} + 2(43-1)\left(\frac{1.76}{2}\right)\left(\frac{7.3}{2}\right)^2$$

$= 5952$ in.^{4}

$$S = \frac{I}{t/2} = 2\frac{5952}{15.625} = 761.8 \text{ in.}^3$$

$$f_b = \frac{M}{S} = \frac{171{,}930}{768.8} = 223.6 \text{ psi}$$

Thus, the combined compression stress

$= 345.9 + 223.6 = 569.5 > 500$ NG

Therefore, it is necessary to increase the area of steel.

Try 4 - #8 bars, $A_{st} = 3.16$ in.^{2}

The revised transformed area is

$(244.1 - 3.16) + 43(3.16) = 376.8$ in.^{2}

The revised moment of inertia of the column section is

$$\frac{(5.625)(15.625)^3}{12} + 2(43-1)\left(\frac{3.16}{2}\right)\left(\frac{7.3}{2}\right)^2$$

$= 6735$ in.^{4}

and the revised section modulus is

$$S = \frac{I}{t/2} = 2\frac{6735}{15.626} = 862.1 \text{ in.}^3$$

Therefore, $f_a + f_b = \dfrac{P}{A} + \dfrac{M}{S}$

$$= \frac{110{,}000}{376.8} + \frac{171{,}930}{862.1}$$

$= 291.9 + 199.4 = 491.3 < 500$ O.K.

Next check MSJC Code Section 2.1.6.4

Max. area of steel $= 0.04\,A_n = 0.04(244.1)$

$= 9.76$ in.^{2}

Min. area of steel $= 0.0025\,A_n = 0.0025(244.1)$

$= 0.61$ in.^{2}

The provided area of steel of 3.16 in.^{2} falls within the prescribed limits.

Lateral ties are provided to enclose the longitudinal column steel reinforcement. MSJC Code Section 2.1.6.5 states that at least a tie diameter of $^1/_4$ in. must be provided and spaced within the following maximum limits:

16 bar diameters = 16 (1.00) = 16 in.

48 tie diameters = 48 (0.375) = 18 in.

(assuming a #3 tie)

least column dimension = 16 in.

Thus, #3 ties at 16 in. on centers, or every other course, works. If #2 ties are available these would be placed at every course, but the #3 tie will require some joint treatment to maintain the proper cover. Note all four cells of Figure 5.36 require grouting, and all four longitudinal bars must be confined by the ties.

5.6.2.2 PROJECTING PILASTER

Vertical load-carrying elements located in a wall but which project from the plane of the wall are called pilasters. Generally, these pilasters are not called columns unless they meet all the tie requirements of columns.

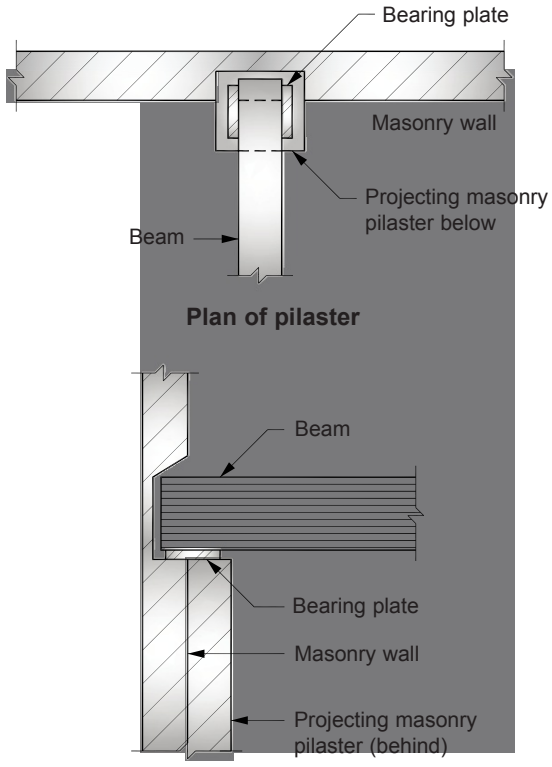


FIGURE 5.37 Masonry pilaster.

Pilasters are built integrally with the wall and in addition to supporting vertical loads can also be designed to carry lateral loads from adjacent wall sections. The magnitude of lateral load to the pilaster is dependent on the height of the pilaster and the spacing between pilasters. For tall, closely spaced pilasters with a height to spacing ratio of 2 or more, it may be assumed that the walls span horizontally.

For lower walls with a wider spacing of pilaster and a height/spacing $\leq$ 1, the walls are usually assumed to span vertically and a triangular section of laterally loaded wall is carried by the pilasters. The triangular area is often assumed as 45 degrees to the horizontal. This procedure is modified if two-way plate action of the wall is considered; however, that theoretical complexity is usually not done in normal masonry design.

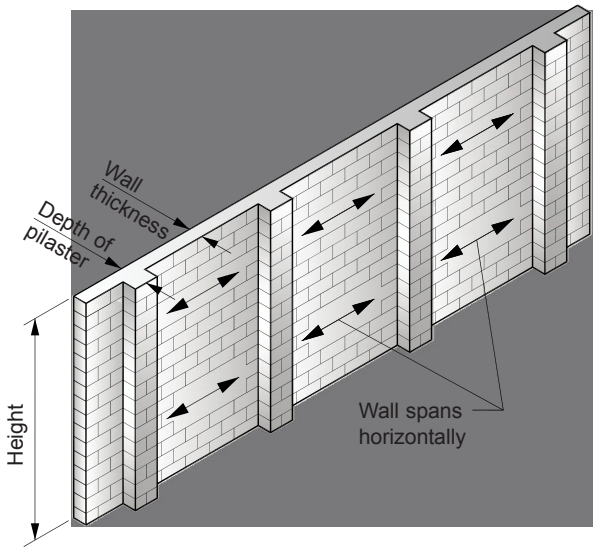


FIGURE 5.38 Wall loads to pilaster.

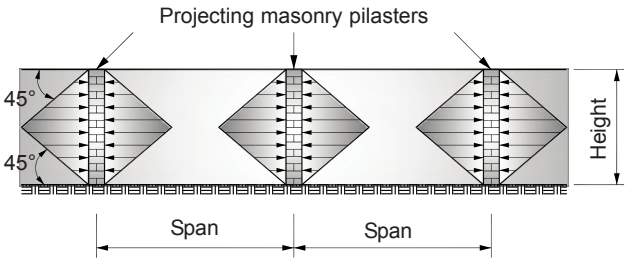


FIGURE 5.39 Lateral wall loads to pilaster.

5.6.2.3 DESIGN OF PILASTERS

For the support of the vertical load, a projecting pilaster can be designed as a reinforced masonry column utilizing the rectangular cross-section of the element.

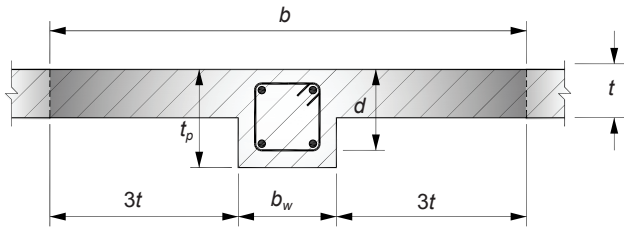


FIGURE 5.40 Projecting pilaster and width of effective wall section.

The lateral loads and eccentric vertical loads on a pilaster impose a moment on the wall and pilaster. Two conditions of loading may be considered.

a. Loads causing tension on the wall and compression on the projecting pilaster.

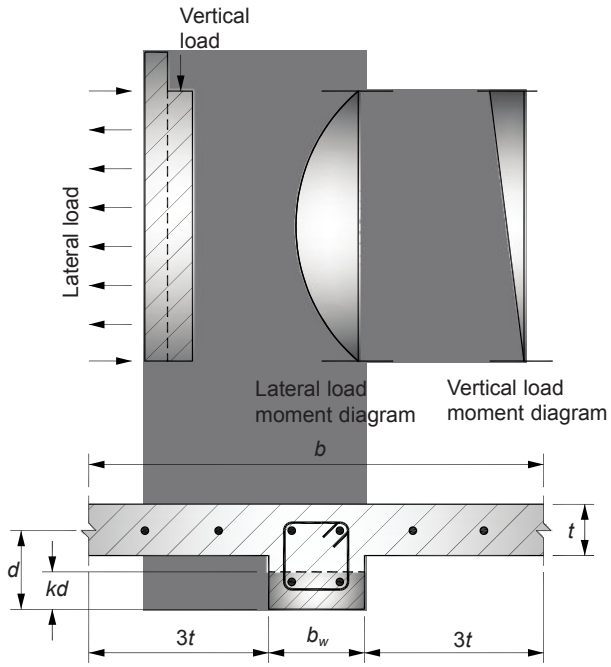


FIGURE 5.41 Wall and pilaster with loads causing tension on wall and compression on the projecting pilaster.

b. Loads causing compression on the wall and tension on the projection pilaster.

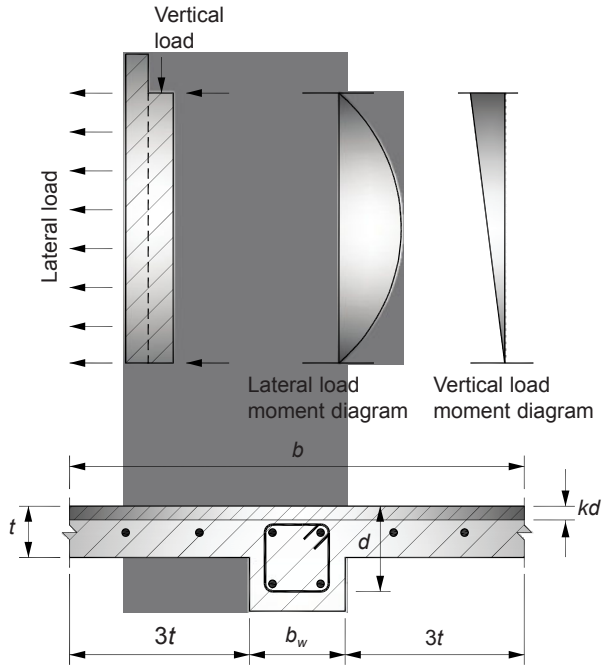


FIGURE 5.42 Wall and pilaster with lateral load causing compression on wall.

Generally the critical loading is the condition shown in Figure 5.41 where the projecting pilaster is in compression. The design for combined load and moment can be made using the methods outlined in Section 5.7 of this text.

The design of a pilaster with vertical load and lateral load can be easily accomplished by satisfying the revised unity equation.

$$\frac{P}{P_a} + \frac{f_b}{F_b} \le 1.00 \le \frac{4}{3}$$

The ratio of the actual load, P, and the maximum allowable load, P_a, is determined. The limiting masonry stress, F_b, is used to calculate the maximum allowable masonry stress, f_b, to satisfy the unity equation.

$$f_b = \left[\left(1.00 \text{ or } \frac{4}{3}\right) - \frac{P}{P_a}\right] F_b$$

5.6.2.4 FLUSH WALL PILASTERS

In order to simplify construction of a wall and to provide support of a beam, flush wall pilasters can be used. This pilaster type permits construction of a wall without projections which speeds construction and provides more floor area.

If a pilaster is to be designed as a column, then the vertical longitudinal reinforcement must be tied, as shown in Figure 5.43. For Seismic Design Category C, IBC Section 2106.4.1 states:

IBC Section 2106.4.1

2106.4.1 Design of discontinuous members that are part of the lateral-force-resisting system. Columns and pilasters that are part of the lateral-force-resisting system and that support reactions from discontinuous stiff members such as walls shall be provided with transverse reinforcement spaced at no more than one-fourth of the least nominal dimension of the column or pilaster. The minimum transverse reinforcement ratio shall be 0.0015. Beams supporting reactions from discontinuous walls or frames shall be provided with transverse reinforcement spaced at no more than one-half of the nominal depth of the beam. The minimum transverse reinforcement ratio shall be 0.0015.

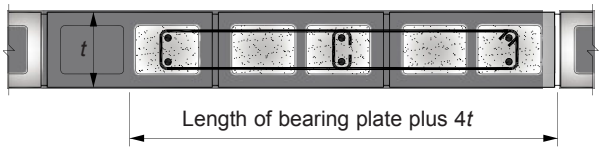


FIGURE 5.43 Flush wall pilaster designed as a column.

A flush wall pilaster can be designed as a reinforced column in which case the vertical reinforcing steel supports part of the load. However, for the steel to be considered effective, it must be tied in accordance with Section 7.14.4. The minimum dimension, which is the thickness of the wall, governs in determining the *h/t* reduction factor.

Alternately, a flush wall pilaster may be designed as a reinforced load bearing wall and the reinforcement is considered to resist only tension from lateral forces and eccentric vertical loads.

The maximum effective width of the in-the-wall columns can be considered to be the length of the bearing plate or angle plus four times the wall thickness, *t*, but not to exceed the center-to-center distance between concentrated loads, in accordance with MSJC Code Section 2.1.9.1.

5.6.3 BEARING

Base plates, beams, steel angles, and other elements which support structural elements transfer load to the masonry support. If these bearing elements cover the masonry support fully, the masonry bearing stress is limited to:

$F_{br} = 0.25f'_m$ (MSJC Code Section 2.1.9.3)

The MSJC addresses bearing stresses based upon the direct bearing area, A_1, or the supporting area, A_2, as follows:

MSJC Code Section 2.1.9

2.1.9 *Concentrated loads*

2.1.9.1 For computing compressive stress f_a for walls laid in running bond, concentrated loads shall not be distributed over the length of supporting wall in excess of the length of wall equal to the width of bearing areas plus four times the thickness of the supporting wall, but not to exceed the center-to-center distance between concentrated loads.

2.1.9.2 Bearing stresses shall be computed by distributing the bearing load over an area determined as follows:

(a) The direct bearing area A_1 , or

(b) $A_1\sqrt{A_2/A_1}$ but not more than $2A_1$, where A_2 is the supporting surface wider than A_1 on all sides, or A_2 is the area of the lower base of the largest frustum of a right pyramid or cone having A_1 as upper base sloping at 45 degrees from the horizontal and wholly contained within the support. For walls in other than running bond, area A_2 shall terminate at head joints.

2.1.9.3 Bearing stresses shall not exceed $0.25f'_m$.

The allowable bearing values are higher than the allowable axial compressive stress for walls since the load and stress rapidly dissipate throughout the wall. The compressive capacity of a wall ($F_a = 0.25f'_mR$) will control over the bearing capacity of the wall ($0.25f'_m$). The bearing capacity of columns will occasionally control over their axial compressive capacity thus mandating larger column sizes. For instance assume in Example 5-U that the entire column area was covered by a beam bearing plate. The bearing capacity of this column would only be:

$P_{br} = (f_{br})$ (area)

$$P_{br} = \frac{0.25\,(1500)}{2}(15.625)(15.625)$$

= 45,776 lbs

This capacity is much less than the applied load of 110 kips and the resulting column size would accordingly have to be increased, if based upon bearing alone.

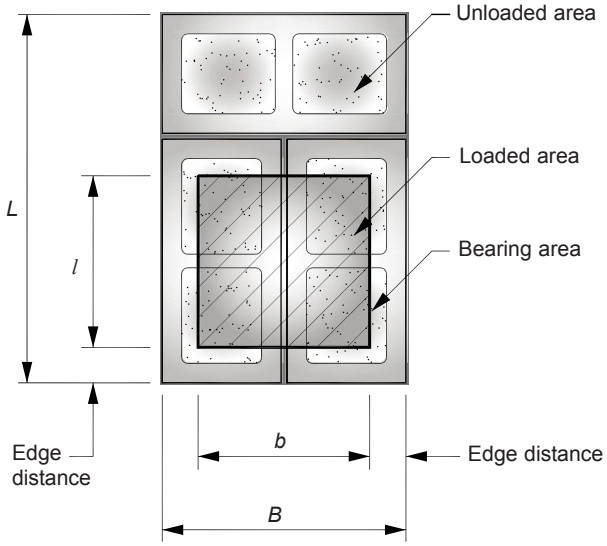


FIGURE 5.44 Relationship of bearing area.

EXAMPLE 5-V Bearing Stresses.

A 16 x 16 in. nominal masonry cross section along with an 11 x 11 in. steel bearing plate is to support a beam load, f'_m = 2000 psi. Determine the maximum load that can be put on the bearing plate.

Solution 5-V

Area of column (15.625)(15.625) = 244.1 sq in.

Area of bearing plate (11)(11) = 121 sq in.

$$\text{Ratio of areas} = \frac{121}{244.1} = 0.50$$

Allowable bearing value

Full area = 0.25 f'_m = 500 psi

Therefore, bearing capacity = (121)(500) = 60,500 lbs

Similar flush wall pilasters, the effective length over which concentrated loads are distributed from bearing plates or angles is the distance between loads or the length of bearing plate or angle plus four times the wall thickness, *t*, whichever is the least (MSJC Code Section 2.1.9.1).

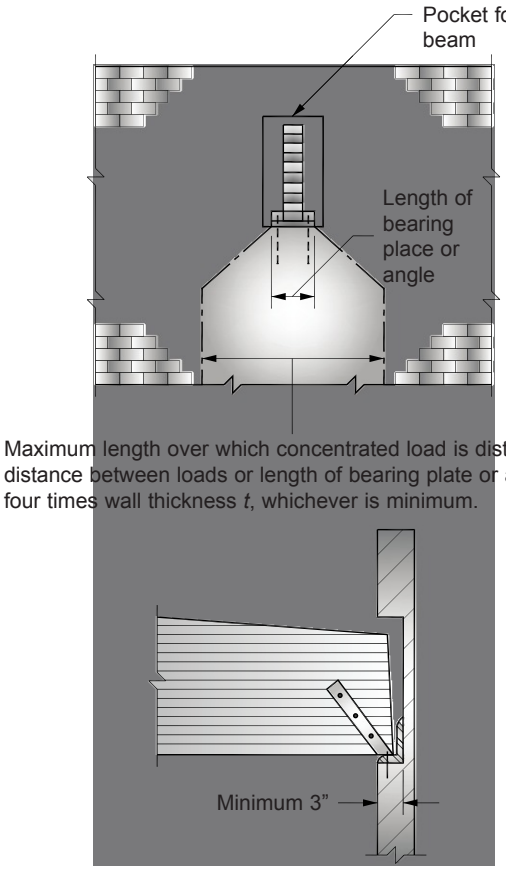


FIGURE 5.45 Distribution of concentrated loads.

The masonry element under a concentrated load (see Figure 5.45) may be designed as a column with reinforcing steel supporting some of the load or as a load bearing wall in which the steel is neglected.

5.7 COMBINED BENDING AND AXIAL LOADS

5.7.1 GENERAL

Most walls and columns are subjected to both axial and bending loads. This is particularly true of bearing walls that carry the loads of floors and roofs and are subjected to a lateral wind or earthquake force. Lateral loads may also be imposed by earth pressure on the wall.

The interaction of vertical load and bending forces will also occur if the vertical load is eccentric to the axis of the wall or column. Interaction of combined stresses may also result when a moment is imposed on the wall or column in addition to the axial load.

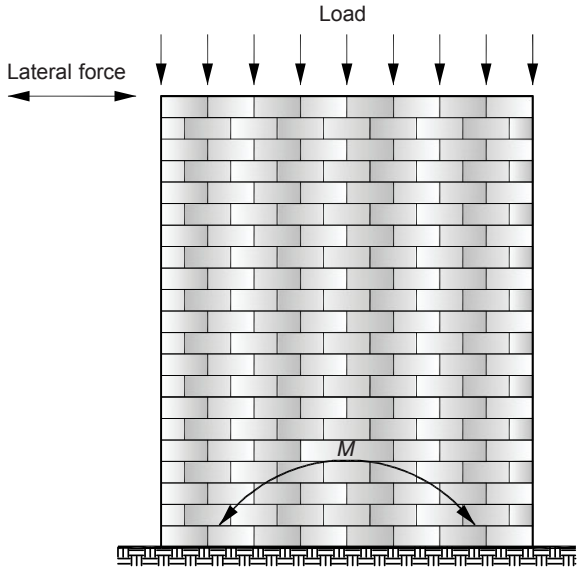


FIGURE 5.46 Combinations of loading causing combined stresses.

Interaction of these forces results in increased compressive stress on the masonry. Tension in the reinforcing steel may also occur if the moment is large enough to overcome the effect of compressive stress due to vertical load.

When a masonry wall or column is subjected to both axial load and moment or eccentric vertical load, an analysis must be made considering the combined effects of the axial and bending stresses.

Such members must be designed in accordance with accepted principles of mechanics or in accordance with the unity equation.

Interaction of load and moment on a section is complex and is represented by the curves in Figure 5.47. The unity equation, Method 1, is represented by Curve 1 and considers each stress from vertical load and moment independently. Curve 2 recognizes the capacity of the section but limits the stress to the combination of vertical stress and flexural stress. The maximum vertical stress is limited to F_a, while the maximum flexural stress is limited to F_b. Curve 2 is based on Method 2.

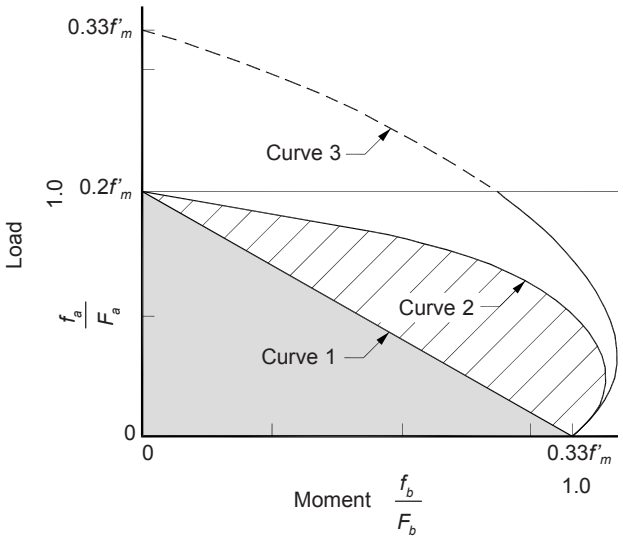


FIGURE 5.47 Graphic representation of interaction.

Curve 3 is similar to Curve 2 except the maximum stress is permitted to be $0.33f'_m$ with the axial load cut off based on $0.2f'_m$. This interaction method is based on code equations and limitations.

5.7.2 METHODS OF DESIGN FOR INTERACTION OF LOAD AND MOMENT

There are several methods by which structural elements can be designed for interaction of loads and moments, three of which are presented. Some methods are more conservative than others and the designing engineer should evaluate the methods accordingly.

Method 1. This method assumes that the vertical load and moment act independently and stresses are determined for each condition. The unity equation is checked to determine compliance.

Modified Method 1. This modification of Method 1 assumes that the vertical load counteracts the tension stress caused by the moment up to the point where the tension stress exceeds the vertical compression stress. The limiting condition for this is when $e > t/6$ or $l/6$. The initial determination of flexural stress can be by assuming a homogeneous section and using the equation $f = M/S$ or Mc/I. When the tension stress exceeds the compression stress or the allowable tension stress, consider each condition for vertical load and moment independently and proceed similar to Method 1.

Method 2. This method determines the axial stress and the maximum allowable flexural compressive stress that will satisfy the unity equation. With these values and the applied loads, the statics of the section are evaluated based on the sum of vertical forces equal 0 ($\Sigma F_v = 0$), and the sum of moments equal to zero ($\Sigma M = 0$). The stress in the steel is calculated and the required area of steel determined.

These equations were developed by Ralph McLean, structural engineer, of the firm McLean and Schultz, Consulting Engineers, Architect and Planners of Fullerton, California.

Method 3. This method assumes that the section is homogeneous and uncracked. The stresses are determined by $P/A \pm Mc/I$ with the moment of inertia based on the gross section. If P/A axial compressive stress is less than the flexural stress, Mc/I, then there will be tension on the section and it must be reinforced for this tension force.

The axial and flexural stresses as determined by P/A and Mc/I must be checked against the maximum allowable stresses to assure compliance with the unity equation.

5.7.2.1 UNITY EQUATION

The classic approach to the interaction of load and moment is the code unity equation. This approach limits the ratio of the actual axial stress to the maximum allowable compressive stress, plus the actual flexural stress to the maximum allowable flexural stress, to 1.00.

The combination of stresses may not exceed the unity equation:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1 \qquad \text{(for Walls)}$$

$$\frac{P}{P_a} + \frac{f_b}{F_b} \le 1 \qquad \text{(for Columns)}$$

Where:

f_a = computed actual axial unit stress due to the load determined from total axial load and effective area:

$= \frac{P}{bt}$ (psi) (for Walls)

bt = actual cross-sectional solid area of wall (sq in.)

F_a = maximum allowable axial stress if the member were carrying axial load only (psi)

= (0.25 f'_m psi) (R) (for Walls)

$R = \frac{h}{r}$ (psi), reduction factor

$= \left[1 - \left(\frac{h}{140r}\right)^2\right]$ for $\frac{h}{r} \le 99$

$= \left(\frac{70r}{h}\right)^2$ for $\frac{h}{r} > 99$

P = actual load on column

$$P_a = (0.25f'_m A_e + 0.65A_sF_{sc})\left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \text{for } \frac{h}{r} \le 99$$

$$P_a = (0.25f'_m A_e + 0.65A_sF_{sc})\left(\frac{70r}{h}\right)^2 \quad \text{for } \frac{h}{r} > 99$$

F_b = maximum allowable flexural stress if members were carrying bending load only

$= \frac{1}{3} f'_m$ (psi)

f_b = actual computed bending stress

In the case of temporary loads, due to wind or earthquake, MSJC Code Section 2.1.2.3 allows a one third increase. An example of this is moment on a wall caused by wind:

$$\frac{\text{Vertical } f_a}{\text{allowable } F_a} + \frac{\text{wind } f_b}{\text{allowable } F_b} \le \frac{4}{3} \text{ (for Walls)}$$

$$\frac{P}{P_a} + \frac{f_b}{F_b} \le \frac{4}{3} \text{ (for Columns)}$$

This is a simple and acceptable technique provided the resulting design is not less than the design determined using only dead and live loads.

5.7.2.1.1 UNCRACKED SECTION

The cross-section of the element is uncracked when the vertical stress is equal to or more than the flexural stress. This occurs when the eccentricity, e, of the load, P, is less than or equal to the kern distance.

Note: Define e_k:

$$e_k = \frac{l}{6} \text{ or } \frac{t}{6} \qquad \text{(rectangular section)}$$

$$e_k = \frac{I}{A_y} = \frac{r^2}{y} \qquad \text{(irregular section)}$$

Where:

I = moment of inertia of section

A = area section

y = distance from neutral axis to extreme edge

r = radius of gyration, or $\sqrt{\frac{I}{A}}$

The stress can also be determined by the equation

$$f_a = \frac{P}{bt};\ f_b = \frac{Mc}{I} = \frac{6M}{bt^2}$$

$$f = \frac{P}{A} \pm \frac{Mc}{I} = \frac{P}{bt} \pm \frac{6M}{bt^2}$$

When f_a is greater than or equal to f_b the section is always under compression.

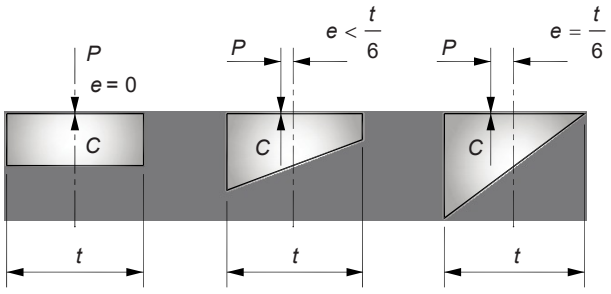


FIGURE 5.48 Wall under compression.

5.7.2.1.2 CRACKED SECTION

If the virtual eccentricity is greater than the kern distance, there is tension on the face of the wall. Since the masonry in reinforced masonry is assumed not to resist tension, then the section is to be reinforced to resist the tension as if there was no vertical force to reduce it. This is a good approximation when the steel is located within the middle third of the wall. The design condition is depicted in Figure 5.49.

If credit is given to the tension bond between the mortar and the masonry unit, the comparative distance e_k may be increased from t/6 to t/5, or t/4 depending on the value given to the tension bond.

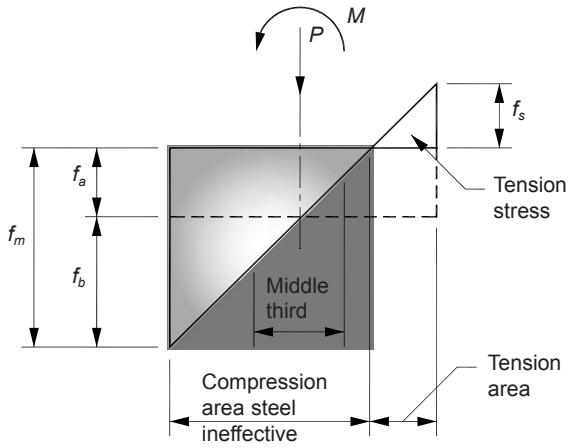


FIGURE 5.49 Wall under combined stresses with flexural stress exceeding axial stress.

When the eccentricity exceeds t/6 or l/6 and the tension capacity of the masonry is ignored, the section may be under compression only until it becomes necessary to provide reinforcing steel to resist tension forces. This condition of compression stress only may be assumed similar to an eccentrically loaded footing, which is capable of imposing only compression forces. (Figure 5.50b).

The limit of the condition where only compression forces exist is when the eccentric load is no longer coincidental with the resultant of the compression force in the stress block and the allowable compression stress on the masonry is not exceeded.

If the force in the reinforcing steel is to be included in the evaluation for the sum of moments and sum of forces, the assumed masonry compressive stress may need to be reduced, thus decreasing the eccentricity of the resultant compression in the masonry.

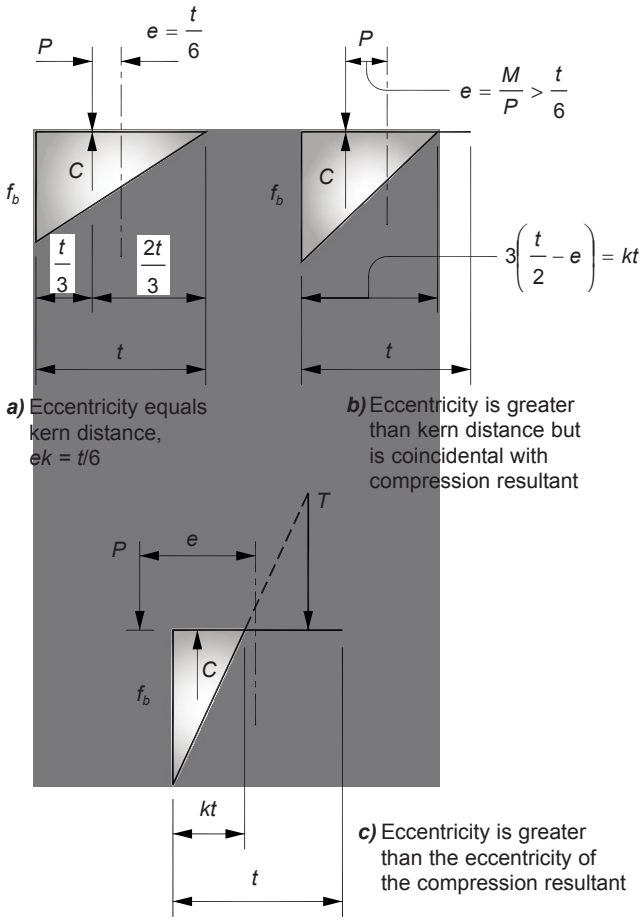


FIGURE 5.50 Conditions of increasing eccentricity of load on wall.

The resultant compression force will be balanced, $\Sigma F_v = 0$, by the eccentric vertical load and the tension force in the steel. See Figure 5.50c.

The maximum compressive stress on the masonry is determined based on satisfying the unity equation:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = 1.00 \text{ or } \frac{4}{3}$$

$$f_m = f_a + f_b$$

EXAMPLE 5-X Combined Loading:

Determine whether steel is required for tension in an 8 in. concrete masonry wall which is 13 ft 4 in. high and subjected to a wind pressure of 30 psf. f'_m = 1500 psi, n = 21.5, F_s = 24,000 psi, Vertical load, P = 4000 plf and Distance to steel, d = 5.3 in. Assume steel @ 32 in. o.c. (r = 2.59 in.).

Solution 5-X

Moment perpendicular to wall due to wind, M:

$$M = \frac{wl^2}{8} = \frac{30(13.33)^2}{8} = 667 \text{ ft lbs/ft}$$

Virtual eccentricity $e = \frac{M}{P} = \frac{(667)(12)}{4000} = 2 \text{ in.}$

Kern distance $e_k = \frac{t}{6} = \frac{7.63}{6} = 1.27 \text{ in.} < 2 \text{ in.}$

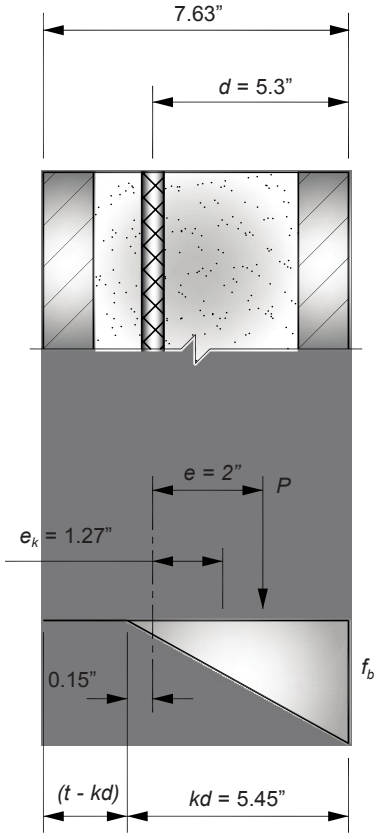
Eccentricity exceeds kern distance

Length of compression area = $3\left(\frac{t}{2} - e\right)$

$$= 3\left(\frac{7.63}{2} - 2\right) = 5.45 \text{ in.}$$

This indicates that (7.63 - 5.45) = 2.18 in. of wall will have no stress on it and that steel located 5.3 in. from the compression face would not be stressed in tension.

Actual compression stress due to eccentric vertical load:



Actual maximum compression stress;

$$C = \frac{1}{2} f_b bkd$$

$$f_b = \frac{2C}{bkd} = \frac{2(4000)}{(12)(5.45)} = 122 \text{ psi}$$

Actual axial stress

$$f_a = \frac{P}{bt} = \frac{4000}{(12)(5.45)} = 61.2 \text{ psi}$$

Maximum allowable axial compression stress

$$F_a = 0.25 f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right]$$

(Note: $h = h'$ for the notation for effective height throughout)

$$= 0.25(1500)\left[1 - \left(\frac{(13.33)(12)}{(140)(2.59)}\right)^2\right]$$

= 302 psi > 61.2 psi O.K.

Check the interaction equation

$$F_b = \frac{1}{3} f'_m$$

= 1500/3

= 500 psi > 122 psi O. K.

Allowable bending stress

Determine the maximum allowable flexural compression stress by using the Unity Equation

$$f_b = F_b\left(\frac{4}{3} - \frac{f_a}{F_a}\right)$$

$$= 500\left(\frac{4}{3} - \frac{61.2}{302}\right)$$

= 563.7 psi > 122 psi O.K.

$f_m = f_a + f_b = 61.2 + 122 = 183.2$ psi

563.7 psi > 183.2 psi O.K.

$\frac{4}{3}(302) = 401.7$ psi > 183.2 psi

The eccentric vertical load *P* is coincidental with the resultant compressive force *C*. No tension steel is required. Provide minimum steel as required by code.

EXAMPLE 5-Y Steel Requirement.

Using Example 5-X, check the requirement for tension steel for the wind load only.

Solution 5-Y

Design reinforcement for lateral wind load moment of 667 ft-lbs/ft; d = 5.3 in.

$$K_f = \frac{M}{bd^2} = \frac{(667)(12)}{(12)(5.3)^2}$$

= 23.7 due to wind

From Table ASD-24b for K_f = 23.7

read ρ = 0.0008

$A_s = \rho bd = 0.0008(12)(5.3)$

= 0.051 sq in./ft

Check against minimum A_s requirement

A_s = 0.0013bt

= 0.0013(12)(7.63)

= 0.119 sq in./ft ← Controls

Use #5 @ 32 in. o.c. (A_s = 0.31(12/32) = 0.116 provided, reasonably close)

There is no tension on the wall and only minimum required reinforcing steel is needed.

5.7.3 METHOD 1. VERTICAL LOAD AND MOMENT CONSIDERED INDEPENDENTLY

The Method 1 analysis for interaction, particularly when the moment is perpendicular to the plane of the wall, is to consider each force independently. Stress for the vertical load is calculated and then stress due to the moment based on a cracked section is calculated. The combination of compressive stresses should not exceed the unity equation.

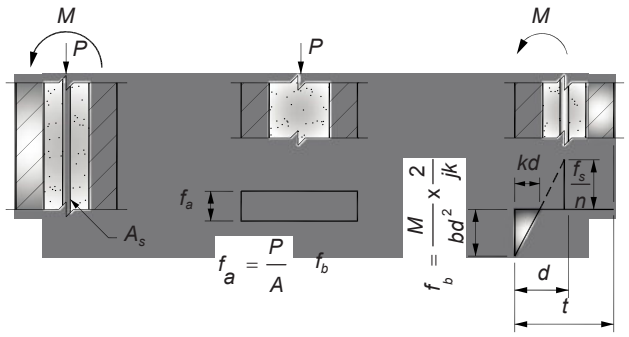


FIGURE 5.51 Unity equation assumed stress distribution; bending stress greater than axial compressive stress; $f_a < f_b$.

This handbook presents a direct method of designing a wall subjected to load and bending without the need to make assumptions for the amount of steel and then checking all stresses. The amount of reinforcement, if needed, can be directly determined for a wall subjected to bending perpendicular to the plane of the wall. Calculate or assume:

M; P; f'_m; b (normally 12 in.); F_s

t (wall thickness); d (distance from compression face to center of steel); and h' (effective or actual height of wall

Solve for

1. Kern distance, $e_k = \frac{t}{6}$ or $e_k = \frac{I}{A_y} = \frac{S}{A}$

2. Virtual eccentricity, $e = \frac{M}{P}$

3a. If $e < e_k$ minimum reinforcement required

3b. If $e > e_k$ design for bending stress

4a. Actual axial stress, $f_a = \frac{P}{bt}$

Note: Use actual cross-sectional area of masonry. For partially grouted walls use Table GN-3a to find equivalent solid thicknesses (*EST*).

4b. Flexural stress assuming uncracked section

$$f_b = \frac{Mc}{I} = \frac{M}{S} = \frac{6M}{bt^2}$$

4c. If $f_a > f_b$, section under compression minimum reinforcement required, see condition 3a.

If $f_a < f_b$, section under tension, design reinforcement for flexural stress, see condition 3b.

However, if tensile stress does not exceed the allowable tensile stress for plain masonry, Table GN-32, only minimum steel need be used.

(See Tables ASD-9a, ASD-9b and ASD-9c)

5. $\frac{h}{r}$ reduction factor, $R = \left[1 - \left(\frac{h}{140r}\right)^2\right]$ for $\frac{h}{r} \le 99$

$$= \left(\frac{70r}{h}\right)^2 \text{ for } \frac{h}{r} > 99$$

6. Maximum allowable axial stress

$F_a = 0.25\, f'_m R$

7. Ratio of axial stresses $\frac{f_a}{F_a}$.

8. Maximum allowable flexural compression stress

$$F_b = \frac{1}{3} f'_m$$

9. Maximum allowable flexural compression stress that will satisfy the unity equation

$$f_b = \left(1 - \frac{f_a}{F_a}\right) F_b \text{ or } f_b = \left(\frac{4}{3} - \frac{f_a}{F_a}\right) F_b$$

if loads are temporary such as wind or earthquake.

10. Compute the flexural coefficient,

$$K_f = \frac{M}{bd^2} \text{ or } K_f = \frac{12{,}000M}{bd^2}$$

11. With K_f from Step 10 and f_b from Step 9, determine the steel ratio, ρ, using Diagrams ASD-23 through ASD-33.

12. With steel ratio ρ determined from above and the given d, from Tables GN-20c and GN-20d select the reinforcing bars and spacing.

EXAMPLE 5-Z Load and Moment on Brick Wall.

A 9 in. solid grouted reinforced clay masonry wall supports a vertical load of 9200 plf and a moment of 1625 ft-lbs/ft due to earth load.

Design the required steel if d = 5 in. and the effective height of wall = 10 ft 6 in.

Solution 5-Z

Assume f'_m = 1500 psi

n = 27.6; F_s = 24,000 psi

1. Kern distance, $e_k = \frac{t}{6} = \frac{9}{6} = 1.5$ in.

2. Virtual eccentricity, $e = \frac{M}{P} = \frac{(1625)(12)}{9200}$

= 2.12 in.

3. $e > e_k$, therefore there is tension on section, assume cracked

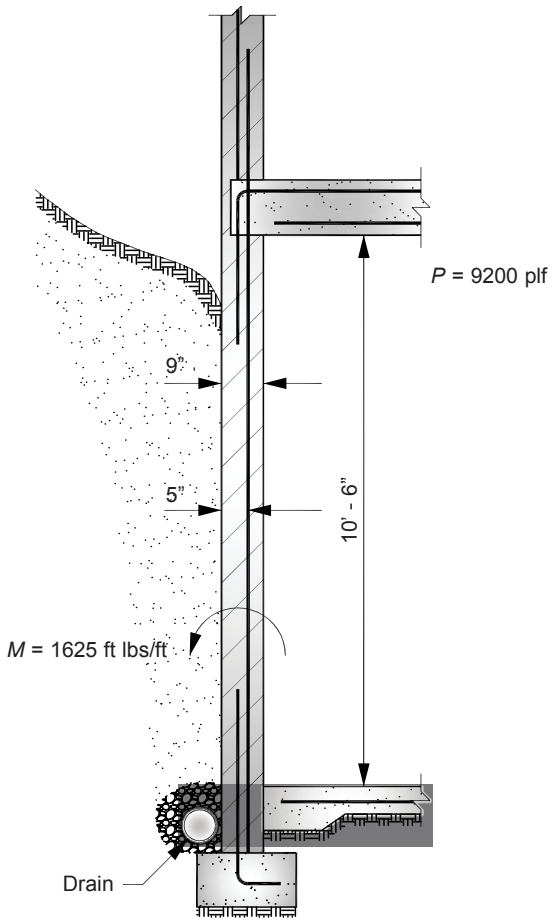


FIGURE 5.52 Cross section of clay masonry wall with loads shown.

4. Actual axial compression stress

$$f_a = \frac{P}{bt} = \frac{9200}{(12)(9)} = 85.2 \text{ psi}$$

Check for $f_b = \frac{6M}{bt^2} = \frac{6(1625)(12)}{(12)(9)^2} = 120.4$ psi

Since the tensile stress of 120.4 psi exceeds the compression stress, 85.2 psi, assume section is cracked. It also exceeds allowable tension for unreinforced masonry; f_t = 25 psi if ungrouted (MSJC Code Table 2.2.3.2). Also see Table ASD-10. However, for a fully grouted wall, the limit of 65 psi would be acceptable. The remainder will be used as a further illustration of the procedure when the tensile bond strength is not acceptable.

5. $r = \sqrt{\frac{I}{A}} = \sqrt{\frac{(12)(9)^3/12}{(12)(9)}} = 2.60$

$\frac{h}{r} = \frac{(10.5)(12)}{2.60} = 48.5$

Enter Table ASD-9b, R = 0.880

6. Use Table ASD-9b to find allowable axial stress,

F_a = 330 psi

7. Ratio of axial stress $\frac{f_a}{F_a} = \frac{85.2}{330} = 0.258$

8. Maximum allowable flexural compression stress

$F_b = \frac{1}{3}f'_m = \frac{1}{3}(1500) = 500$ psi Table ASD-3

9. Maximum allowable flexural compression stress to satisfy the unity equation

$f_b = \left(1 - \frac{f_a}{F_a}\right)F_b = (1 - 0.258)500 = 371$ psi

10. The flexural coefficient, $K_f = \frac{M}{bd^2}$ for

b = 12 in., d = 5 in. and M = 1625 ft-lbs/ft

$K_f = \frac{M}{bd^2} = \frac{(1625)(12)}{(12)(5)^2} = 65$

11. In Diagram ASD-24a, K_f vs ρ for n = 27.6

Enter K_f = 65 move right to intersect

f_b = 371 psi

Move down and read ρ = 0.0052

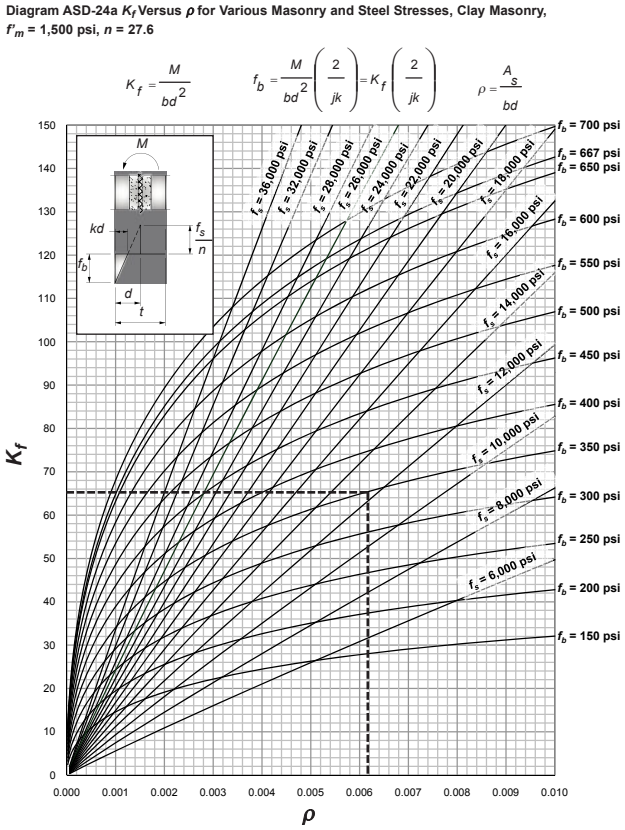


Diagram ASD-24a

12. Select reinforcing steel

$A_s = \rho bd$ = 0.0052(12)(5) = 0.312 sq in./ft

See Table GN-20c

Use #5 at 12 in. o.c. (A_s = 0.31 sq in./ft)

or #6 at 16 in. o.c. (A_s = 0.33 sq in./ft)

Spacing of vertical reinforcement in double wythe walls is acceptable at non-eight inch modules.

Alternate method to determine steel requirement:

After determining the maximum allowable flexural compressive stress that will satisfy the unity of equation as shown in step 9.

$f_b = \left(1 - \frac{f_a}{F_a}\right)F_b$

or $f_b = \left(\frac{4}{3} - \frac{f_a}{F_a}\right)F_b$ For wind or seismic loads

Equate to flexural formula

$f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right)$

$$\frac{2}{jk} = f_b \frac{bd^2}{M}$$

Solve for $\frac{2}{jk}$

From Table ASD-34a or ASD-34b for $\frac{2}{jk}$; read $n\rho$

Solve for $\rho = \frac{n\rho}{n}$

Solve for $A_s = \rho bd$

From Example 5-Z, Step 9, f_b = 371 psi

Solve for $\frac{2}{jk}$

$$\frac{2}{jk} = \frac{(371)(12)(5)^2}{(1625)(12)}$$

= 5.7

From Table ASD-34b, for $\frac{2}{jk} = 5.7$

read $n\rho$ = 0.138 ρ = 0.005

and $A_s = \rho bd$ = (0.005) (12) (5)

= 0.30 in.^{2}/ft required

Use #5 at 12 in. o.c. or #6 at 16 in. o.c.

EXAMPLE 5-AA Method 1 Determination of Reinforced Clay Masonry Column with an Eccentric Load.

Design a hollow clay masonry column, 12 ft high, to support a live load of 8 kips and a dead load of 10 kips. The loads have an eccentricity of 6 in. from the center line of the column. Column is located in SDC B.

Assume: f'_m = 2500 psi, f_y = 60,000 psi, n = 16.6

Solution 5-AA

Maximum allowable axial column stress on masonry is:

$F_a = 0.25 f'_m R$

For initial design assume R = 0.95

F_a = (0.25)(2500) 0.95 = 594 psi

Initial column area

$$A_s = \frac{(8+10)1000}{594} = 30.3 \text{ sq in.}$$

Use two hollow clay masonry units, 6 in. x 4 in. x 12 in. (nominal), 5½ in. x 3½ in. x 11½ in. (actual)

A_e = (11.5)(11.5) = 132 sq in.

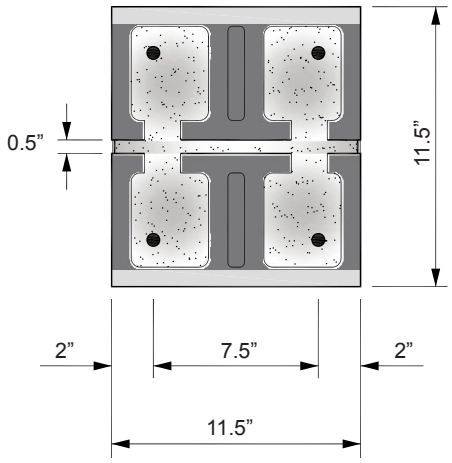


FIGURE 5.53 Cross section of hollow clay masonry column.

Use minimum area of vertical steel ρ = 0.005

$A_s = \rho bt$

= 0.005(11.5)(11.5)

= 0.66 sq in.

Try 4 - #5 bars

A_s = 1.24 sq in. (excess steel for moment consideration)

Check reduction coefficient, R; radius of gyration, r

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{(11.5)^4/12}{(11.5)^2}} = 3.32$$

$$\frac{h}{r} = \frac{(12)(12)}{3.32} = 43.4 < 99$$

$$R = \left[1 - \left(\frac{(12)(12)}{(140)(3.32)}\right)^2\right] = 0.90$$

Maximum allowable load with 4 - #5 bars

$$P_a = (0.25f'_m A_e + 0.65A_s F_{sc})\ R$$

$$= [0.25\ (2500)\ (11.5)^2 + 0.65\ (1.24)\ (24{,}000)]\ (0.90)$$

$$= 91{,}800 \text{ lbs}$$

Ratio of vertical loads,

$$\frac{P}{P_a} = \frac{18{,}000}{91{,}800} = 0.196$$

Based on unity equation the maximum allowable flexural compression masonry stress is:

$$f_b = \left(1 - \frac{P}{P_a}\right) F_b$$

$$F_b = \frac{1}{3}(2500)$$

$$= 833 \text{ psi}$$

$$f_b = (1 - 0.196)(833)$$

$$= 670 \text{ psi}$$

Determine the area of reinforcing steel required for the moment and limiting stress condition by the $n\rho j$ method.

Moment due to eccentric load

$$M = (8{,}000 + 10{,}000)(6)$$

$$= 108{,}000 \text{ in.-lbs}$$

Solve for $\frac{2}{jk}$

$$\frac{2}{jk} = \frac{bd^2 f_b}{M}$$

$$= \frac{(11.5)(9.5)^2(670)}{108{,}000} = 6.44$$

From Table ASD-34a for $\frac{2}{jk} = 6.44$

$n\rho = 0.0955$

Solve for $n\rho j$

$$n\rho j = \frac{nM}{bd^2 f_s}$$

$$= \frac{(16.6)(108{,}000)}{(11.5)(9.5)^2(24{,}000)}$$

$$= 0.072$$

From Table ASD-34a for $n\rho j = 0.072$

$n\rho = 0.081$

Use maximum $n\rho$ value, masonry controls

$$\rho = \frac{n\rho}{n} = \frac{0.0955}{16.6} = 0.0058$$

$A_s = \rho bd$

$= 0.0058(11.5)(9.5) = 0.63$ sq in.

Use 2 - #5 bars on each side; $A_s = 0.62$ sq in.

Alternate Solution:

Flexural coefficient $K_f = \frac{M}{bd^2} = \frac{108{,}000}{(11.5)(9.5)^2} = 104.1$

From Diagram ASD-26a

for $K_f = 104.1$ and $f_b = 670$ psi,

Read $\rho = 0.0062$ (approximately same as above), $A_s = 0.68$ sq in.

Ties in column

No special conditions since moment is not due to seismic forces.

From Table ASD-88

Use 1/4 in. ties at 12 in. o.c.

Closer tie spacing (8 in.) would be prescriptively required for Seismic Design Categories D, E, and F

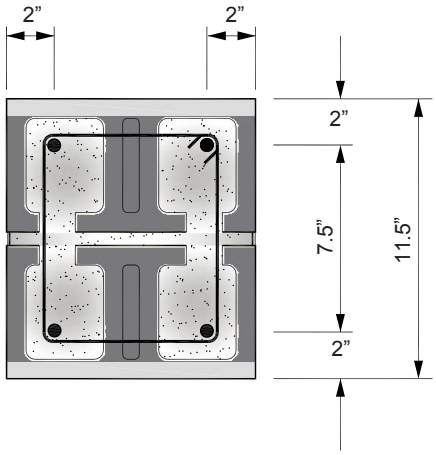


FIGURE 5.54 Cross section of column showing reinforcement and ties.

5.7.4 METHOD 2. EVALUATION OF FORCES BASED ON STATIC EQUILIBRIUM OF $\Sigma F_v = 0$ AND $\Sigma M = 0$

Given:

Length of wall = l in.

Thickness of wall = t in.

Distance to steel = d in.

Distance to steel = d_1 in.

Axial load = P pound or w plf

Compression force = C pounds

Tension force = T pounds

Moment = M foot pounds

Steel stress = f_s psi

Masonry stress = f_m psi

Height of wall = h ft

Compression force, $C = \frac{1}{2}tkdf_b$

Tension force, $T = C - P$

Radius of gyration = r in.

Taking the sum of the moments about the center line or axis of the vertical load:

$$C\left(\frac{l}{2} - \frac{kd}{3}\right) + T\left(\frac{l}{2} - d_1\right) - M = 0$$

and $T = C - P$

$$C\left(\frac{l}{2} - \frac{kd}{3}\right) + (C - P)\left(\frac{l}{2} - d_1\right) - M = 0$$

substituting for $C = \frac{1}{2}tkdf_m$

$$= \left(\frac{1}{2}tkdf_m\right)\left(\frac{l}{2} - \frac{kd}{3}\right) + \left(\frac{1}{2}tkdf_m - P\right)\left(\frac{l}{2} - d_1\right) - M = 0$$

$$\frac{1}{4}tf_m lkd - \frac{1}{6}tf_m(kd)^2 + \frac{1}{2}tkd\left(\frac{l}{2} - d_1\right)f_m$$

$$- P\left(\frac{l}{2} - d_1\right) - M = 0$$

$$\frac{1}{4}tf_m lkd - \frac{1}{6}tf_m(kd)^2 + \frac{1}{4}tf_m lkd - \frac{1}{2}tf_m d_1 kd$$

$$- P\left(\frac{l}{2} - d_1\right) - M = 0$$

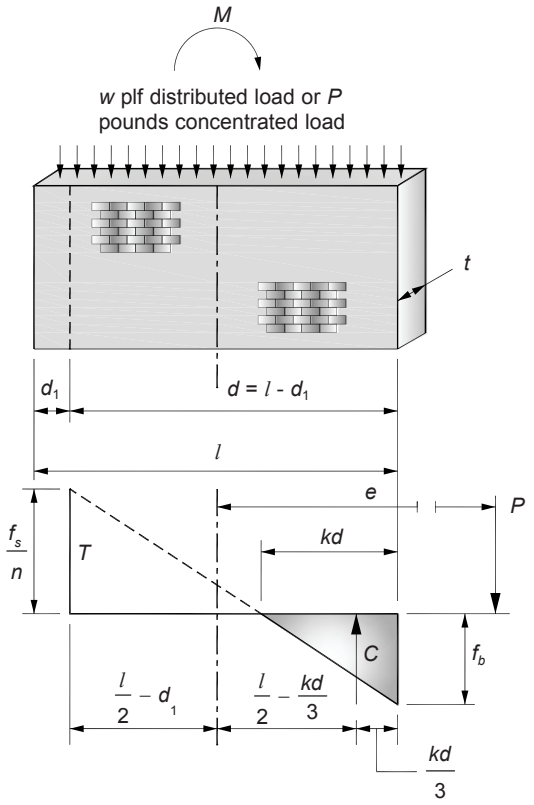


FIGURE 5.55 Load and moment on wall.

Change signs and combine terms

$$\frac{1}{6}tf_m(kd)^2 - \frac{1}{2}tf_m(l - d')kd + P\left(\frac{l}{2} - d_1\right) + M = 0$$

Solving this quadratic equation

$$\mathbf{a}x^2 + \mathbf{b}x + \mathbf{c} = 0$$

let $x = kd$

$$\mathbf{a} = \frac{1}{6}tf_m$$

$$\mathbf{b} = -\frac{1}{2}tf_m(l - d_1)$$

$$\mathbf{c} = P\left(\frac{l}{2} - d_1\right) + M$$

but $d = l - d$, so the equation for b simplifies to:

$$\mathbf{b} = -\frac{1}{2}tf_m d_1$$

Using the binominal formula to solve the quadratic equation,

$$kd = \frac{-\mathbf{b} \pm \sqrt{\mathbf{b}^2 - 4\mathbf{ac}}}{2\mathbf{a}}$$

Note:

The term $\frac{-\mathbf{b}}{2\mathbf{a}} = \frac{\frac{1}{2}tdf_m}{2\left(\frac{1}{6}tf_m\right)}$

= 1.5*d* would result in a negative *kd* distance

$$kd = \frac{-\mathbf{b} - \sqrt{\mathbf{b}^2 - 4\mathbf{ac}}}{2\mathbf{a}}$$

$$kd = \frac{\frac{1}{2}tf_m d - \sqrt{\left(\frac{1}{2}tf_m d\right)^2 - 4\left(\frac{1}{6}tf_m\right)\left[P\left(\frac{l}{2} - d_1\right) + M\right]}}{2\left(\frac{1}{6}tf_m\right)}$$

Determine the maximum allowable masonry stress, f'_m.

$$f_a = \frac{P}{l\,t}$$

$$F_a = 0.25f'_m\left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ if } \frac{h}{r} \le 99$$

or

$$F_a = 0.25f'_m\left(\frac{70r}{h}\right)^2 \text{ if } \frac{h}{r} > 99$$

$$F_b = \frac{1}{3}f'_m$$

$$f_b = F_b\left(1.33 - \frac{f_a}{F_a}\right)$$

$f_m = f_a + f_b$

$C = {}^1/_2tkdf_m$

$T = C - P$

$$k = \frac{kd}{d}$$

$$f_s = \left(\frac{1-k}{k}\right)nf_m \text{ or}$$

f_s = allowable steel stress plus allowable increases.

$$A_s = \frac{T}{f_s}$$

If f_s exceeds allowable F_s, decrease f_m, and re-compute values.

EXAMPLE 5-AB Determine the Reinforcement for a Shear Wall (Method 2).

An 8 in. concrete masonry shear wall in a high rise building is subjected to a vertical load, *P* of 845 kips and a seismic moment *M* of 5840 ft-kips. The wall is 9 ft 4 in. between floors, 30 ft long and d_1 is assumed 8 in. f'_m = 3000 psi, n = 10.7, r = 2.66, h/r < 99. Assume prescriptive SDC requirements are satisfied.

Solution 5-AB

$$f_a = \frac{P}{l\,t} = \frac{(845)(1000)}{(12)(30)(7.63)}$$

= 308 psi

$$F_a = 0.25(3000)\left[1 - \left(\frac{(12)(9.33)}{(140)(2.66)}\right)^2\right]$$

= 750 (0.910)

= 682 psi

$$F_b = \frac{1}{3}f'_m$$

F_b = 0.333 (3000) = 1000 psi

$$f_b = 1000\left[1.33 - \left(\frac{308}{682}\right)\right]$$

= 878 psi

$f_m = f_a + f_b$

= 308 + 878 = 1186 psi maximum

= 1.186 ksi

Solve values *kd*, f_s, *C*, *T* and A_s

$\mathbf{a} = tf_m/6$

= (7.63)(1.2)/6

= 1.52

$$\mathbf{b} = -\frac{1}{2}tf_m(l - d_1)$$

$$= -\frac{1}{2}(7.63)(1.2)(360 - 8)$$

= -1611

$$\mathbf{c} = P\left(\frac{l}{2} - d_1\right) + M$$

$$= 845\left(\frac{360}{2} - 8\right) + (5840)(12)$$

$$= 215{,}420$$

$$kd = \frac{-\mathbf{b} - \sqrt{\mathbf{b}^2 - 4\mathbf{ac}}}{2\mathbf{a}}$$

$$= \frac{+1611 - \sqrt{(-1611)^2 - 4(1.52)(215{,}420)}}{2(1.52)}$$

$$= 157 \text{ in.}$$

$$k = \frac{kd}{d} = \frac{157}{360 - 8}$$

$$= 0.446$$

$$C = \frac{1}{2} tkdf_m$$

$$= \frac{1}{2}(7.63)(157)(1.2)$$

$$= 719 \text{ kips}$$

$$T = C - P$$

$$= 719 - 845 = -126 \text{ kips}$$

The negative sign indicates that no tension reinforcing steel is required and the eccentric axial load can be coincidental with the resultant compression force.

Virtual eccentricity $e = \frac{M}{P} = \frac{(5840)(12)}{845}$

$$= 83 \text{ in.}$$

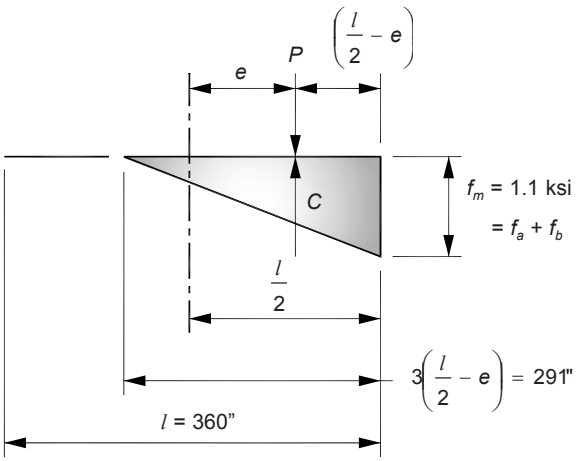
$$\frac{l}{2} - e = \frac{360}{2} - 83$$

$$= 97 \text{ in.}$$

$$3\left(\frac{l}{2} - e\right) = 3(97)$$

$$= 291 \text{ in.} > kd = 157$$

No tension steel required



Use minimum steel

A_s = 0.0013 bt/2 (each side)

= 0.0013(360)(7.63)/2

= 1.79 sq in.

Use 2 - #9 bars each side.

A_s = 2.00 sq in.

and #4 at 48 in. o.c. as prescriptive steel in balance of wall

Note: The stress in the masonry will actually be less than the maximum allowable stress of 1.2 ksi. Then the stress block will be 291 in. and the applied eccentric load, P, will be colinear with the resultant force C.

EXAMPLE 5-AC Overturning Steel in a Wall (Method 2).

Determine the overturning steel for the wall shown.

Given:

Wall thickness nominal 10 in. CMU

t = 9.63 in.

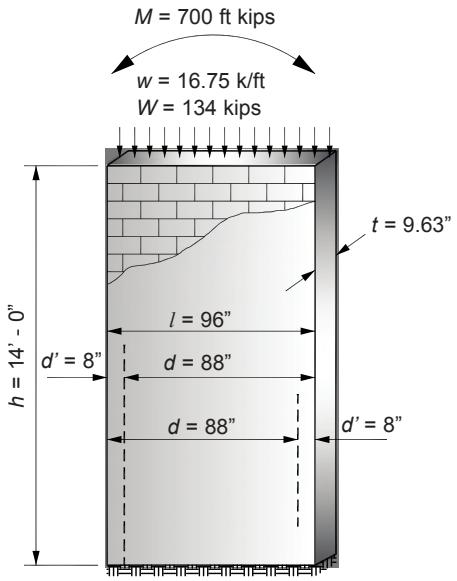
f'_m = 3000 psi

n = 10.7

F_s = 24,000 psi

Moment due to seismic forces

r = 2.77, h/r < 99



Solution 5-AC

Actual axial stress

$$f_a = \frac{P}{tl} = \frac{134}{(9.63)(96)}$$

= 0.145 ksi

$$F_a = (0.25)(3)\left[1 - \left(\frac{(12)(14)}{(140)(2.77)}\right)^2\right]$$

= 0.609 ksi

F_b = 1.0 ksi

$$f_b = F_b\left(\frac{4}{3} - \frac{f_a}{F_a}\right)$$

$$= 1.0\left(\frac{4}{3} - \frac{0.145}{0.609}\right)$$

= 1.09 ksi

$f_m = f_a + f_b$

= 0.145 + 1.09

= 1.235 ksi

Solve for kd, f_s, C, T and A_s

$\mathbf{a} = tf_m/6$

= (9.63)(1.235)/6

= 1.98

$\mathbf{b} = -tf_m(l - d_1)/2$

= -(9.63)(1.235)(96 - 8)/2

= -523

$$\mathbf{c} = P\left(\frac{l}{2} - d_1\right) + M$$

$$= 134\left(\frac{96}{2} - 8\right) + (700)(12)$$

= 13,760

$$kd = \frac{-\mathbf{b} - \sqrt{\mathbf{b}^2 - 4\mathbf{ac}}}{2\mathbf{a}}$$

$$= \frac{+523 - \sqrt{(523)^2 - 4(1.98)(13760)}}{2(1.98)}$$

= 29.6 in.

$$C = \frac{1}{2}tkdf_m$$

$$= \frac{1}{2}(9.63)(29.6)(1.235)$$

= 176.0 kips

$T = C - P$

= 176.0 - 134

= 42.0 kips

$$k = \frac{kd}{d} = \frac{29.6}{(96 - 8)} = 0.336$$

$$f_s = \left(\frac{1 - k}{k}\right)nf_m$$

$$= \left(\frac{1 - 0.336}{0.336}\right)(10.7)(1.235)$$

= 26.1 ksi

$$A = \frac{T}{f_s} = \frac{42.0}{26.1} = 1.61 \text{ sq in.}$$

Use 2 - #8 bars each end (A_s = 1.58 sq in.) (Location may be one in each of first two cells for constructability).

5.7.5 METHOD 3. SECTION ASSUMED HOMOGENEOUS FOR COMBINED LOADS, VERTICAL LOAD WITH BENDING MOMENT PARALLEL TO WALL

Walls and piers which resist forces parallel to the wall are subjected to overturning moments. The vertical load and the overturning moment cause combined stresses on the wall or pier. These overturning moments may be caused by wind, seismic or other lateral forces.

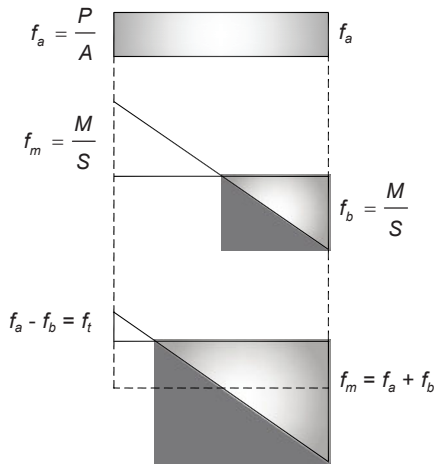
a. If the compressive stress, f_a, due to vertical load exceeds the flexural tension stress, f_b, due to overturning moment, the section is under compression and only minimum jamb steel is required.

b. If the tension stress due to the overturning moment exceeds the compression stress due to vertical load, determine the total net tension force and provide reinforcing steel to accommodate the tension force.

This method has been presented in the National Concrete Masonry Association Design Manual, *The Application of Reinforced Concrete Masonry Load Bearing Walls in Multi-Story Structures*, in the Concrete Masonry Association of California and Nevada publication, *Reinforced Load Bearing Concrete Block Walls for Multistory Construction* and in the *Recommended Practice for Engineered Brick Masonry* by the Brick Industry Association.

This method assumes that the section is homogeneous and the tension is resisted by reinforcing steel.

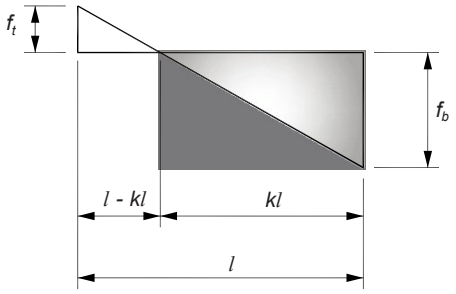
1. $f_m = \frac{P}{A} \pm \frac{M}{S}$



2. Check unity equation

$$\frac{f_a}{F_b} + \frac{f_b}{F_b} \le 1.00 \text{ or } \frac{4}{3}$$

3. Determine the total net tension force



$$kl = \left(\frac{f_m}{f_m + f_t}\right)(l)$$

$$(l - kl) = \left(\frac{f_t}{f_m + f_t}\right)(l)$$

$$\text{Tension Force} = \frac{1}{2} f_t b(l - kl)$$

4. Area of steel

The area of steel may be determined by dividing the tension force by the allowable tension stress which may be increased by one third if the force is due to wind or earthquake.

$$A_s = \frac{T}{F_s} \text{ or } \frac{T}{\frac{4}{3}F_s} = \frac{3T}{4F_s}$$

Using the allowable tensile stress for steel in the above equation is assuming that it will be strained sufficiently to produce a stress in the steel equal to the allowable stress.

An analysis in which the basic assumptions of:

a. Plane section remain plane after bending
b. Strain is proportional to the distance from the neutral axis.

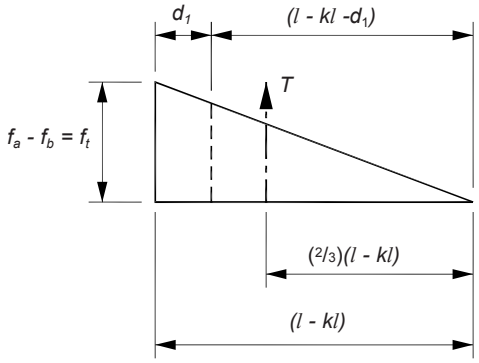
May give results that indicate the strains may be of such a value that the actual steel stresses are less than allowable values.

The steel may be assumed to be stressed to its allowable value because of the following assumptions:

c. Plane sections may not remain plane after bending

d. The section is cracked and the localized cracks will open up and cause a strain and thus a stress equivalent to the assumed stress level. This will then provide the required tension force.

5. Moment resistance of tension steel



The moment of the tension force, T, about the neutral axis is:

$$M_{N.A.} = T\left[\frac{2}{3}(l - kl)\right]$$

If the reinforcing steel is moved from the centroid of the stress triangle, two thirds of the distance from the neutral axis, to the actual location, d', from the edge of the wall to the jamb steel, then the tension force can be reduced because the moment arm is increased.

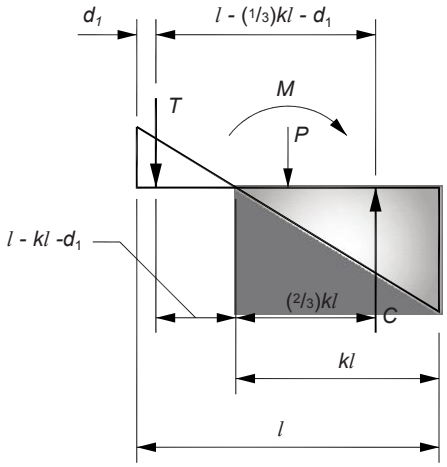
The equivalent tension force, T_{eq}, required is:

$$T_{eq} = T\left[\frac{2}{3}(l - kl)\right] \text{x} \frac{1}{1 - kl - d_1}$$

The adjusted area of steel would be

$$\text{Equivalent } A_s = \frac{T_{eq}}{\frac{4}{3}f_s} = \frac{3T_{eq}}{4f_s}$$

6. The section is then investigated to assure that the sum of the vertical forces equals zero and that the internal resisting moment equals the external applied moment.



$$\Sigma F_v = C - T - P = 0$$

$$\Sigma M_c = M - T\left(l - \frac{1}{3}kl - d_1\right) = 0$$

EXAMPLE 5-AD Interaction Design (Method 3).

Using Method 3 determine the stress and reinforcing steel required for a nominal 8 in. solid load bearing reinforced concrete masonry wall. The wall is 12 ft 0 in. long and spans vertically 10 ft 0 in. high between horizontal supports. The wall carries a total load of 2500 plf and an overturning moment due to seismic forces of 500 ft-kips.

Solution 5-AD

Assume f'_m = 1500 psi, n = 21.5, Allowable steel stress, F_s = 24,000 psi, r = 2.19, h/r < 99.

Following the outlined procedure:

1. $$f_a = \frac{P}{A} = \frac{(2500)(12)}{(7.63)(12)(12)}$$

$$= 27.3 \text{ psi}$$

$$F_a = 0.25f'_m\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

$$\frac{h}{r} = \frac{(10)(12)}{2.19} = 54.8 < 99$$

From Table ASD-9b, F_a = 317 psi > 27.3 O.K.

$$f_b = \frac{M}{S} = \frac{6M}{bd^2}$$

$$= \frac{6(500{,}000)(12)}{(7.63)[(12)(12)]^2}$$

= 227.5 psi

$$F_b = \frac{1}{3}f'_m = \frac{1500}{3}$$

= 500 psi > 227.5 O.K.

$f_t = f_a - f_b = 27.3 - 227.5 = -200.3$ psi

$f_m = f_a + f_b = 27.3 + 227.5 = 254.8$ psi

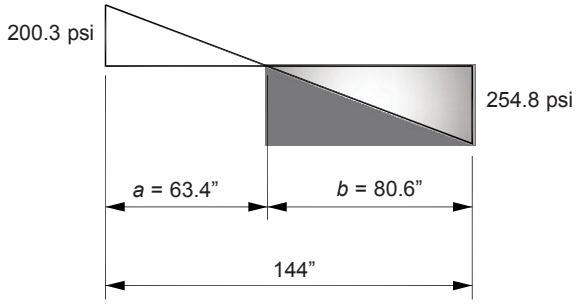
2. Unity check

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{27.3}{317} + \frac{227.5}{500}$$

$$= 0.086 + 0.455$$

$$= 0.541 < \frac{4}{3}$$

3. Tension force



$$a = \frac{200.3}{200.3 + 254.8}(144) = 63.4 \text{ in.}$$

$$b = \frac{254.8}{200.3 + 254.8}(144) = 80.6 \text{ in.}$$

Tension force = $f_t\ t\ a/2$

= (200.3)(7.63)(63.4)/2

= 48,446 lbs

4. Area of steel

$$A_s = \frac{T}{\frac{4}{3}F_s} = \frac{3T}{4F_s} = \frac{3(48{,}446)}{4(24{,}000)} = 1.51 \text{ sq in.}$$

Use 2 - #8 bars (A_s = 1.58 sq in.)

5. Equivalent tension force

$$T_{eq} = T\left[\frac{2}{3}(l - kL)\right]\frac{1}{(l - kL - d_1)}$$

$$= 48{,}446\left[\frac{2}{3}(144 - 80.6)\right]\frac{1}{(144 - 80.6 - 8)}$$

= 36,961 lbs

Equivalent $A_s = \dfrac{36{,}961}{1.33(24{,}000)} = 1.16$ sq in.

Use 2 - #7 bars (A_s = 1.20 sq in.)

6. Compression force

C = (254.8)(7.63)(80.6)/2 = 78,348 lbs

7. Check sum of vertical forces

$$C - T_{eq} = P$$

78,348 – 36,961 = 12(2500)

$$41{,}387 \neq 30{,}000$$

The sum of vertical forces are not in equilibrium, adjust the size of the compression stress block and magnitude of compression stress.

8. The stress block can be adjusted by iteration (trial and error) and by solving the relationship for sum of the forces and sum of the moments about the centroid simultaneously.

$$\sum F_{vertical} = C - T_{eq} - P = C - T_{eq} - 30{,}000 = 0$$

$$\sum M_{centroid} = M - \left(\frac{L}{2} - d'\right)T_{eq} - \left(\frac{L}{2} - \frac{kL}{3}\right)C =$$

$$6{,}000{,}000 - \left(\frac{144}{2} - 8\right)T_{eq} - \left(\frac{144}{2} - \frac{kL}{3}\right)C = 0$$

Substituting $T_{eq} = C - 30{,}000$ into the summation of moments:

$$\sum M_{centroid} = 6{,}000{,}000 - (64)(C - 30{,}000) - \left(72 - \frac{KL}{3}\right)C = 0$$

Solving for C gives:

$$C = \frac{7{,}920{,}000}{\left(136 - \frac{b}{3}\right)}$$

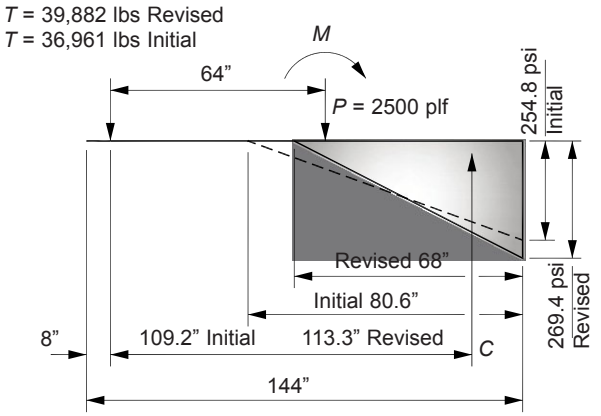
Note that the notation $b = kL$.

Try kL = 68 in. as the revised trial:

$$C = \frac{7{,}920{,}000}{\left(136 - \frac{68}{3}\right)} = 69{,}882 \text{ lbs}$$

$$f_m = \frac{C}{\frac{1}{2}(kL)t} = \frac{69{,}882}{\frac{1}{2}(68)(7.63)} = 269 \text{ psi}$$

$$T_{eq} = C - 30{,}000 = 69{,}882 - 30{,}000 = 39{,}882 \text{ lbs}$$



All subsequent trials with the expressions above will provide solutions which satisfy the balanced forces and moments, however, the solutions do not necessarilly provide strain compatibility between the tension in the steel and compression in the masonry. Strain compatibility can be utilized by including an expression which assumes a linear distribution of the strains for the wall. The resulting solution to the three simultaneous equations (shown as (1), (2), and (3) below) is the root of a cubic equation, illustrated after (1), (2), and (3). Table 5.3 provides an alternative trial and error solution with strain compatibility included.

The length of the compressive stress block and the area of tension reinforcement is adjusted to satisfy equilibrium of forces and moment, and the unity check.

Cubic Solution

The equivalent cubic equation for solving for kL or k would be derived from the three simultaneous conditions that must be satisfied, based on a linear distribution of strains:

TABLE 5.3 - Trial and Error[1,2,3]

Input Values		e_m[4]	f_m[5]	C[6]	ΣF[7]	ΣM_c[8]	f_b[9]	Unity Check[10]
kL	A_s							
80.6	1.2	0.001164	1571.26	483146	424,346	-17,649,201	1,543.96	3.17
68	1.2	0.000800	1080.00	280174	221,374	-9,665,098	1,052.70	2.19
53	1.2	0.000511	689.64	139441	80,641	-3,419,520	662.33	1.41
45	1.2	0.000396	534.07	91686	32,886	-1,069,289	506.76	1.10
40	1.2	0.000333	450.00	68670	9,870	128,160	422.70	0.93
35	1.2	0.000277	374.26	49973	-8,827	1,141,779	346.95	0.78
36	1.2	0.000288	388.80	53398	-5,402	952,932	361.50	0.81
37	1.2	0.000299	403.64	56975	-1,825	757,274	376.33	0.84
38	1.2	0.000310	418.78	60710	1,910	554,680	391.47	0.87
37.5	1.2	0.000305	411.17	58823	23	656,852	383.86	0.85
37.5	1.58	0.000305	411.17	58823	-9,097	73,172	383.86	0.85
38	1.58	0.000310	418.78	60710	-7,210	-29,000	391.47	0.87

1. Note the changes in sign as the trials progress (shaded). These are used to adjust kL and later A_s. The remaining negative values can be interpreted as having real values for f_m and f_s somewhat less than those computed and since the f_m and f_s values are less than the allowables and the unity check is satisfied, then the design is ok.
2. Based on: $f_s = F_s$ = 24,000 psi, $f_s = \varepsilon_s E_s$, E_s = 30,000,000 psi, such that ε_s = 0.0008 in./in.
3. Based on: $T_{eq} = F_s A_s$ = 28,800 lbs for 2-#7 bars (A_s = 1.2 in.^{2}) and T_{eq} = 37,920 lbs for 2-#8 bars (A_s = 1.58 in.^{2})
4. $\varepsilon_m = kL\,(f_s/E_s)/(L - kL - d') = kL\,(0.0008) / (136 - kL)$
5. $f_m = \varepsilon_m E_m$ where $E_m = 900\,f'_m = 900(1500)$ = 1,350,000 psi
6. $C = f_m t kL/2 = (7.63)\,f_m\,(kL) / 2$
7. $\Sigma F_{vertical} = C - T - P = C - T - 30{,}000$ which should equal zero
8. $\Sigma M_c = M - (L/2 - d')T - (L/2 - kL/3)C = 6{,}000{,}000 - (64)T - (72 - kL/3)C$ which should equal to zero
9. $f_b = f_m - f_a = f_m - 27.3$
10. $\frac{f_a}{F_a} + \frac{f_b}{F_b}$ which should be ≤ 1 $\left(\text{or with wind or seismic} \le \frac{4}{3}\right)$

$$\sum F_{vertical} = C - T_{eq} - P = 0 \quad (1)$$

$$\sum M_{centroid} = M - \left(\frac{L}{2} - d'\right)T_{eq} - \left(\frac{L}{2} - \frac{kL}{3}\right)C = 0 \quad (2)$$

$$\varepsilon_m = kL\ (\varepsilon_s) / (L - kL - d') \quad (3)$$

Substituting $T_{eq} = C - P$ into the second equation and using the linear strain relationship gives:

$$M - \left(\frac{L}{2} - d'\right)(C - P) - \left(\frac{L}{2} - \frac{kL}{3}\right)C = 0$$

$$M + \left(\frac{L}{2} - d'\right)P - \left(L - d' - \frac{kL}{3}\right)C = 0$$

$$M + \left(\frac{L}{2} - d'\right)P - \left(L - d' - \frac{kL}{3}\right)\left(\frac{1}{2}(kL)tf_m\right) = 0$$

$$M + \left(\frac{L}{2} - d'\right)P - \frac{t}{2}\left((L - d')(kL) - \frac{(kL)^2}{3}\right)(\varepsilon_m E_m) = 0$$

$$M + \left(\frac{L}{2} - d'\right)P - \frac{t}{2}\left((L - d')(kL) - \frac{(kL)^2}{3}\right)$$

$$\left(\left[\frac{kL(\varepsilon_s)}{(L - kL - d')}\right]E_m\right) = 0$$

$$M + \left(\frac{L}{2} - d'\right)P - \frac{t}{2}\left((L - d')(kL) - \frac{(kL)^2}{3}\right)$$

$$\left(\left[\frac{(kL)}{(L - kL - d')}\frac{f_s}{E_s}\right]E_m\right) = 0$$

$$\frac{n}{f_s}\left[M + \left(\frac{L}{2} - d'\right)P\right](L - kL - d')$$

$$- \frac{t}{2}\left((L - d')(kL)^2 - \frac{(kL)^3}{3}\right) = 0$$

$$(kL)^3 - 3(L - d')(kL)^2 - \frac{6n}{tf_s}\left[M + \left(\frac{L}{2} - d'\right)P\right](kL)$$

$$+ \frac{6n}{tf_s}\left[M + \left(\frac{L}{2} - d'\right)P\right](L - d') = 0$$

Substituting in the problem values, with $f_s = F_s$ = 24,000 psi:

$$\frac{6n}{tf_s}\left[M + \left(\frac{L}{2} - d'\right)P\right] = \frac{6(21.5)}{(7.63)(24{,}000)}$$

$$\left[6{,}000{,}000 + \left(\frac{144}{2} - 8\right)(30{,}000)\right] = 5579.3$$

$$(kL)^3 - 3(144 - 8)\ (kL)^2 - 5579.3\ (kL)$$

$$+ 5579.3(144 - 8) = 0$$

$$(kL)^3 - 408\ (kL)^2 - 5579.3\ (kL) + 758{,}784 = 0$$

kL = 38.39 in.

From the linear strain relationship:

$$\varepsilon_m = \frac{kL(\varepsilon_s)}{(L - kL - d')} = \frac{(38.386)\varepsilon_s}{(144 - 38.386 - 8)} = 0.39323\varepsilon_s$$

$$= 0.39323\varepsilon_s\left(\frac{24{,}000}{30{,}000{,}000}\right) = 0.39323(0.0008)$$

$$= 0.00031458$$

From the masonry modulus and the masonry compression block force C:

E_m = 900 f'_m = 900(1500) = 1,350,000 psi

$f_m = \varepsilon_m E_m$ = 0.00031458 (1,350,000) = 424.7 psi

$$C = \frac{1}{2}(kL)tf_m = \frac{1}{2}(38.386)(7.63)(424.7) = 62{,}192 \text{ lbs}$$

The steel area could be incorporated into the solution as well:

$$\sum F = C - T_{eq} - P = \frac{1}{2}(kL)tf_m - A_s f_s - P$$

$$= 62{,}192 - A_s(24{,}000) - 30{,}000 = 0$$

$$A_s = \frac{32{,}192}{24{,}000} = 1.3413 \text{ in.}^2 > 1.2 \text{ in.}^2$$

so use 2 - #8 bars $(A_s = 1.58 \text{ in.}^2)$

Note: that the cubic equation process above determines kL to be 38.39 in., which substantially agrees with the equilibrium approach shown in conjunction with the trial and error values given in the table above.

5.8 WALLS WITH FLANGES AND RETURNS, INTERSECTING WALLS

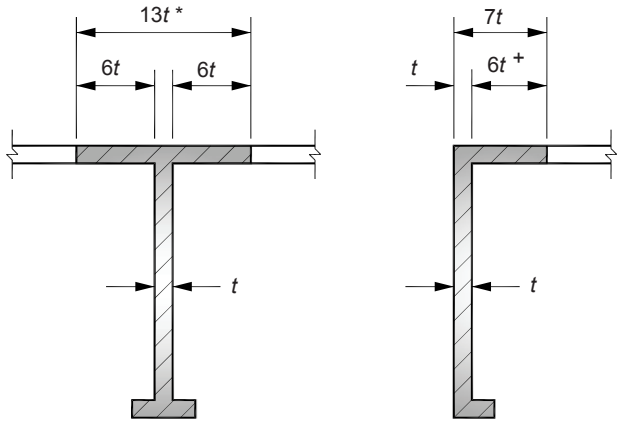
5.8.1 GENERAL

The design and analysis for combined stresses, axial and moment has been given in Section 5.8 for uniform rectangular sections. However, many walls intersect other walls and form I, U, C, Z and T sections. The sections provide greater moments of inertia and section moduli than a regular rectangular section.

Any reasonable assumption may be adopted for computing relative flexural stiffness of walls for the distribution of moment due to wind load. T-Beam action may be assumed where a shear wall intersects another wall or walls, using the effective flange for calculations width as one sixth of the total wall height above the level being analyzed and its overhanging width on either side of the shear wall up to six times the thickness of the intersected wall, as stated in MSJC Code Section 1.9.4.2.3:

MSJC Code Section 1.9.4.2.3

1.9.4.2.3 The width of flange considered effective on each side of the web shall be the lesser of 6 times the flange thickness or the actual flange on either side of the web wall.



*Effective flange width shall not exceed one sixth of the total wall height above level being analyzed

+Effective overhang flange width shall not exceed one sixteenth of the total wall height above level being analyzed

FIGURE 5.56 Flange on an intersection wall.

See Figure 5.56 for an illustration of 6*t*. MSJC Code Section 1.9.4.2.5 contains connection requirements for intersecting walls. The design for shear at the intersections conforms to the usual shear requirements as given in MSJC Code Sections 2.2.5 or 2.3.5. The vertical shear stress at the intersection may not exceed the allowable shear stress.

5.8.2 DESIGN PROCEDURE

The design procedure presented is similar to Design Method 3 of Section 5.8.5. Proceed as follows:

Given, calculate or assume

M; *P*; f'_m; *l* (length of wall); *t* (wall thickness); *I* (moment of inertia of wall); *S* (Section modulus of wall to each side); *d* (distance from compression face to center of steel, each direction); flange width if applicable; *h* (effective or actual height of wall).

Solve for:

1. Effective width of flange at each end; 1/16 to 1/6 of the wall height, 6*t* maximum each side.
2. Moment of inertia, uncracked section
3. Section modulus to each side
4. Kern distance $e_k = \frac{I}{Ac} = \frac{S}{A}$
5. Virtual eccentricity $e = \frac{M}{P}$
6. If $e \le e_k$ minimum reinforcement required

   If $e > e_k$, consider tension bond capability or design the reinforcement for flexural stresses if the tension stress exceeds flexural bond.
7. Actual axial stress $f_a = \frac{P}{A} = \frac{P}{bt}$

   Use actual cross-sectional area of masonry, web and flanges, and equivalent solid thickness for partially grouted walls.
8. *h/r* Reduction factor, *R* $\left(r = \sqrt{I / A_e}\right)$

$$= \left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ for } \frac{h}{r} \le 99$$

$$= \left(\frac{70r}{h}\right) \text{ for } \frac{h}{r} > 99$$

(See Tables ASD-9a, ASD-9b and ASD-9c)

The distance between points of support may be either horizontal, length of wall between the flanges, or the vertical, height between the floor and the roof, as presented in Section 5.6.1.2.

9. Maximum allowable axial stress

$F_a = 0.25\, f'_m\, R$

(See Tables ASD-9a, ASD-9b and ASD-9c)

10. Flexural stress assuming an uncracked section

$f_b = \frac{Mc}{I}$ for each side.

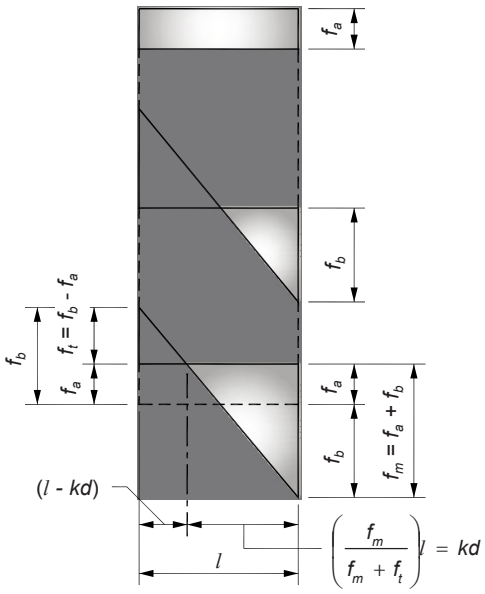
11. Maximum flexural compression stress

$F_b = \frac{1}{3} f'_m$

12. Unity equation check

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.00 \text{ or } \le \frac{4}{3}$$

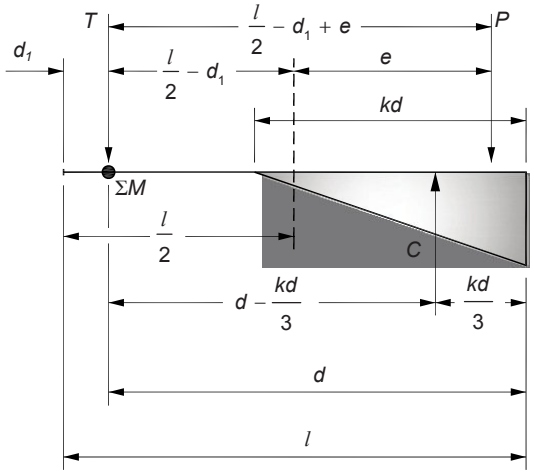
13. Combine stresses, f_a and f_b to establish the stress distribution on the wall.



14. Tension force, T, obtained by the average tension stress times the tension area.

15. Compression force, C, obtained by taking moments about centroid of tension steel. The moment of load P times moment arm must equal the compression force times the moment arm.

$$\Sigma M_T = \left(\frac{l}{2} - d_1 + e\right)P = \left(d - \frac{kd}{3}\right)C$$



16. Sum of the vertical forces must equal zero

$\Sigma F_v = T + P - C = 0$

If not in balance, adjust compression force and moment arm accordingly.

17. The steel area using maximum steel stress values. This is an acceptable approximation as presented in Design Method 3.

$$A_s = \frac{T}{F_s} \text{ or } \frac{T}{\frac{4}{3}F_s} = \frac{3T}{4F_s}$$

If centroid of steel is not at previously assumed location adjust the value of T and moment arm.

18. Select reinforcing steel to satisfy the area requirement.

19. Select balance of steel for wall.

EXAMPLE 5-AE Reinforcing Steel for Moment in a Flanged Wall.

Design the flanged wall section shown which is in a high rise building subjected to a wind moment of 4000 ft. kips and an axial load of 400 kips. The wall is 8 in. nominal thickness concrete masonry with a clear height between lateral supports of 16 ft 0 in.

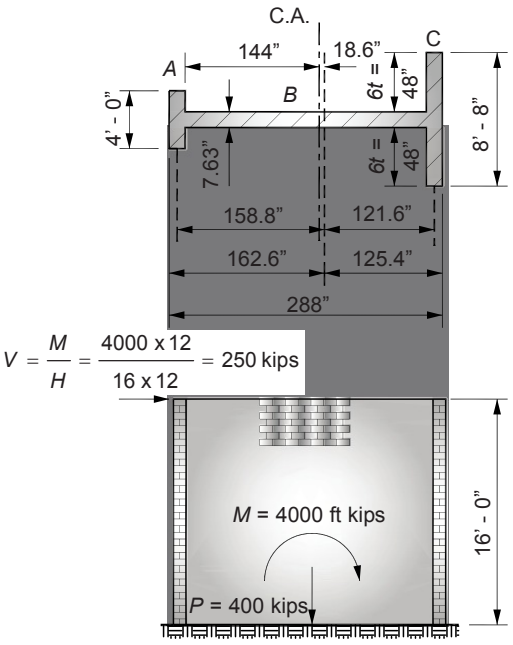


FIGURE 5.57 Shear wall with intersecting walls forming *I* section.

Assume solid grouted reinforced hollow unit masonry, f'_m = 2500 psi, F_s = 24,000 psi.

Solution 5-AE

1. Flanges are as shown = 48 in. on one end of wall, 104 in. on the other end of wall, maximum overhang of 6*t* (48 in.) on each side of wall.

2. Locate centroidal axis and determine moment of inertia.

$$\bar{x} = \frac{\text{Moment area}}{\text{area}} = \frac{525{,}850}{3235} = 162.6 \text{ in.}$$

3. Section modulus, $S = \frac{I}{c}$

$$S = \frac{34{,}489{,}000}{162.6} = 212{,}109 \text{ in.}^3 \qquad \text{to flange } A$$

$$S = \frac{34{,}489{,}000}{125.4} = 275{,}032 \text{ in.}^3 \qquad \text{to flange } C$$

4. Kern distance

$$e_k = \frac{S}{A} = \frac{212{,}109}{3235} = 65.6 \text{ in.} \qquad \text{to flange } C$$

$$e_k = \frac{S}{A} = \frac{275{,}031}{3235} = 85.0 \text{ in.} \qquad \text{to flange } A$$

5. Virtual eccentricity

$$e = \frac{M}{P} = \frac{(4000)(12)}{400}$$

= 120 in. > 65.6 in. and > 85.0 in.

6. Virtual eccentricity exceeds the kern distance for each direction from the neutral axis, therefore there will be tension on the section. Provide reinforcing steel to resist tension.

7. Actual axial stress

$$f_a = \frac{P}{A} = \frac{(400)(1000)}{3235} = 123.6 \text{ psi}$$

8. *h/r* Reduction factor

r = 2.19

$$\frac{h}{r} = \frac{(16)(12)}{2.19} = 87.7$$

R = 0.608 (Table ASD-9c)

TABLE 5.4 Location of Centroidal Axis and Determination of Moment Inertia

Section	Area (in.^{2})	Arm (in.)	Moment Area (in.^{3})	(in.^{4})	d = (x - Arm) (in.)	Ad^2 (in.^{4})	$I + Ad^2$ (in.^{4})
A	366	3.81	1,395	1,772	158.8	9,229,583	9,231,360
B	2,075	144.0	298,800	12,795,286	18.6	717,867	13,513,153
C	794	284.2	225,655	3,850	121.6	11,740,530	11,744,380

Σ = 3,235 sq in. Σ = 525,850 in.^{3} Σ ($I = Ad^2$) = 34,488,893

$\cong$ 34,489,000

9. Maximum allowable axial stress

$F_a = 0.25\, f'_m\, R$

$= 0.25(2500)(0.608) = 380$ psi

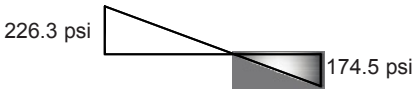
10. Flexural stress

$$f_b = \frac{M}{S} = \frac{(4000)(12)(1000)}{212{,}109}$$

= 226.3 psi on narrow end, tension or compression

$$f_b = \frac{M}{S} = \frac{(4000)(12)(1000)}{275{,}031}$$

= 174.5 psi on wide end, tension or compression



The flexural stress calculation only considers *M*. There is also likely some additional flexure due to the location of *P* not being coincident with the center of gravity for the flanged wall. The additional forces are assumed to be minor and are not considered in this example.

11. Maximum flexural compression stress

$$F_b = \frac{1}{3} f'_m = \frac{2500}{3} = 833 \text{ psi}$$

12. Check unity equation

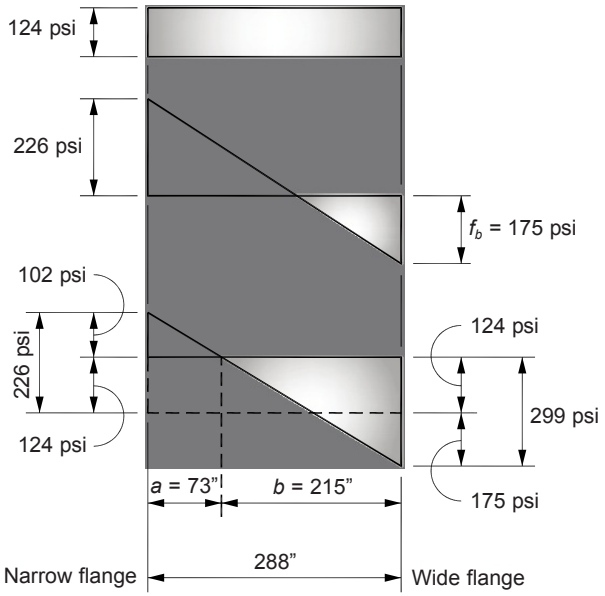
$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le \frac{4}{3} \qquad \text{(Wind forces)}$$

$$\frac{123.6}{380} + \frac{226.3}{833} = 0.325 + 0.272$$

$$= 0.597 < \frac{4}{3} \qquad \text{O.K.}$$

(Note that the unity equation is satisfied either with or without the one-third stress increase)

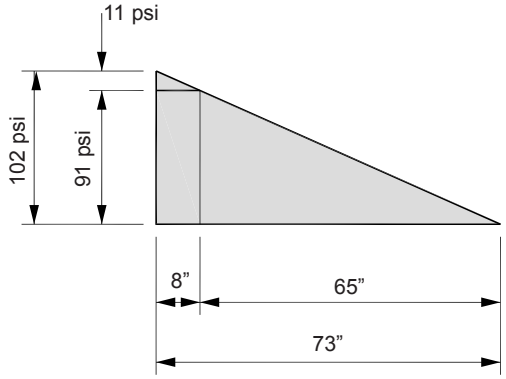
13. Combine stress



$$a = \frac{102}{102 + 299}(288) = 73 \text{ in.}$$

$$b = \frac{299}{102 + 299}(288) = 215 \text{ in.}$$

14. Tension force



Maximum tension stress in web.

$$= \frac{65}{73}(102) = 91 \text{ psi}$$

Tension force = (91)(7.63)(65)/2
+ (91)(7.63)(48) + (11)(7.63)(48)/2

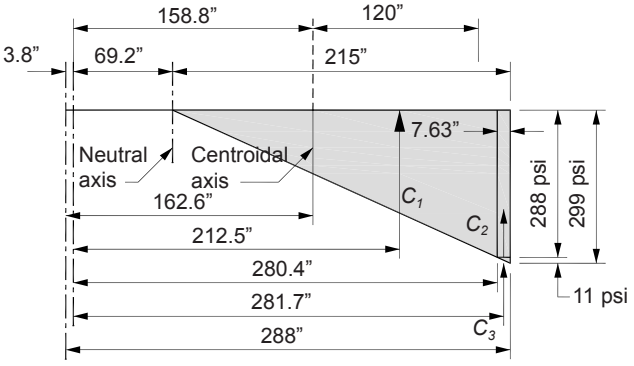
= 22,566 + 33,328 + 2,015

= 57,910 lbs

$$A_s = \frac{57{,}910}{1.33(24{,}000)} = 1.81 \text{ in.}^2$$

Use 2 - #9 bars (A_s = 2.0 sq in.)

15. Calculate compression force



$$\frac{215 - 7.6}{215}(299) = 288 \text{ psi} = 0.288 \text{ ksi}$$

C_1 = (0.288)(7.63)(215 - 7.6)/2 = 227.9 kips

C_2 = (0.288)(7.63)(104) = 228.5

C_3 = (0.011)(7.63)(104)/2 = 4.4

460.8 kips

ΣM_T = 212.5(227.9) + 280.4(228.5) + 281.7(4.4)

= (158.8 + 120) 400

= 48,429 + 64,071 + 1,239 = 111,520 ft-k

113,739 ≅ 111,520 ft - kips

Difference is small = 2,219 ≠ 0

16. Sum of vertical forces

$\Sigma F_V = T + P - C = 0$

= 57,910 + 400,000 ≅ 460,800 lbs

Difference is small = -2,890 ≠ 0

17. The values above are within a 1% range of error, and are acceptable.

The moment compression force and compression forces can be considered in equilibrium with the moment of the load and the tension force plus load, respectively.

18. Horizontal shear

$$V = \frac{M}{h} = \frac{4000}{16} = 250 \text{ psi}$$

$$f_v = \frac{V}{bd} = \frac{(250)(1000)}{(7.63)(288)} = 113 \text{ psi}$$

$$\frac{M}{Vd} = \frac{4000}{(250)(24)} = 0.67$$

For f'_m = 2500 psi and $\frac{M}{Vd} = 0.67$

From Tables ASD-5 and ASD-6

Allowable shear on masonry =

$$= \frac{4}{3}(50)$$

= 66.7 psi < 113 psi N.G.

Allowable shear with reinforcement =

$$= \frac{4}{3}(84)$$

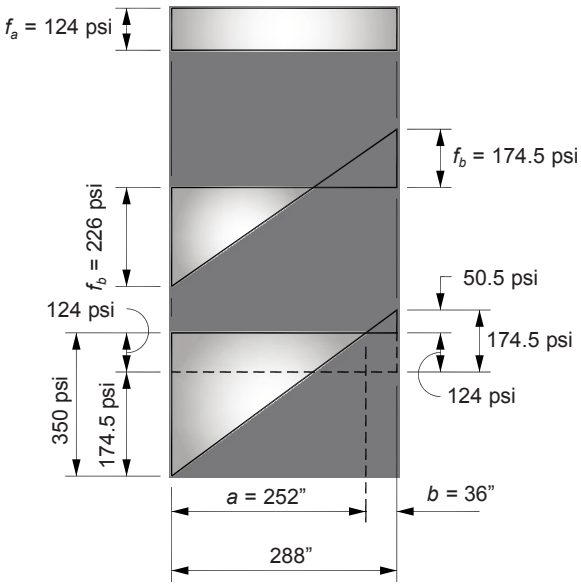
$$= 112 \text{ psi} \cong 113 \text{ psi}$$

From Diagram ASD-58

For t = 7.63 in., v = 113 psi, F_s = 32,000 psi

Use #8 at 24 in. o.c. spaced vertically

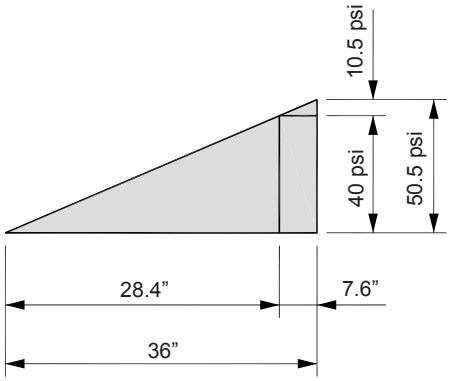
19. Consider moment in other direction. Flange A in compression



$$a = \frac{350}{350 + 50.5}(288) = 252 \text{ in.}$$

$$b = \frac{50.5}{350 + 50.5}(288) = 36 \text{ in.}$$

20. Tension in flange *C*.



$$\text{Maximum tension stress} = \frac{28.4}{36}(50.5)$$

$$= 40 \text{ psi}$$

Tension force

$$= (40)(7.63)(28.4)/2 + (40)(7.63)(104) + (10.5)(7.63)(104)/2$$

$$= 4334 + 31{,}741 + 4166 = 40{,}241 \text{ lbs}$$

$$A_s = \frac{40{,}241}{32{,}000} = 1.26 \text{ sq in.}$$

Use 2 - #8 (A_s = 1.58 sq in.)

5.8.3 CONNECTIONS OF INTERSECTING WALLS

When cross walls are considered as flanges to walls that resist overturning moments, the connections between them must be properly designed. The intersection of the flange or cross wall element to the web section is the critical location for stress concentrations. This stress is a vertical shear stress for it is delivering compression forces to the masonry or tension forces to the flange steel.

These connections should be evaluated to determine flange masonry or the amount and location of reinforcement required to permit the connection to function as desired. This evaluation is based on calculating the shear stress at the intersection:

$$v = \frac{VQ}{It} = \frac{VA_f y}{It}$$

Where

- v = Vertical shear stress
- V = Total shear
- A_f = Area of flange
- y = Distance from centroidal axis of the section to the centroid of the flange
- I = Moment of inertia
- t = Thickness of web

The limiting allowable shear stress is based on either the masonry or the reinforcing steel resisting all shear and is governed by the *M/Vd* or *h/d* value. See Tables ASD-5 and ASD-6 for limiting values.

If the shear stress is equal to or less than the allowable value for masonry, no reinforcement is required. If it is equal to or less than the allowable value for reinforcement to resist the shear forces, provide shear steel. If it exceeds the allowable value for reinforcing steel, increase the thickness of the wall and recompute all stresses.

The shear steel shall be determined by the equation:

$$A_v = \frac{Vs}{F_s d}$$

Where

- A_v = Area of shear steel
- V = Total shear
- s = Spacing of shear steel
- F_s = Allowable tensile stress for shear steel. May be increased one third for wind and seismic forces.
- d = Depth or length of shear wall

EXAMPLE 5-AF Intersecting Walls – Vertical Shear

Calculate the vertical shear at the intersection of the web and the flange from Example 5-AE.

$$V = \frac{M}{h} = \frac{4000}{16} = 250 \text{ kips}$$

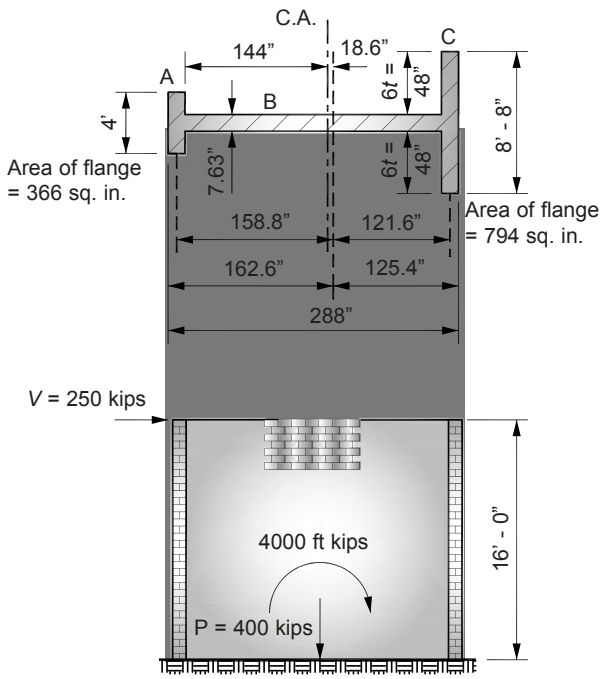


FIGURE 5.58 Flanged shear wall.

Vertical shear $v = \dfrac{VA_f y}{It}$

V = 250 kips

A_{fa} = 366 sq in. y_a = 158.8 in.

A_{fc} = 794 sq in. y_c = 121.6 in.

I = 34,489,000 in.^{4} t = 7.63 in.

$$v_{fa} = \frac{VA_f y}{It}$$

$$= \frac{(250)(1000)(366)(158.8)}{(34{,}489{,}000)(7.63)}$$

= 55.2 psi

$$v_{fc} = \frac{(250)(1000)(794)(121.6)}{(34{,}489{,}000)(7.63)}$$

= 91.7 psi

$$\frac{M}{Vd} = \frac{4000}{(250)(24)} = 0.67$$

for f'_m = 2500 psi and $\dfrac{M}{Vd} = 0.67$

From Tables ASD-5 and ASD-6

Allowable shear stress, masonry resisting shear,

$$\frac{4}{3}(35 \text{ psi}) = 44.7 \text{ psi}$$

Allowable shear stress, reinforcement resisting shear,

$$\frac{4}{3}(75 \text{ psi}) = 100 \text{ psi}$$

Provide shear reinforcement for vertical shear forces

Vertical shear = vA_w

= 91.7(7.63)(16)(12)

= 134.3 kips

$$A_v = \frac{Vs}{F_s d} = \frac{(134.3)(1000)(24)}{(32{,}000)(288)} = 0.35 \text{ sq in.}$$

Use #6 at 24 in. o.c. spaced vertically (A_s = 0.44 in.^{2})

The tension steel provided at the end will be adequate to resist and transfer the vertical shear between the web (cross wall) and the flanges (end walls).

Use 2 - #9 bars at wall A and 2 - #8 bars at wall C.

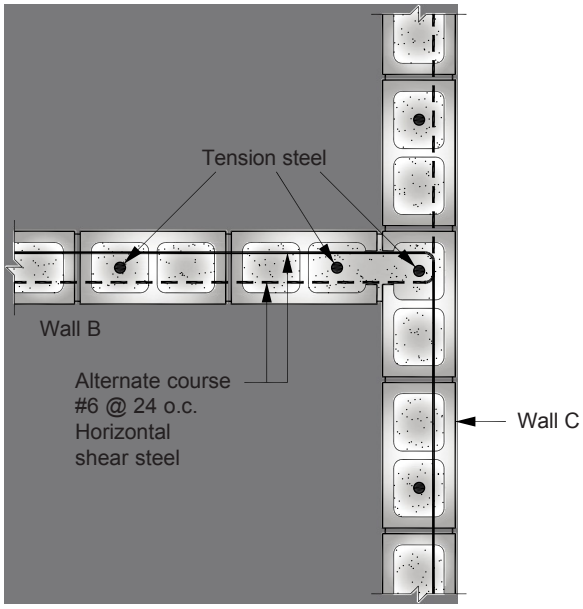
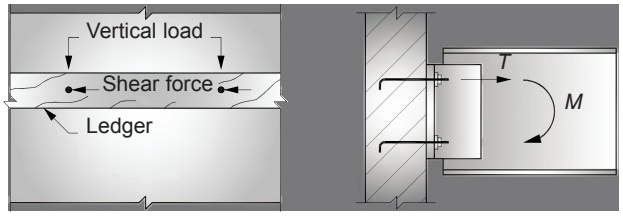


FIGURE 5.59 Detail of connection of intersecting walls.

5.9 EMBEDDED ANCHOR BOLTS

Embedded anchor bolts are structural connections used to secure beams, columns, angles and other load bearing systems to masonry. The embedded bolts may be stressed in tension, shear or combined tension and shear.



Anchor bolts in ledger subjected to vertical load and lateral shear

Anchor bolts in connecting angle subjected to vertical shear and tension

FIGURE 5.60 Typical loads on anchor bolts.

The maximum allowable tension on the masonry from an embedded anchor bolt is given by the equation.

$$B_a = 0.5A_p\sqrt{f'_m} \qquad \text{(MSJC Code Eq 2-1)}$$

The limiting area, A_p, is the lesser of the following two equations based on depth of embedment, l_b, or the edge distance, l_{be}.

$$A_p = \pi\, l_b^{\,2} \qquad \text{(MSJC Code Eq 2-3)}$$

$$A_p = \pi\, l_{be}^{\,2} \qquad \text{(MSJC Code Eq 2-4)}$$

When the projected areas of adjacent anchor bolts overlap, the A_p of each anchor bolt is reduced by one half of the overlapping area. See Table ASD-7c for the percent capacity reduction of anchor bolts in tension based on embedment and spacing.

The maximum allowable tension on the anchor bolt is given by the equation.

$$B_a = 0.2A_bf_y \qquad \text{(MSJC Code Eq 2-2)}$$

The limiting value for B_a must be used for design.

The maximum allowable shear load is the lesser of the shear load on the masonry or on the bolt as determined by the following equations:

$$B_v = 350\sqrt[4]{f'_m\, A_b} \qquad \text{(MSJC Code Eq 2-5)}$$

$$B_v = 0.12A_bf_y \qquad \text{(MSJC Code Eq 2-6)}$$

The anchor bolt edge distance, l_{be}, in the direction of the shear load should be 12 bolt diameters for MSJC Code Equation 2-5 but the shear stress may be reduced linearly to zero when the l_{be} is 1 in. (see Table ASD-8b). For combined tension and shear on anchor bolts, the unity equation must be satisfied.

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \le 1.0 \text{ or } \le \frac{4}{3} \qquad \text{(MSJC Code Eq 2-7)}$$

EXAMPLE 5-W Anchor Bolt Analysis.

Determine the adequacy of an embedded anchor connection supporting a cantilever steel beam with a load of 400 lbs as shown.

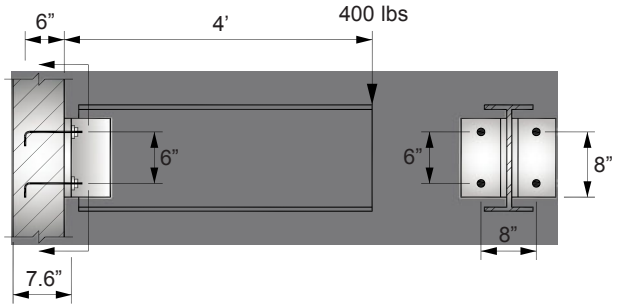


FIGURE 5.61 Section of cantilever beam connection.

Given: f'_m = 1500 psi; Nominal 8 in. CMU solid grouted; 3/4 in. anchor bolts embedded 6 in. into the wall.

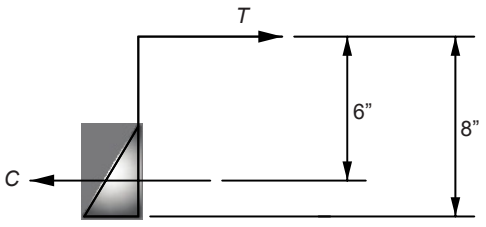
Solution 5-W

Moment on connection

$M = Pl$ = (400)(4) = 1600 ft-lbs

Assume moment resistance on connection is as shown:

Tension pull on bolt



$$b_t = b_a = \frac{M}{d} = \frac{(1600)(12)}{(6)(2)}$$

$$= 1600 \text{ lbs/bolt}$$

Shear on bolts

$$b_v = 400/4 = 100 \text{ lbs/bolt}$$

Allowable tension on 3/4 in. diameter anchor bolts with 6 in. embedment and 8 in. spacing.

From Table ASD-7a, masonry value

B_a = 2190 lbs/bolt for a spacing of $2l_b$ or more

From Table ASD-7c, find the percent capacity of the anchor bolts:

spacing = 8 in., l_b = 6 in.

$$\frac{8}{6} = 1.3l_b$$

% capacity = 88%

Allowable masonry value = (2190)(0.88) = 1927 lbs/bolt

From Table ASD-7b, steel value

B_a = 3180 lbs/bolt

Tension on masonry governs

Allowable shear on bolts

From Table ASD-8a, B_v = 1780 lbs

Check compliance with interaction unity equation

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \leq 1.00$$

$$\frac{1600}{1927} + \frac{100}{1780} = 0.83 + 0.07$$

$$= 0.90 < 1.00$$

Embedded anchor bolt connection is satisfactory.

5.10 QUESTIONS AND PROBLEMS

5-1 What are the basic assumptions in elastic design of a flexural member?

5-2 Is strain compatible with stress? What is its significance with respect to compression steel?

5-3 What is the modular ratio? What is its significance?

5-4 Explain the function of the flexural coefficient, K_f. How does it vary from an under-reinforced section to an over-reinforced section.

5-5 Given, a 10 in. (nominal) concrete masonry cantilever retaining wall reinforced with vertical steel #6 bars 24 in. on center. What is the maximum d value that this wall could be designed for? Locate the neutral axis by means of transformed areas if this wall is solid grouted and f'_m is 2500 psi. If the reinforcing steel has a maximum allowable stress of 24,000 psi, what is the allowable moment for the section?

5-6 From basic principles, establish the following values for a rectangular section for f'_m = 2250 psi, f_s = 18,000 psi

a) balanced steel ratio, ρ

b) balanced flexural coefficient, K_b

c) j, k values for balanced condition

5-7 What is the limiting stress in compressive reinforcement? Explain in terms of n, f'_m, f'_s. What are the limiting features?

5-8 Determine the moment capacity and maximum size reinforcing bar that can be placed in an 8 in. CMU and still have the neutral axis in the face shell. Given face shell thickness = 1 1/4 in.; d = 5.3 in.; bar spacing = 24, 32 and 48 in. o.c. and modular ratio, n = 21.48; 15.5 and 9.7, F_s = 24,000 psi.

5-9 A two-wythe clay masonry lintel beam is 10 in. wide by 32 in. deep. It spans over an opening 20 ft wide. What is the maximum uniform load that can be placed on this lintel beam if shear is the governing stress? The f'_m of the masonry is 2000 psi.

Determine the allowable super-imposed load for:

a) masonry that resists all shear,

b) steel that resists all shear,

Shear steel is #6 vertical bars at 14 in. on center, F_s = 24,000 psi.

5-10 What is the allowable shear stress parallel to a 10 in. thick clay masonry shear wall if the wall is 20 ft long and 40 ft high and if the wall is 50 ft long and 15 ft high? Consider that the masonry is to resist all of the shear, f'_m = 2500 psi. Consider that the masonry is to resist none of the shear and that horizontal steel reinforcement (#6 bars 24 in. on center, F_s = 24,000 psi) resists all the shear.

5-11 What is the shear resistance per linear foot of a 10 in. thick clay masonry wall at the floor joint both parallel to and perpendicular to the wall if the axial stress is 135 psi?

5-12 What is the anchorage length required for a #7 bar in masonry (f'_m = 1500 psi) and in concrete (f'_c = 2000 psi)?

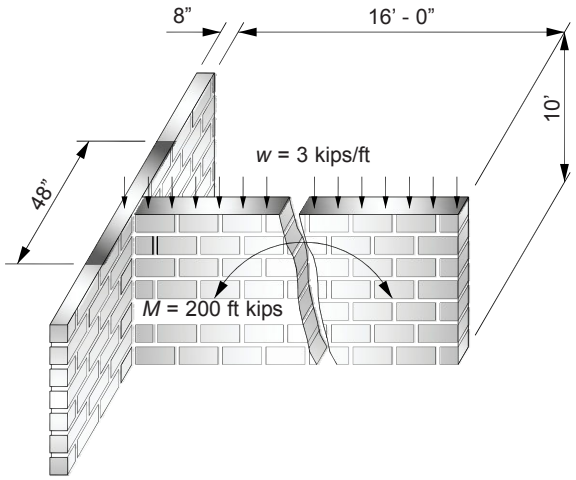
5-13 Design the tension reinforcement and specify the minimum allowable strength of masonry, f'_m, for a wall subjected to axial load and seismic overturning moment. The wall is a nominal 10 in. thick, 10 ft long and 12 ft high. F_s = 24,000 psi. Axial load = 100 kips, overturning moment = 300 ft-kips parallel to the wall.

5-14 An 8 in. concrete masonry wall, solid grouted is 12 ft high and is reinforced with #7 bars at 24 in. on center. Axial load is 3 kips per foot, f'_m = 1500 psi. What is the maximum moment that can be applied perpendicular to the wall if d is 3.75 in. and if d is 5.25 in.?

5-15 What is the reinforcement required for a wall subjected to vertical load of 100 kips and an overturning moment of 200 ft-k. The masonry is 8 inches solid grouted, f'_m = 2500 psi, F_s = 24,000 psi, h = 10 ft,

a) Ignore Tee Flange

b) Include Tee Flange



5-16 A 10 in. thick CMU beam spans 25 ft. The beam has a total depth of 48 in. and is continuous on both ends. It carries a live load of 1000 lbs per linear foot. Design the reinforcement both in the center and over the supports and the shear reinforcement, if required. f'_m = 1500 psi, special inspection is used in its construction. If the masonry strength is not sufficient, what f'_m should be used?

5-17 What is the moment capacity of a grouted concrete block beam 8 in. thick, total depth 32 in., d = 26 in. and reinforced with two #8 bars? Use f'_m = 1500 psi and F_s = 24,000 psi.

5-18 A two wythe masonry wall 8 1/2 in. thick is reinforced vertically with #6 bars at 30 in. on centers in the center of the wall. It is subjected to a bending moment of 1000 ft pounds per foot. Assume that f'_m = 1800 psi and n = 21.5. What are the stresses in the masonry and steel? If the bending moment is 1.5 kip ft/ft what are the stresses? Are they within the allowable range?

5-19 A masonry beam 12 in. wide and 30 in. deep (d = 24 in.) spans 20 feet. It carries a live and dead load of 1000 plf. For f'_m = 2000 psi and F_s = 24,000 psi, design the tension reinforcement and compression reinforcement if needed, and the shear reinforcement. Also design reinforcement if the *LL* plus *DL* is 2000 plf.

5-20 Design a 13 in. wide double reinforced clay masonry beam for a total moment, M = 500 ft-kips using f'_m = 2500 psi, Grade 60 reinforcing steel and a cover of 4 in. to center of steel. Assume d' = 4 in. and d = 60 in. Determine the required steel.

5-21 A 12 in. by 48 in. concrete block beam has d = 40 in., d' = 4 in., and A 615, Grade 60 steel.

What is the moment capacity if

1) A_s = 2 - #9, and A'_s = 2 - #6

2) A_s = 2 - #10, and A'_s = 2 - #11

3) A_s = 3 - #11, and A'_s = 2 - #11

5-22 Calculate the allowable load on the following columns if h =13 ft 4 in. and the columns have 3/8 in. head joints.

Size (in.)	Reinforcement	f'_m (psi)	Inspection
8 x 32	4 - #6	1500	no
10 x 16	4 - #7	1500	yes
16 x 16	4 - #8	1500	no
24 x 32	8 - #9	2000	yes

5-23 A 20 ft high interior column supports and axial load of 200 kips. Determine the size of the column, vertical reinforcing steel, and the tie spacing,

a) Reinforced clay masonry, f'_m = 2500 psi

b) Reinforced concrete masonry f'_m = 1500 psi

Assume F_s = 24,000 psi.

5-24 A concrete masonry column 16 in. x 16 in. (nominal) is 14 ft high and is reinforced with four No. 9, grade 60 bars. What vertical load at an eccentricity of 12 in. can it support?

5-25 Design a 22 ft high reinforced clay masonry wall to carry an axial load of 5 kips/ft and a moment perpendicular to the wall of 2 ft-kips/ft. Use f'_m = 2500 psi, F_s = 24,000 psi.

5-26 Select the reinforcement required for a 10 in. clay masonry wall which is subjected to an axial load of 2000 plf and a moment perpendicular to the wall of 2000 ft-lbs/ft. Use f'_m = 4000 psi, F_s = 24,000 psi, h = 18 ft, steel in center of wall.

5-27 For the concrete masonry beam shown below, f'_m = 1500 psi and F_s = 24,000 psi. Neglecting the weight of beam, calculate the depth, d, and total depth of the beam for these items individually.

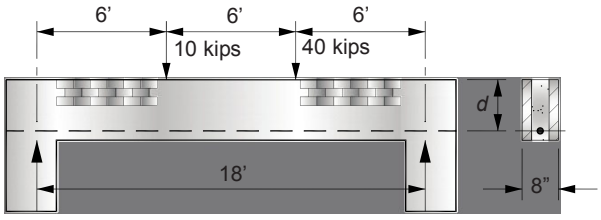
a) depth without stirrups

b) depth with stirrups

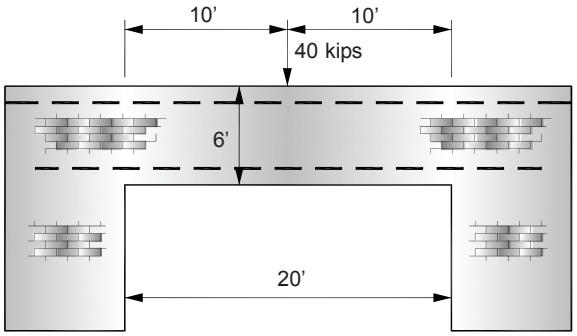
c) depth for bond

d) depth for stress in steel

e) depth for maximum stress in steel or masonry

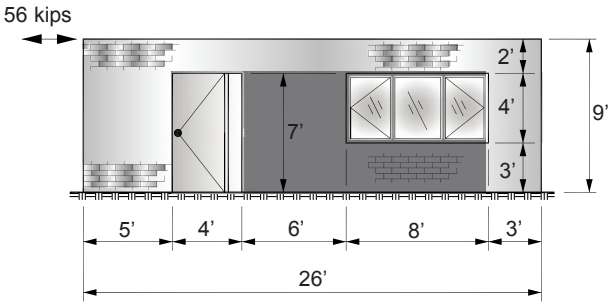


5-28 Design the flexural tension reinforcement, compression reinforcement, if needed, and shear reinforcement for the lintel beam shown below. Use f'_m = 3000 psi, 8 in. CMU, normal weight, solid grouted and F_s = 24,000 psi.



5-29 Design the shear reinforcement and calculate the embedment length for the cantilevered beam shown assuming f'_m = 2000 psi, F_s = 24,000 psi

5-30 Determine the shear in the 8 in. concrete masonry piers shown below. Determine the shear stress and shear reinforcement, if necessary. Assume f'_m = 1500 psi, F_s = 24,000 psi.



5-31 Design a reinforced masonry wall for a commercial building with walls 14 ft high from floor to roof ledger beam. Walls are 6 inches thick and the building is in Seismic Design Category C. Wind = 15 psf, f'_m = 1500 psi and F_s = 24,000 psi

5-32 Determine the shear reinforcement and overturning steel for an 8 in. CMU shear wall which is 10 ft long and 12 ft high. Assume f'_m = 2000 psi, F_s = 24,000 psi and the lateral seismic force at the top of the wall is 90 kips.

CHAPTER 6

DESIGN OF STRUCTURAL MEMBERS BY STRENGTH DESIGN

6.1 GENERAL

The structural design of reinforced masonry is changing from using entirely the elastic allowable stress method to now providing for strength design procedures. The previous chapter concentrated solely on the Allowable Stress Design (ASD), whereas this chapter will focus on Strength Design (SD) procedures. In general, the philosophy of the reinforced condition is similar to SD in reinforced concrete. There are, however, significant differences between SD of reinforced concrete and reinforced masonry that a designer needs to be aware of. Chapter 3 of the MSJC Code provides the requirements for "Strength Design of Masonry".

Charles Whitney pioneered the concept of strength design flexure in his technical paper *Plastic Theory of Reinforced Concrete* published in the 1942 ASCE Transaction 107. His theory states when a reinforced concrete section is subjected to high flexural moments, the concrete stress from the neutral axis to the extreme compression fibers would conform to the stress strain curve of the materials as if it were tested in compression.

The distribution of stress in concrete and masonry is roughly parabolic as illustrated in Figure 6.1.

Whitney also states that when the tension reinforcement reaches its yield stress, it will continue to elongate without an increase in moment force. This condition occurs at the yield plateau of the steel as shown by the idealized lines on the stress-strain curve in Figure 6.2.

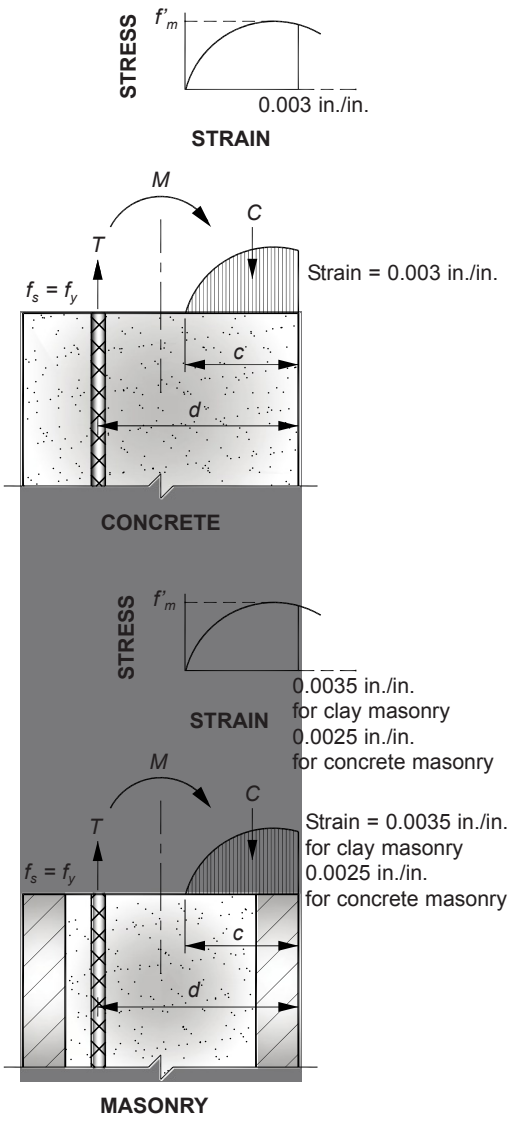


FIGURE 6.1 **Stress due to flexural moment at balanced condition.**

Masonry systems have compression stress-strain curves similar to those of concrete, in that the curves are curved or parabolic shaped and that they reach a strain of at least 0.0025 for concrete masonry and a strain of 0.0035 for clay masonry. These strain values come from the research program Technical Coordinating Committee for Masonry Research (TCCMaR). The program was for the U.S.-Japan joint research program.

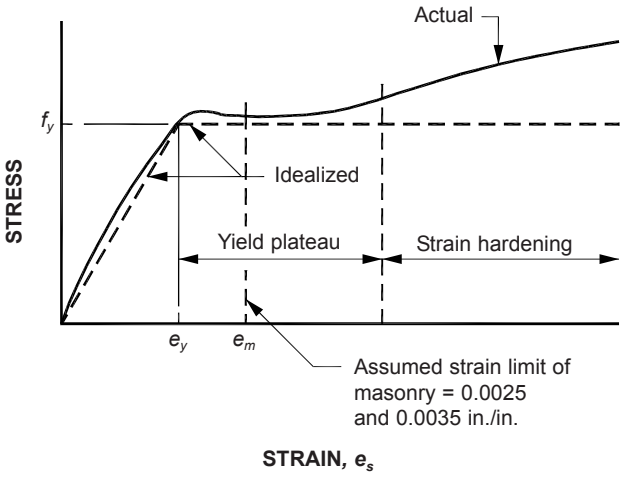


FIGURE 6.2 Idealized stress diagram for reinforcing steel.

The compressive stress block of the concrete, as shown in Figure 6.3, is simplified from the curved or parabolic shape to a rectangular configuration. This rectangular stress block, which is now often called Whitney's stress block, is approximated as having a length of a and a height of 0.85 f'_c for concrete strength design.

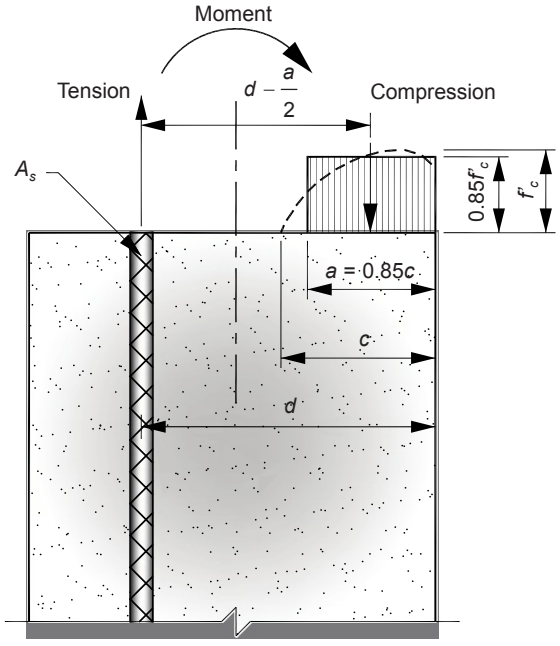


FIGURE 6.3 Assumed stress block at yield condition for concrete.

These values are not exactly the same for masonry structures for strength design. Several investigators in conjunction with the US TCCMaR research program have arrived at the conclusion that the height or thickness of the stress block is $0.80f'_m$ and the depth of this equivalent stress block is expressed as:

$$a = 0.80\ c,$$

where a is the depth of the stress block and c is the depth to the neutral axis, as shown in Figure 6.4. These are prescribed in MSJC Code Section 3.3.2.

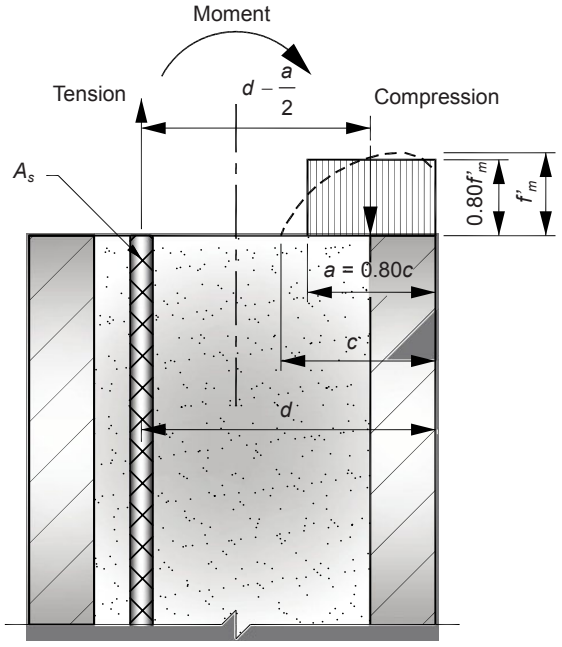


FIGURE 6.4 Masonry stress block.

6.2 DEVELOPMENT OF STRESS CONDITIONS

A structural element is loaded in flexure with one side is stressed in tension while the other is stressed in compression. When the modulus of rupture is reached, the tension side of the element cracks and the reinforcing steel resists the tension force. As the moment is increased, the stress in the steel and masonry also increases. The shape of the stress block for the masonry parallels a stress-strain curve (Figure 6.5).

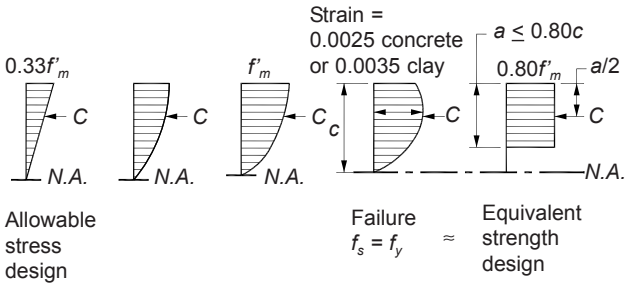


FIGURE 6.5 Variation in stress block as moment increases and the steel yields.

For safety, concrete and masonry sections are designed to be under-reinforced so the reinforcing steel is stressed to yield strength well before the masonry reaches full strength capacity. This under-reinforced concept prevents masonry from failing suddenly in compression.

When the steel is stressed to yield (which for Grade 60 steel is assumed to be 60,000 psi at an initial strain of 0.002 in./in.), it continues to stretch without a significant increase in stress as can be seen in Figure 6.2. As the steel stretches, the depth of the masonry stress block decreases and the stress and strain increase until the masonry is strained to the assumed maximum strain of either 0.0025 in./in. for concrete masonry or 0.0035 in./in. for clay masonry at which point the masonry is assumed to fail in a crushing compression failure. The total maximum masonry compression strain actually ranges from 0.003 to 0.005 in./in. The values of 0.0025 in./in. for concrete masonry and of 0.0035 in./in. for clay masonry are conservatively used.

Items 1, 2 and 3 below describe conditions that occur on the stress and strain diagrams shown in Figure 6.6.

1. Allowable stress flexural tension stress for steel, $f_s = 0.4f_y = 24{,}000$ psi, (for Grade 60)

   Allowable stress flexural compression stress in masonry, $f_b = 0.33f'_m$.

2. Reinforcing steel is stressed to yield, $f_s = f_y$

   Masonry is stressed from 0.64 to 0.8 f'_m

3. Reinforcing steel stretches,

   Strain increases in steel until the strain in masonry is 0.0025 or 0.0035 in./in.

6.3 STRENGTH DESIGN PROCEDURE

There are two conditions included in strength design. They are the load and the design parameters.

6.3.1 LOAD PARAMETERS

6.3.1.1 LOAD FACTORS

Service loads or actual loads are generally used for allowable stress design procedures. For strength design procedures, however, the actual or specified code loads are increased by prescribed load factors. These load design factors which are given in IBC Section 1605.2 or MSJC Code Section 3.1.2 which in turn refers to ASCE 7 load combinations, consider live load, dead load, wind, earthquake, temperature, settlement and earth pressure. The appropriate or most severe loading condition is used to design the structural element. The load factors are for the purpose of the following:

- Deviations of the actual loads from the prescribed loads,
- Uncertainties in the analysis and distribution of forces that create the load effects,
- The probability that more than one extreme load effect will occur simultaneously.

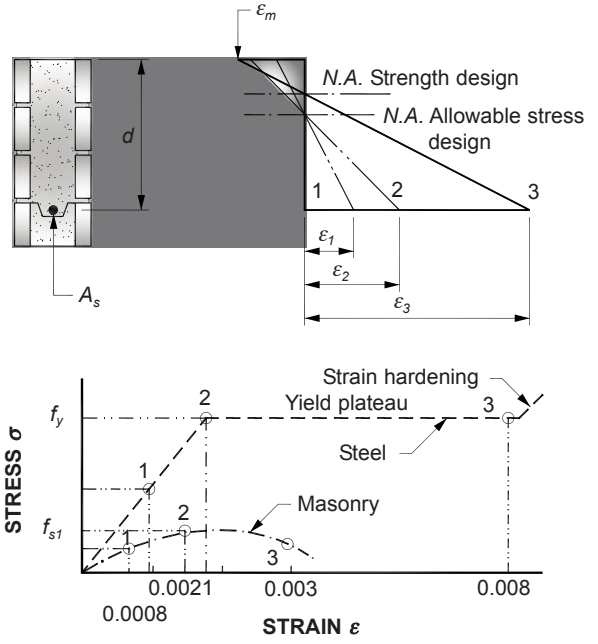


FIGURE 6.6 Development of stress and strain in a flexural member. (Leet, 1982)

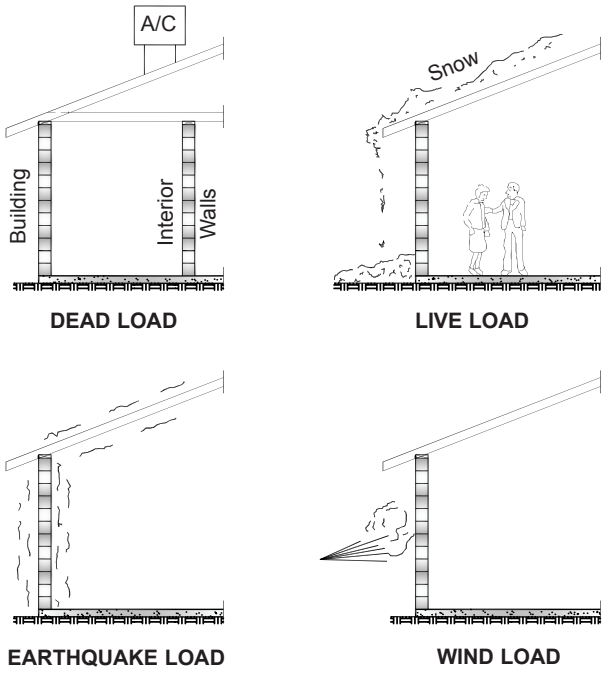


FIGURE 6.7 Types of loads.

IBC Section 1605.2.1

1605.2.1 Basic load combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4\ (D + F)$ **(Equation 16-1)**

$1.2(D + F + T) + 1.6(L + H) + 0.5\ (L_r \text{ or } S \text{ or } R)$ **(Equation 16-2)**

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (f_1 L \text{ or } 0.8W)$ **(Equation 16-3)**

$1.2D + 1.6W + f_1 L + 0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-4)**

$1.2D + 1.0E + f_1 L + f_2 S$ **(Equation 16-5)**

$0.9D + 1.6W + 1.6H$ **(Equation 16-6)**

$0.9D + 1.0E + 1.6H$ **(Equation 16-7)**

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 N/m^2), and for parking garage live load, and

= 0.5 for other live loads.

f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and

= 0.2 for other roof configurations.

Exception: Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.

1605.2.2 Other loads. Where F_a is to be considered in the design, the load combinations of Section 2.3.3 of ASCE 7 shall be used.

Additional Required Strength provisions are given in MSJC Code Section 3.1.2.

MSJC Code Sections 3.1.2

3.1.2 *Required strength*

Required strength shall be determined in accordance with the strength design load combinations of the legally adopted building code. When the legally adopted building code does not provide factored load combinations, structures and members shall be designed to resist the combination of loads specified in ASCE 7-02 for strength design. Members subject to compressive axial load shall be designed for the factored moment accompanying the factored axial load. The factored moment, M_u, shall include the moment induced by relative lateral displacement.

Note that the 2005 MSJC Code references ASCE 7-02, but the 2002 edition of the MSJC Code references ASCE 7-98. The reader is also cautioned that in the various versions of the MSJC Code, ASCE 7 references may not be the same throughout the entire document.

6.3.1.2 STRENGTH REDUCTION FACTOR, ϕ

No material is precisely as specified and no construction is exactly in accordance with the plans. In each case, there are variations in the strength, size, and placement of materials that will change, and possibly reduce the capacity of the section.

Accordingly, a strength reduction factor, ϕ, is used to lower the capacity of an ideally constructed member to a realistic capacity that can be assured.

The strength reduction factor, ϕ, is based on:

(a) the ratio of the mean capacity to nominal design moment,

(b) the uncertainty or quality of construction and analytical modeling and,

(c) the level of safety that the design criterion seeks to attain for the specific limit state under consideration.

MSJC Code Section 3.1.3

3.1.3 *Design strength*

Masonry members shall be proportioned such that the design strength equals or exceeds the required strength. Design strength is the nominal strength multiplied by the strength-reduction factor, ϕ, as specified in Section 3.1.4.

The design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the member, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u.

3.1.3.1 *Seismic design provisions* — At each story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls. Along each column line at a particular story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls.

Exception: Where seismic loads are determined based on a seismic response modification factor, R, not greater than 1.5, piers and columns are permitted to be used to provide seismic load resistance.

For masonry elements, for example, the strength reduction factor, ϕ, for flexural capacity is 0.90. The Strength Design method, as well as ASD, requires that inspection be provided during construction for quality assurance.

The various capacity reduction factors are shown in MSJC Code Section 3.1.4:

MSJC Code Section 3.1.4

3.1.4 *Strength-reduction factors*

3.1.4.1 *Combinations of flexure and axial load in reinforced masonry* — The value of ϕ shall be taken as 0.90 for reinforced masonry subjected to flexure, axial load, or combinations thereof.

3.1.4.2 *Combinations of flexure and axial load in unreinforced masonry* — The value of ϕ shall be taken as 0.60 for unreinforced masonry subjected to flexure, axial load, or combinations thereof.

3.1.4.3 *Shear* — The value of ϕ shall be taken as 0.80 for masonry subjected to shear.

3.1.4.4 *Anchor bolts* — For cases where the nominal strength of an anchor bolt is controlled by masonry breakout, ϕ shall be taken as 0.50. For cases where the nominal strength of an anchor bolt is controlled by anchor bolt steel, ϕ shall be taken as 0.90. For cases where the nominal strength of an anchor bolt is controlled by anchor pullout, ϕ shall be taken as 0.65.

3.1.4.5 *Bearing* – For cases involving bearing on masonry, ϕ shall be taken as 0.60.

6.3.2 DESIGN PARAMETERS

The parameters for Strength Design are:

a) The steel is at yield stress.

b) The masonry stress block is rectangular.

c) The masonry strain is limited to 0.0025 in./in. for concrete masonry and 0.0035 in./in. for clay masonry.

d) The steel ratio, ρ, is limited to various strain compatibility variations depending upon the kind of element and the type of seismic condition to assure that a ductile mechanism forms prior to brittle, crushing behavior. Section 6.4.1.1 contains more detail on the balanced steel ratio, ρ. The MSJC Code Section 3.3.3.5 prescribes the conditions for the maximum reinforcement percentages, as follows:

MSJC Code Section 3.3.3.5

3.3.3.5 *Maximum area of flexural tensile reinforcement*

3.3.3.5.1 For masonry members where $M_u/V_u d_v \geq 1$, the cross-sectional area of flexural tensile reinforcement shall not exceed the area required to maintain axial equilibrium under the following conditions:

(a) A strain gradient shall be assumed, corresponding to a strain in the extreme tensile reinforcement equal to 1.5 times the yield strain and a maximum strain in the masonry as given by 3.3.2(c).

(b) The design assumptions of Section 3.3.2 shall apply.

(c) The stress in the tension reinforcement shall be taken as the product of the modulus of elasticity of the steel and the strain in the reinforcement, and need not be taken as greater than f_y.

(d) Axial forces shall be taken from the loading combination given by $D + 0.75L + 0.525Q_E$.

(e) The effect of compression reinforcement, with or without lateral restraining reinforcement, shall be permitted to be included for purposes of calculating maximum flexural tensile reinforcement.

3.3.3.5.2 For intermediate reinforced masonry shear walls subject to in-plane loads where $M_u/V_u d_v \geq 1$, a strain gradient corresponding to a strain in the extreme tensile reinforcement equal to 3 times the yield strain and a maximum strain in the masonry as given by 3.3.2(c) shall be used. For intermediate reinforced masonry shear walls subject to out-of-plane loads, the provisions of Section 3.3.3.5.1 shall apply.

3.3.3.5.3 For special reinforced masonry shear walls subject to in-plane loads where $M_u/V_u d_v \geq 1$,

a strain gradient corresponding to a strain in the extreme tensile reinforcement equal to 4 times the yield strain and a maximum strain in the masonry as given by 3.3.2(c) shall be used. For special reinforced masonry shear walls subject to out-of-plane loads, the provisions of Section 3.3.3.5.1 shall apply.

3.3.3.5.4 For masonry members where $M_u/V_u d_v \leq 1$ and when designed using $R \leq 1.5$, there is no upper limit to the maximum flexural tensile reinforcement. For masonry members where $M_u/V_u d_v \leq 1$ and when designed using $R \geq 1.5$, the provisions of Section 3.3.3.5.1 shall apply.

6.4 Derivation of Flexural Strength Design Equations

6.4.1 Strength Design for Sections with Tension Steel Only

As stated above, limits for flexural design using strength methods are that the stress in the steel is at yield strength and the strain in the masonry is at either 0.0025 or 0.0035. When these conditions occur simultaneously, the section is considered to be a balanced design.

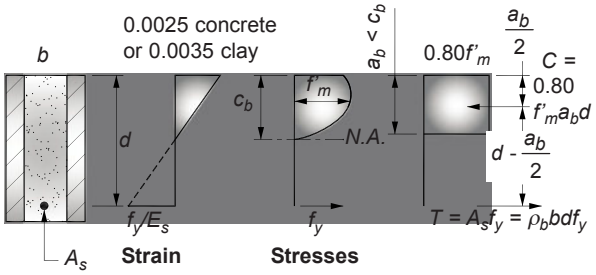


FIGURE 6.8 Masonry strain and stress blocks for a beam.

The depth to the neutral axis, c_b, for a balanced design is:

For Concrete Masonry:

With the compressive strain taken at 0.0025:

Using the similar triangles gives:

$$\frac{0.0025}{c_b} = \frac{0.0025 + \frac{f_y}{E_s}}{d}$$

Solving for c_b, gives

$$c_b = \frac{0.0025}{0.0025 + \frac{f_y}{E_s}} d = \frac{72{,}500}{72{,}500 + f_y} d$$

For Clay Masonry:

Likewise, with a compressive strain taken at 0.0035:

$$c_b = \frac{0.0035}{0.0035 + \frac{f_y}{E_s}} d = \frac{101{,}500}{101{,}500 + f_y} d$$

For f_y = 60,000 psi and for concrete masonry:

$$c_b = \frac{72{,}500}{72{,}500 + 60{,}000} d = 0.547d$$

For f_y = 60,000 psi and for clay masonry:

$$c_b = \frac{101{,}500}{101{,}500 + 60{,}000} d = 0.629d$$

The depth of the stress block for a balanced design, a_b is

$a_b = 0.80c_b$

$a_b = 0.80c_b = 0.80\ (0.547d)$

$= 0.438d$ for concrete masonry, and

$a_b = 0.80c_b = 0.80\ (0.629d)$

$= 0.503d$ for clay masonry.

Thus, the $0.438d$ and the $0.503d$ are the depths of the stress block for balanced conditions for concrete and clay masonry, respectively.

When design conditions are not at balanced conditions, the depth of the stress block will be less than a_b. The designation for the resulting depth of the stress block is a.

Equating the compression and tension forces

Compression force = $0.80f'_m ab$

Tension force = $A_s f_y = \rho bdf_y$

$C = T$

$0.80\ f'_m ab = \rho bdf_y$

Solve for a

$$a = \frac{\rho bdf_y}{0.80f'_m\, b}$$

$$= \rho\left(\frac{f_y}{f'_m}\right)\left(\frac{d}{0.80}\right)$$

The steel quotient is defined as

$q = \rho(f_y/f'_m)$

Therefore

$$a = \left(\frac{\rho f_y}{f'_m}\right)\left(\frac{d}{0.80}\right) = \frac{qd}{0.80}$$

The moment capacity of the section can be calculated:

$$M_n = C\left(d - \frac{a}{2}\right) = T\left(d - \frac{a}{2}\right)$$

$$M_n = 0.80f'_m ab\left(d - \frac{a}{2}\right) \qquad \text{(Masonry capacity)}$$

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) \qquad \text{(Steel capacity)}$$

Substituting masonry capacity in the equation yields:

$$a = \frac{\rho f_y d}{0.80f'_m}$$

$$M_n = 0.80f'_m b\left(\frac{\rho f_y d}{0.80f'_m}\right)\left(d - \frac{\rho f_y d}{2(0.80)f'_m}\right)$$

$$= \rho f_y bd^2\left(1 - \frac{0.625\rho f_y}{f'_m}\right)$$

Substituting $q = \rho f_y / f'_m$ and $\rho f_y = qf'_m$

$M_n = bd^2f'_mq(1 - 0.625q)$

The flexural coefficient K is then

$M_n = bd^2f'_mq(1 - 0.625q)$

$M_n = K_nbd^2$

$K_n = f'_mq(1 - 0.625q)$

Introducing the capacity reduction factor, ϕ, the equations are:

Nominal moment, $M_n \geq M_u / \phi$ and

$M_u \leq \phi bd^2f'_mq(1 - 0.625q)$

$M_u \leq \phi K_nbd^2 = K_ubd^2$

6.4.1.1 BALANCED STEEL RATIO

In order to insure that reinforcing steel will be stressed to yield before masonry achieves the strain limitation of 0.0025 or 0.0035 in./in., the amount of reinforcing steel in the section must be limited.

The definition of balanced design for strength design is that steel is stressed to yield strength just as the masonry achieves a strain of 0.0025 or 0.0035 in./in. for concrete and clay masonry, respectively.

The balanced steel ratio:

For Concrete Masonry:

With the compressive strain taken at 0.0025:

$$\rho_b = \frac{0.80(0.80)f'_m}{f_y}\left(\frac{72{,}500}{72{,}500 + f_y}\right)$$

For Clay Masonry:

With a compressive strain taken at 0.0035:

$$\rho_b = \frac{0.80(0.80)f'_m}{f_y}\left(\frac{101{,}500}{101{,}500 + f_y}\right)$$

For f_y = 60,000 psi, the balanced steel ratio is:

For Concrete Masonry:

$$\rho_b = \frac{0.80(0.80)f'_m}{60{,}000}\left(\frac{72{,}500}{72{,}500 + 60{,}000}\right)$$

$= 0.00000584f'_m$

For Clay Masonry:

$$\rho_b = \frac{0.80(0.80)f'_m}{60{,}000}\left(\frac{101{,}500}{101{,}500 + 60{,}000}\right)$$

$= 0.00000670f'_m$

Table 6.1 shows the values of ρ_b, for various f'_m values and for clay and concrete masonry materials.

The balanced steel ratio, $\rho_b = A_{sb} / bd$, can also be determined by balancing the tension and compression forces.

For Concrete Masonry, the compression force $= 0.80f'_m\,(0.438d)\,b$

$= 0.350\,f'_m\,bd$

For Clay Masonry, the compression force = $0.80f'_m(0.503d)b$

$= 0.402f'_m\ bd$

Tension force = $A_{sb}f_y = \rho_b bdf_y$

$C = T$

Thus, for concrete masonry: $0.350f'_m bd = \rho_b bdf_y$

$$\rho_b = \frac{0.350\ f'_m}{f_y}$$

and, for clay masonry: $0.402f'_m\ bd = \rho_b bdf_y$

$$\rho_b = \frac{0.402f'_m}{f_y}$$

TABLE 6.1 Balanced Steel Ratio ρ for f_y = 60,000 psi for concrete and clay masonry systems

f'_m	ρ_b for concrete masonry	ρ_b for clay masonry
1500	0.0088	0.0101
2000	0.0117	0.0134
2500	0.0146	0.0168
3000	0.0175	0.0201
3500	0.0204	0.0235
4000	0.0233	0.0268

EXAMPLE 6-A Balanced Steel Ratio, ρ_b.

Determine the steel ratio for a balanced design condition for strength design and compare to allowable stress design for both concrete and clay masonry.

Given:

Strength of masonry, f'_m = 1,500 psi;

Grade 60 steel, f_y = 60,000 psi

Solution 6-A

For strength design, balanced steel variable from Section 6.4.1.1.

For concrete masonry: ρ_b = 0.00000584 (1,500) = 0.0088 (Table 6.1).

For clay masonry: ρ_b = 0.00000670 (1,500) = 0.0101 (Table 6.1).

For comparison, allowable stress design Table ASD-24b for concrete masonry (f'_m = 1,500 psi and f_s = 24,000 psi) yields ρ_b = 0.00322. Table ASD-24a for clay masonry (f'_m = 1,500 psi, f_s = 24,000 psi) yield ρ_b = 0.00380.

The ratios of the rho balanced for strength design to rho balanced for allowable stress design for concrete masonry is 0.0088/0.00322 = 2.73 and the same ratio for clay masonry is 0.0101/0.00380 = 2.65. Thus, the balanced condition for strength design for concrete masonry requires 2.73 times the amount of steel as that for allowable stress design and likewise for clay masonry requires 2.65 times the amount of steel than for allowable stress design.

Example 6-A (as well as 6-B) show that more reinforcement is needed to achieve "balanced conditions" for strength design than for allowable stress design. However, the concept of balanced for ASD is based on allowable stress values; whereas, the concept of balanced for SD is based upon strain compatibility and equilibrium.

EXAMPLE 6-B Comparison of SD and ASD Balanced Steel Ratios.

Determine the balanced steel ratios by the strength design and allowable stress design methods when, f'_m = 3,000 psi and f_y = 60,000 psi for concrete and clay masonry.

Solution 6-B

For strength design, the balanced steel ratio from Section 6.4.1.1 or Table 6.1 is:

For concrete masonry: ρ_b = 0.00000584 (3,000) = 0.0175 (Table 6.1)

For clay masonry: ρ_b = 0.00000670 (3,000) = 0.0201 (Table 6.1)

For allowable stress design of concrete masonry when , f'_m = 3,000 psi and f_s = 24,000 psi, ρ_b = 0.0064 (from Table ASD-27b). For the clay masonry with the same f'_m and f_s, ρ_b = 0.0076 (from Table ASD-27a).

The ratios of the rho balanced for strength design to rho balanced for allowable stress design for concrete masonry is 0.00175/0.00644 = 2.72 and the same ratio for clay masonry is 0.0201/0.00761 = 2.64. Thus, the balanced condition for strength design for concrete masonry requires 2.72 times the

amount of steel as that for allowable stress design and likewise for clay masonry requires 2.64 times the amount of steel than that amount for allowable stress design.

Equation manipulations for the Example 6-C below are as follows:

$$a_u = \frac{K_u}{1200\rho} = \frac{M_u\,(\text{in.-lbs})}{12{,}000bd^2\rho} = \frac{M_u\,(\text{ft-k})}{bd^2\rho} = \frac{M_u}{A_s d}$$

and therefore, $A_s = \dfrac{M_u}{a_u d}$

EXAMPLE 6-C Depth of Beam and Reinforcing Steel.

Determine the beam depth and reinforcing steel for a nominal 8 in. concrete masonry beam to support a factored bending moment, M_u, of 90 ft kips.

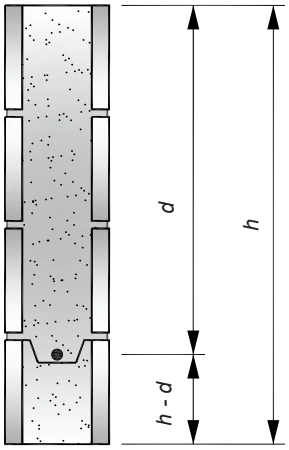


FIGURE 6.9 Beam layout for Example 6-C.

Solution 6-C

Assume

f'_m = 1,500 psi

f_y = 60,000 psi

For ductility and for an under-reinforced situation, select an initial trial steel ratio, $\rho = 0.5\,\rho_b$

From Table SD-2 for steel ratio of $0.5\rho_b$, $\rho = 0.5(0.0088) = 0.0044$; $a_u = 3.56$ and $K_u = 187.0$

$$bd^2 = \frac{M_u}{K_u} = \frac{90\,(1{,}000)(12)}{187} = 5775$$

b = 7.63 in.

$$d^2 = \frac{5775}{7.63} = 757$$

d = 27.5 in.

Use total depth (h) of 32 in., with $h - d = 4\,1/2$ in.

Determine reinforcement

$$A_s = \frac{M_u}{a_u d} = \frac{90}{3.56(27.5)} = 0.919 \text{ sq in.}$$

Alternate procedure, start with the trial ρ = 0.0044

$A_s = \rho bd$

= 0.0044 (7.63) (27.5)

= 0.923 sq in.

Use 1 - #9 bar (A_s = 1.00 sq in.)

or consider less reinforcement, try

2 - #6 (A_s = 0.88 sq in.)

Check capacity:

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) \qquad \text{(Steel capacity)}$$

with $a = \dfrac{\rho f_y d}{0.80 f'_m}$ and $\phi = 0.9$ so that $M_u = \phi M_n$

$$\rho = \frac{0.88}{7.63(27.5)} = 0.0042 < 0.0044$$

$$a = \frac{0.0042(60{,}000)(27.5)}{0.80(1{,}500)} = 5.77 \text{ in.}; \frac{a}{2} = 2.88 \text{ in.}$$

M_u = 0.9(0.88)(60,000)(27.5 - 2.88)

= 1,269,942 in.-lbs.

= 97.5 ft.-kips > 90 ft-kips; OK

Note that once a trial cross section is selected, the area of steel should be computed directly from finding ρ from the following sequence:

1. find K_u,
2. find ρ (from Table SD-2) and
3. find A_s (area of steel).

Or, use the procedure with Table SD-12. See Example 6-F for that procedure.

EXAMPLE 6-D Area of Steel, Strength Design.

What is the area of reinforcement required for a beam subjected to a factored moment of 150 ft kips? The beam is grouted clay masonry 9.5 in. wide by 48 in. deep. The d distance is 42 in., f'_m = 2500 psi, and f_y = 60,000 psi.

Solution 6-D

Determine the flexural coefficient

$$K_u = \frac{M_u}{bd^2} = \frac{150(1000)(12)}{9.5(42)^2} = 107.4$$

From Table SD-4 for K_u = 107.4;

ρ = 0.0023 and a_u = 3.86

$A_s = \rho bd$ = 0.0023 (9.5)(42) = 0.93 sq in.

Use 1 - # 9 bar (A_s = 1.00 sq in.)

EXAMPLE 6-E Moment Capacity.

If the beam in Example 6-D was reinforced with 2 - #8 bars, what would be its factored moment capacity? Consider the beam to be constructed using running bond and Type S portland cement/lime mortar.

Solution 6-E

$$\rho = \frac{A_s}{bd} = \frac{2\,(0.79)}{9.5\,(42)} = 0.0040$$

From Table SD-4 for ρ = 0.0040

K_u = 178.7

$M_u = K_u\, bd^2$

$$= \frac{178.7(9.5)(42)^2}{12{,}000} = 250 \text{ ft kips}$$

As a check on above answer using the full flexural calculation shows:

$$a = \frac{\rho f_y d}{0.80 f'_m} = \frac{(0.0040)(60{,}000)(42)}{(0.80)(2{,}500)} = 5.04$$

$$\frac{a}{2} = 2.52$$

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) = (1.58)(60{,}000)(42 - 2.52)$$

= 3,742,704 in. lbs = 311.9 ft-k

ϕM_n = 0.9(311.9) = 280.7 ft-k

Using solution 6-E as a basis, the increased capacity may be compared to the cracked moment capacity. The Modulus of Rupture is determined by using Table 6.2

TABLE 6.2 Modulus of Rupture (f_r) for Clay and Concrete Masonry, psi

Direction of flexural tensile stress and masonry type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints in running or stack bond				
Solid units	100	75	60	38
Hollow units[1]				
Ungrouted	63	48	38	23
Fully grouted	163	158	153	145
Parallel to bed joints in running bond				
Solid units	200	150	120	75
Hollow units				
Ungrouted and partially grouted	125	95	75	48
Full grouted	200	150	120	75
Parallel to bed joints in stack bond	0	0	0	0

1. For partially grouted masonry, modulus of rupture values shall be determined on the basis of linear interpolation between fully grouted hollow units and ungrouted hollow units based on amount (percentage) of grouting.
2. Based on MSJC Code Table 3.1.8.2.1.

From the table the Modulus of Rupture for a beam, where flexural tensile stress is parallel to bed joints in running bond with hollow units fully grouted and mortar is Type S, f_r = 200 psi.

$M_{cr} = S_n f_r$ (MSJC Code Section 3.1.8.2.1)

$$= \left[\frac{I}{c}\right] f_r = \left[\frac{\left(\frac{bh^3}{12}\right)}{\frac{h}{2}}\right] f_r = \frac{bh^2}{6} f_r$$

$$= \frac{9.5(48)^2}{6}(200) = 729{,}600 \text{ in. lbs (or 60.8 ft kips)}$$

where:

M_{cr} = cracking moment strength of the masonry section

S_n = section modulus

f_r = modulus of rupture as given in Table 6-2 and Table SD-24.

$$\frac{M_n}{M_{cr}} = \frac{311.9}{60.8} = 5.13 > 1.3$$

(required by MSJC Code Section 3.3.4.2.2.2)

The nominal flexural strength of the beam is sufficiently greater than the cracking strength.

EXAMPLE 6-F Design Aid Strength Design Table SD-12.

Using Table SD-12 determine the required steel area for a nominal 8 in. concrete masonry solid grouted beam carrying a live load of 3,000 plf and dead load including the weight of the beam of 2,000 plf, f'_m = 1,500 psi, f_y = 60,000 psi, d = 58 in. and overall depth = 64 in. The beam spans 20 ft.

MSJC Code Section 3.1.8.1.1 requires that the specified compressive strength of masonry, f'_m, shall equal or exceed 1,500 psi.

Solution 6-F

Factored loads: (Note that MSJC Code Section 3.1.2 mentions the load combinations and refers to ASCE 7-02 for the load factors)

$U = 1.2D + 1.6L$

w_u = 1.2(2,000) + 1.6(3,000)

= 7,200 plf

Factored moment, M_u

$$M_u = \frac{wl^2}{8} = \frac{7{,}200\,(20)^2}{8} = 360{,}000 \text{ ft lbs}$$

= 360 ft kips

Determine the steel requirement using Table SD-12

$$q\left(1 - 0.625q = \frac{M_u}{\phi bd^2 f'_m}\right)$$

$$= \frac{360{,}000\,(12)}{0.9\,(7.63)(58)^2\,(1{,}500)} = 0.1246$$

From Table SD-12 for $q(1 - 0.625q)$ = 0.1246

q = 0.136

Steel ratio

$$\rho = \frac{qf'_m}{f_y} = \frac{0.136(1{,}500)}{60{,}000} = 0.0034$$

$A_s = \rho bd$ = 0.0034 (7.63) (58)

= 1.51 sq in.

Using Table GN-20a, select 2 - #8 bars (A_s = 1.58 sq in.). Check whether 2 - #8 bars fit inside a lintel or bond beam block:

Width required = 2 (1.0 in. bars) + 1 (1.0 in. spacing) + 2 (1.25 in. face shells) + 2 (0.5 in. cover)

= 6.5 in. < 7.625 in. OK

To check the maximum amount of reinforcement, check MSJC Code Section 3.3.3.5, as follows:

$$\frac{M_u}{V_u d_v} = \frac{360{,}000(12)}{10(7{,}200)(58)} = 1.0344 > 1,$$

therefore MSJC Code Section 3.3.3.5.1 does apply.

For a beam and in a structure with $R \leq 1.5$ and

$\frac{M_u}{V_u d_v} > 1$, strain distribution (using similar triangles):

$$\frac{c}{d} = \frac{\varepsilon_m}{\varepsilon_m + \varepsilon_s}$$

ε_y = 60,000/29,000,000= 0.00207 in./in.

$$\frac{c}{d} = \frac{0.0025}{0.0025 + 1.5(0.00207)} = 0.446$$

c = 0.446 (58) = 25.87 in.

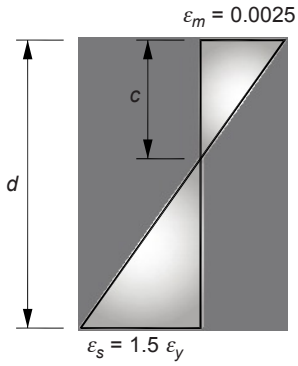


FIGURE 6.10 Strain similar triangles.

From equilibrium of $C = T$:

$A_{s\ max}$ (60,000) = 0.8 (1,500) (0.8) (25.87) (7.63)

$A_{s\ max}$ = 3.16 in.^{2} > 1.58 in.^{2} OK

For illustration purposes, consider the case where $M_u/V_u d_v \leq 1$; MSJC Code Section 3.3.3.5.1 no longer applies and ε_y is used rather than 1.5 ε_y. However, if $R \geq 1.5$, then MSJC Code Section 3.3.3.5.4 requires conformance to MSJC Code Section 3.3.3.5.1.

In the example, A_s = 1.58 in.^{2}. Using Table 6.3b for the case $R \geq 1.5$ the factor required from MSJC Code 3.3.3.5.1 would be 1.5 and ρ_{max} would be:

ρ_{max} = 0.0071

A_s = 0.0071 (7.63) (58) = 3.14 in.^{2} > 1.58, OK (reinforcement is less than the maximum allowed)

As an interesting comparison, the conventional past way of comparing the percents of a balanced condition, this beam would result in the following amounts in terms of the balanced percentages:

From the above derivations for ρb:

$$\rho_b = \frac{0.350 f'_m}{f_y}$$

$$\rho_b = \frac{0.350(1{,}500)}{60{,}000} = 0.0087$$

versus the actual $\rho = \dfrac{1.58}{7.63(58)} = 0.00357$

TABLE 6.3a & b Maximum Steel Ratio ρ_{max} for f_y = 60,000 psi and for clay and concrete masonry systems

TABLE 6.3a	**Steel Strain Factor to compute ρ_{max} for clay masonry**			
f'_m	**1.0**	**1.5[1]**	**3.0[2]**	**4.0[3]**
1500	0.0101	0.0085	0.0058	0.0048
2000	0.0134	0.0113	0.0077	0.0063
2500	0.0168	0.0141	0.0096	0.0079
3000	0.0201	0.0170	0.0115	0.0095
3500	0.0235	0.0198	0.0135	0.0111
4000	0.0268	0.0226	0.0154	0.0127
TABLE 6.3b	**Steel Strain Factor to compute ρ_{max} for concrete masonry**			
f'_m	**1.0**	**1.5[1]**	**3.0[1,2]**	**4.0[1,3]**
1500	0.0088	0.0071	0.0046	0.0037
2000	0.0117	0.0095	0.0061	0.0049
2500	0.0146	0.0119	0.0077	0.0062
3000	0.0175	0.0143	0.0092	0.0074
3500	0.0204	0.0167	0.0107	0.0087
4000	0.0233	0.0190	0.0122	0.0099

1. For MSJC Code Section 3.3.3.5.1 masonry members where $\dfrac{M_u}{V_u d_v} \geq 1$

2. For MSJC Code Section 3.3.3.5.2 intermediate reinforced masonry shear walls subject to in-plane loads

3. For MSJC Code Section 3.3.3.5.3 special reinforced masonry shear walls subject to in-plane loads

Therefore, the percentage of balanced

$$= \frac{0.00357}{0.0087} = 0.4103\rho_b$$

This percentage agrees with the conventional approach of an approximate level of ductility for an under-reinforced section.

Note in the above examples for the maximum area of reinforcement using the MSJC Code criteria, the amount of reinforcement can be summarized as follows:

1. From MSJC Code Section 3.3.3.5.1, for $R >$ 1.5 or $M_u/V_u d_v \geq 1$, for flexural members subject to in-plane or out-of-plane forces, the maximum ratio of reinforcement is:

$$\rho_{\max} = \frac{0.64f'_m\left(\frac{\varepsilon_m}{\varepsilon_{mu} + 1.5\varepsilon_y}\right) - \frac{P}{bd}}{f_y}$$

2. From MSJC Code Section 3.3.3.5.1, for $R >$ 1.5 or $M_u/V_u d_v \geq 1$, for walls subject to in-plane or out-of plane forces, the maximum area of reinforcement is:

$$A_{s_{\max}} = \rho_{\max} bd = \frac{064f'_m\, bd\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + 1.5\varepsilon_y} - P\right)}{f_y}$$

3. From MSJC Code Section 3.3.3.5.4, for $R <$ 1.5 and $M_u/V_u d_v \leq 1$, there is no limit to the maximum ratio of reinforcement. As a reasonable precaution for beam flexural members, the balanced condition should not be exceeded:

$\rho_{max} \leq \rho_{balanced}$

6.4.2 STRENGTH DESIGN FOR SECTIONS WITH TENSION AND COMPRESSION STEEL

The use of compression steel is very seldom required in masonry design. However, when there is steel in the compression stress block, it will contribute to the compression capacity of the section.

If more factored moment capacity is required than available by using the maximum permissible amount of steel, additional tension and compression steel can be added to provide the increased moment capacity, however, doing so may cause the section to be undersized, possibly causing excessive deflection or cracking.

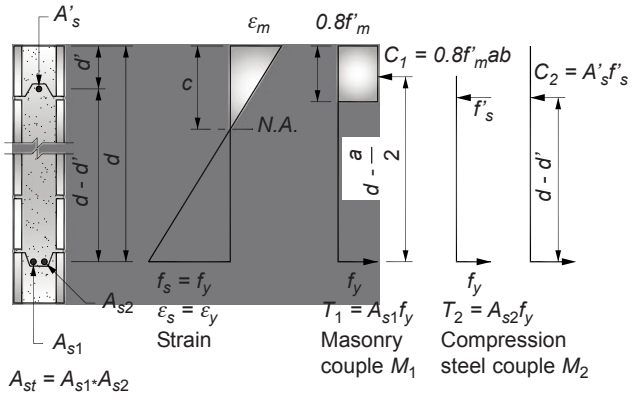


FIGURE 6.11 Strain, stress and moment diagram for flexural member with compression steel.

Factored moment capacity $M_u = \phi M_n = \phi(M_1 + M_2)$

Where: $\phi = 0.9$ for flexural conditions:

$$M_1 = T_1\left(d - \frac{a}{2}\right)$$

$M_2 = T_2\,(d - d')$

Calculate the moment, M_1, for a given amount of steel or a trial amount, say, for example, $0.5\rho_b$ for a member with tension steel only, or calculate the M_1 based upon the maximum area of steel for the singly-reinforced case. For example,

$A_{s1} = 0.5\ \rho_b bd$ $\qquad$ $T_1 = A_{s1}\ f_y$

c/d from SD Tables SD-2 through SD-7

$$c = d\left(\frac{c}{d}\right) \qquad a = 0.80c$$

Determine the value of M_2 as the difference between M_n and M_1. The moment arm is $(d - d')$.

The area of steel is based on the stress in the steel.

Tension steel $f_s = f_y$

Compression steel $f'_s \leq f_y$

Stress in the compression steel can be determined by the geometry of the maximum masonry strain of 0.0025 for concrete or 0.0035 for clay masonry, c distance to the neutral axis and the d' or $(c - d')$ value. The distance c is based on a flexural member with tension steel only.

Stress in compression steel:

f'_s = (strain) (E_s)= $\varepsilon_s E_s$

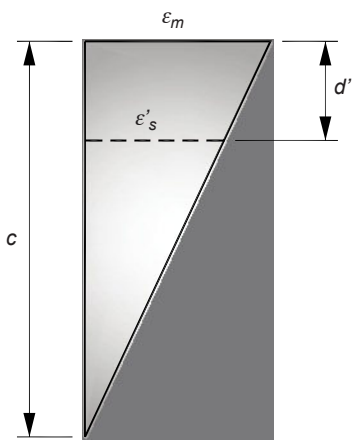


FIGURE 6.12 Compression strain.

Thus, for the concrete masonry:

$$f_s = 0.0025\left(\frac{c-d'}{c}\right)(29{,}000{,}000)$$

$$= 72{,}500\left(\frac{c-d'}{c}\right)$$

and, for the clay masonry:

$$f_s = 0.0035\left(\frac{c-d'}{c}\right)(29{,}000{,}000)$$

$$= 101{,}500\left(\frac{c-d'}{c}\right)$$

Where:

$$c = \frac{d\rho f_y}{0.80(0.80)f'_m}$$

$$c = \frac{d\rho f_y}{0.64f'_m}$$

for f_y = 60,000 psi

$$c = \frac{93{,}750d\rho}{f'_m}$$

For Concrete Masonry:

$$f'_s = 72{,}500\left[\frac{\left(\frac{93{,}750d\rho}{f'_m}\right)-d'}{\left(\frac{93{,}750d\rho}{f'_m}\right)}\right]$$

$$= 72{,}500\left(1-\frac{d'f'_m}{93{,}750d\rho}\right)$$

$$72{,}500-\frac{0.773d'f'_m}{d\rho}$$

For Clay Masonry:

$$f'_s = 101{,}500\left[\frac{\left(\frac{93{,}750d\rho}{f'_m}\right)-d'}{\left(\frac{93{,}750d\rho}{f'_m}\right)}\right]$$

$$= 101{,}500\left(1-\frac{d'f'_m}{93{,}750d\rho}\right)$$

$$= 101{,}500-\frac{1.083d'f'_m}{d\rho}$$

The additional tension steel is based on the yield stress, f_y.

$$A_{s2} = \frac{T_2}{f_y}$$

The compression steel area is based on either f_y or f_s if it is below yield strain.

$$A'_s = \frac{M_2}{(d-d')f'_s} = \frac{C_2}{f'_s}$$

EXAMPLE 6-G Area of Tension and Compression Steel:

Given an 8 in. CMU beam with 32 in. of total depth, and d = 26 in., d' = 4 in. and subjected to a factored moment; M_u, of 150 ft kips and factored shear, V_u, of 12 kips. Determine the area of tension steel and compression steel if required. f'_m = 2,000 psi, f_y = 60,000 psi, ϕ = 0.9

Solution 6-G

$M_u = \phi M_n$

$M_u = \phi(M_1 + M_2)$

$150 = 0.9(M_1 + M_2)$

$M_1 + M_2$ = 166.7 ft. kips

Select a trial steel ratio with ρ_b:

$0.5\rho_b$ = 0.5 (0.0117) = 0.00585

(See Table 6-1 for ρ_b)

$A_{s1} = \rho bd$ = 0.00585 (7.63) (26)

= 1.16 in.^{2}

$$a = \frac{T_1}{0.80f'_m b} = \frac{1.16\,(60{,}000)}{0.80\,(2{,}000)(7.63)} = 5.7\,\text{in.}$$

$$M_1 = T_1\left(d - \frac{a}{2}\right)$$

$$= \frac{1.16(60{,}000)\left(26 - \frac{5.7}{2}\right)}{12{,}000}$$

= 134.3 ft. kips

$M_2 = M_n - M_1$

= 166.7 - 134.3

= 32.4 ft kips

Check whether maximum reinforcement governs:

$$\frac{M_u}{V_u d_v} = \frac{150(12)}{12(26)} = 5.77 > 1$$

Since $M_u/V_u d_v > 1$, regardless of the value of R, MSJC Code Section 3.3.3.5.1 applies and a larger strain must be developed in the tension reinforcement.

Strain distribution (using similar triangles):

$$\frac{c}{d} = \frac{\varepsilon_m}{\varepsilon_m + \varepsilon_s}$$

$$\varepsilon_s = 1.5\varepsilon_y = \frac{1.5(60{,}000)}{29{,}000{,}000} = 0.0031$$

$$\frac{c}{d} = \frac{0.0025}{0.0025 + 0.0031} = 0.4464$$

c = 0.4464(26) = 11.61 in.

From equilibrium of $C = T$:

$A_{s\,max}$ (60,000) = 0.8(2,000)(0.8)(11.61)(7.63)

$A_{s\,max}$ = 1.89 in.^{2} > 1.16 in.^{2} OK

Additional tension steel

$$T_2 = \frac{M_2}{d - d'}$$

$$A_{s2} = \frac{T_2}{f_y} = \frac{M_2}{(d - d')f_y}$$

$$= \frac{32.4\,(12{,}000)}{(26 - 4)60{,}000} = 0.295\,\text{in.}^2$$

Tension steel = $A_{s1} + A_{s2}$

= 1.16 + 0.295 = 1.46 in.^{2}

Use 2 - #8 (A_s = 1.58 sq in.) (Note: tension reinforcement is selected after adding $A_{s1} + A_{s2}$ and is not selected separately).

Compression steel

Check stress in compression steel

$$c = \frac{a}{0.80} = \frac{5.7}{0.80}$$

= 7.12 in.

$$f'_s = 72{,}500\left(\frac{7.12 - 4}{7.12}\right)$$

= 31,770 psi

The additional compression force C_2 is:

$M_2 = M_n - M_1$ = 166.7 - 134.3 = 32.4 ft kips

$$C_2 = \frac{M_2}{d - d'} = \frac{32.4\,(12{,}000)}{26 - 4}$$

= 17,673 lbs

Compression steel = A'_s

$$A'_s = \frac{C_2}{f'_s} = \frac{17{,}673}{31{,}770}$$

= 0.556 in.^{2}

Use 2 - #5 (A'_s = 0.62 sq in.)

Check MSJC Code Section 3.3.3.5.1 requirements for maximum area of tension reinforcement. From strain distribution computed previously:

c = 0.4464 (26) = 11.61 in.

From equilibrium of $C = T$

$A_{s\,max}$ (60,000) = 0.8 (2,000) (0.8) (11.61) (7.63)

$A_{s\,max}$ = 1.89 in.^{2} > 1.58 in.^{2} OK

Note that even if this rough comparison fails, MSJC Code Section 3.3.3.5.1(e) allows compression reinforcement to be included in the computation for calculating maximum flexural tensile reinforcement. Doing so would increase the maximum tension reinforcement area allowed.

6.4.3 STRENGTH DESIGN FOR COMBINED AXIAL LOAD AND MOMENT

Many walls are subjected to combined vertical loads and moments due to dead and live loads plus lateral forces either in-plane or out-of-plane. Accordingly, design is based on parameters of strength design for factored loads, maximum allowable steel ratio and limitation of masonry strain.

6.4.3.1 DERIVATION FOR *P-M* LOADING

The following derivation is based on simple statics by summing the moments and the vertical forces to equal zero.

Derivation:

$$C\left(\frac{l}{2}-\frac{a}{2}\right)+T\left(\frac{l}{2}-d_1\right)-M=0$$

Sum of the moments about centroid of the load *P*.

Sum of the vertical forces

$T = C - P$

Substituting for T

$$C\left(\frac{l}{2}-\frac{a}{2}\right)+(C-P)\left(\frac{l}{2}-d_1\right)-M=0$$

but

$C = 0.80f'_m at$

substituting for C

$$(0.80f'_m at)\left(\frac{l}{2}-\frac{a}{2}\right)+(0.80f'_m at-P)\left(\frac{l}{2}-d_1\right)-M=0$$

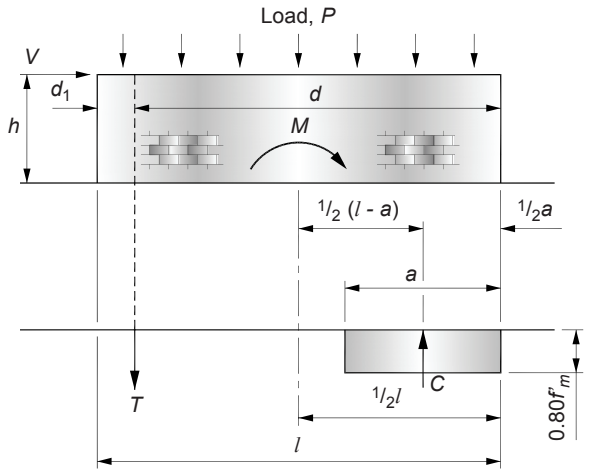


FIGURE 6.13 Shear wall with vertical and lateral load, stress conditions shown.

$$0.4f'_m atl - 0.4f'_m a^2 t + 0.4f'_m atl - 0.80f'_m atd_1 - P\left(\frac{l}{2}-d_1\right)-M=0$$

Change signs and combine terms

$$\underbrace{0.4f'_m t}_{a}\,a^2-\underbrace{0.80f'_m t(l-d_1)}_{b}a+\underbrace{P\left(\frac{l}{2}-d_1\right)+M}_{c}=0$$

Solving this quadratic equation for a

$\mathbf{a} = a^2 + \mathbf{b}a + \mathbf{c} = 0$

Let $\mathbf{a} = 0.4f'_m t$

$\mathbf{b} = -0.8f'_m t(l - d_1)$

Note $(l - d_1) = d$

$= 0.80f'_m td$

$$c = P\left(\frac{l}{2}-d_1\right)+M$$

Using the binomial formula to solve the quadratic equation

$$a=\frac{-\mathbf{b}-\sqrt{\mathbf{b}^2-4\mathbf{ac}}}{2\mathbf{a}}$$

$$a=\frac{0.80f'_m td-\sqrt{(-0.80f'_m td)^2-4(0.4f'_m t)\left(P\left(\frac{l}{2}-d_1\right)+M\right)}}{2(0.4f'_m t)}$$

Determining the size of the stress block *a*, calculate the compression force.

$$C = 0.80 f'_m a t$$

Determine the tension force

$$T = C - P$$

If the value is zero or negative, no tension steel is required. Use minimum steel in accordance with code requirements.

Calculate the area of steel

$$A_s = \frac{T}{f_y}$$

6.5 TALL SLENDER WALLS

6.5.1 GENERAL

In 1980 and 1981, the Structural Engineers Association of Southern California (SEAOSC) and the Southern California Chapter of the American Concrete Institute (ACI) conducted a major research testing program to develop criteria for the design of tall, slender walls. A total of 32 test panels were built with h'/t ranging from 30 to 57. Panels were tested with a typical eccentric vertical roof load applied to a steel ledger at the top. Lateral pressure was applied through an air bag which loaded the wall for its full height and width. Based on the test results, design techniques were developed and code requirements are included in the IBC and MSJC Code to reflect the performance of the walls in the test program.

This design criteria limits the deflection under service loads and requires ductile yield strength with factored loads. An acceptable design must satisfy both criteria.

FIGURE 6.14 Slender wall masonry panels ready to be tested.

6.5.2 SLENDER WALL DESIGN REQUIREMENTS

The MSJC Code parameters for slender walls are:

a) When the slenderness ratio exceeds 30, the vertical load stress is limited to a maximum of $0.05f'_m$ (MSJC Code Section 3.3.5.4).

b) Maximum lateral out-of-plane deflection due to service loads is $0.007h$.

Minimum f'_m = 1500 psi and maximum f'_m = 4000 psi (MSJC Code Section 3.1.8.1.1).

MSJC Code Section 3.3.5.4

3.3.5.4 *Walls with factored axial stress of $0.20 f'_m$ or less* — The procedures set forth in this Section shall be used when the factored axial load stress at the location of maximum moment satisfies the requirement computed by Eq. (3-23).

$$\left(\frac{P_u}{A_g}\right) \le 0.20 f'_m \qquad (3\text{-}23)$$

When the slenderness ratio exceeds 30, the factored axial stress shall not exceed $0.05f'_m$.

Factored moment and axial force shall be determined at the midheight of the wall and shall be used for design. The factored moment, M_u, at the midheight of the wall shall be computed using Eq. (3-24).

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u \qquad (3\text{-}24)$$

Where:

$$P_u = P_{uw} + P_{uf} \qquad (3\text{-}25)$$

The deflection due to factored loads (δ_u) shall be obtained using Eq. (3-30) and (3-31) and replacing M_{ser} with M_u.

The design strength for out-of-plane wall loading shall be in accordance with Eq. (3-26).

$$M_u \le \phi M_n \qquad (3\text{-}26)$$

The nominal moment shall be calculated using Eqs. (3-27) and (3-28) if the reinforcing steel is placed in the center of the wall.

$$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right) \qquad (3\text{-}27)$$

$$a = \frac{\left(P_u + A_s f_y\right)}{0.80 f'_m b} \qquad (3\text{-}28)$$

The nominal shear strength shall be determined by Section 3.3.4.1.2.

3.3.5.5 *Deflection design* - The horizontal midheight deflection, δ_s, under service lateral and service axial loads (without load factors) shall be limited by the relation:

$$\delta_s \leq 0.007\ h \qquad (3\text{-}29)$$

P-delta effects shall be included in deflection calculation. The midheight deflection shall be computed using either Eq. (3-30) or Eq. (3-31), as applicable.

(a) Where $M_{ser} < M_{cr}$

$$\delta_s = \frac{5M_{ser}h^2}{48E_mI_g} \qquad (3\text{-}30)$$

(b) Where $M_{cr} < M_{ser} < M_n$

$$\delta_s = \frac{5M_{cr}h^2}{48E_mI_g} + \frac{5\left(M_{ser} - M_{cr}\right)h^2}{48E_mI_{cr}} \qquad (3\text{-}31)$$

The cracking moment strength of the wall shall be computed using the modulus of rupture, f_r, taken from Table 3.1.8.2.1.

6.5.2.1 EFFECTIVE STEEL AREA

The vertical load on a wall acts as a reinforcing force and is therefore transformed into an equivalent steel area. The resulting effective steel area may be determined as:

$$A_s = \frac{P_u + A_sf_y}{f_y}$$

6.5.2.2 NOMINAL MOMENT STRENGTH

The nominal moment strength, M_n, of the wall is determined based on the following formulas:

$$M_n = A_sf_y\left(d - \frac{a}{2}\right)$$

$$a = \frac{P_u + A_sf_y}{0.80f'_m\,b} \qquad \text{(MSJC Code Eq 3-28)}$$

6.5.3 DESIGN OR FACTORED STRENGTH OF WALL CROSS-SECTION

The design strength provided by a reinforced masonry wall cross section is computed as the nominal strength multiplied by a strength reduction force, ϕ:

$$M_u \leq \phi\ M_n$$

and $$P_u < \phi\ P_n$$

Where

P_n = nominal axial strength for a cross section subjected to combined flexural and axial load

P_u = $P_{uw} + P_{uf}$

= factored wall load plus factored tributary floor or roof loads.

M_n = nominal moment strength for a cross section subjected to combined flexural and axial load.

M_u = factored moment on a section due to lateral loads and eccentric roof and wall loads causing combinations of flexure and axial load, ϕ = 0.9.

6.5.3.1 DEFLECTION CRITERIA

The mid-height deflection is limited so that a serviceable wall is designed. The maximum deflection permitted by the MSJC Code is:

$$\delta_s \leq 0.007h \qquad \text{(MSJC Code Eq 3-29)}$$

The maximum deflections allowed are thus directly proportional to the height of the wall.

This limitation is based on the capability of the wall to deflect elasticity to at least δ_s and still rebound to its original vertical position. This recognizes that the wall may crack but will not impair the structural capacity. The SEAOSC/ACI committee recommended a deflection criteria of 0.01h but this was reduced when it was adopted by the MSJC Code from the provisions of the UBC.

6.5.3.2 DEFLECTION OF WALL

Lateral and vertical service loads (unfactored) are used in computing the maximum horizontal deflection, which typically occurs at the mid-height of the wall.

Secondary moments induced by deflections at the mid-height of the wall are represented in the deflection calculation.

Phi (ϕ) factors are not used in the deflection calculation since deflections result from unfactored loads and moments. The load-deflection relation for

walls is assumed to follow a curve similar to Figure 6.15.

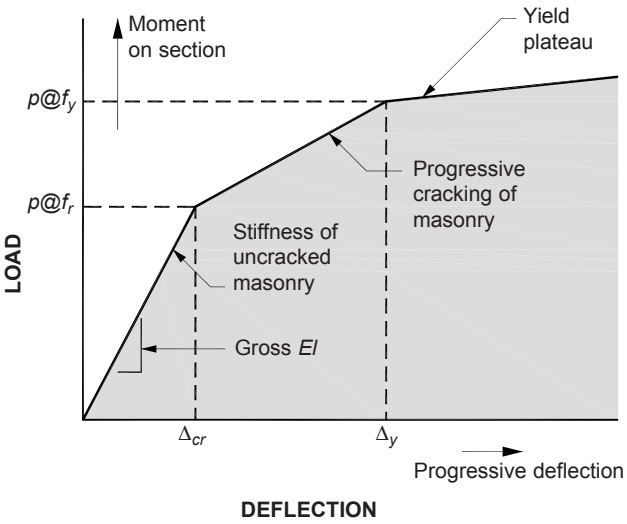


FIGURE 6.15 Load - deflection curve for a wall.

Slopes of the straight line parts of the load-deflection curve are as follows:

(a) up to cracking load, the gross section moment of inertia, I_g, is used to compute deflection from the load;

(b) additional deflection beyond the cracking load is computed using the cracked moment of inertia, I_{cr}.

Deflection of the wall at mid-height is determined by the following formula or an equivalent procedure.

Deflection at service load, δ_s:

$$\delta_s = \frac{5M_{cr}h^2}{48E_mI_g} + \frac{5(M_{ser} - M_{cr})h^2}{48E_mI_{cr}}$$

(MSJC Code Eq 3-31)

Where

M_{ser} = service moment on the masonry wall

M_{cr} = cracking moment strength of the masonry wall

Gross moment of inertia, I_g, Solid Grouted

$$I_g = \frac{bt^3}{12}$$

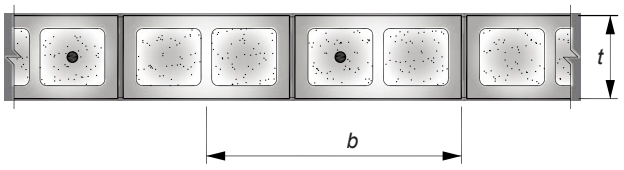


FIGURE 6.16 Moment of inertia factors for solid wall.

Cracked moment of inertia, I_{cr}

$$I_{cr} = nA_{se}(d - c)^2 + \frac{bc^3}{3}$$

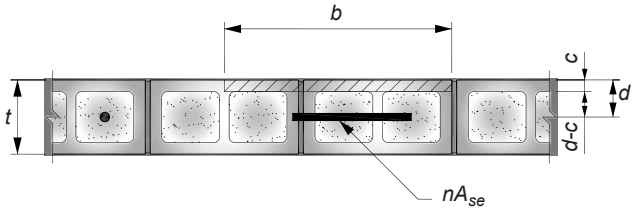


FIGURE 6.17 Moment of inertia factors for cracked wall.

Distance to neutral axis, $c = \frac{a}{0.80}$

Service moment, M_s

$$M_s = \frac{wh^2}{8} + P_f\left(\Delta + \frac{e}{2}\right) + P_w\frac{\Delta}{2}$$

Where

w = unfactored lateral service load

P_f = unfactored load on the ledger from tributary floor or roof loads

e = eccentricity of the ledger load

P_w = unfactored weight of wall

Δ = deflection due to load and weight of wall

6.5.4 DETERMINATION OF MOMENTS AT THE MID-HEIGHT OF THE WALL

Moment at the mid-height of the wall can be determined using statics. Consider the wall support and free body diagrams shown in Figure 6.18.

The horizontal force at the roof line, H_t is found by summing moments about B.

Where

H_t = horizontal force at the roof line

w = lateral load acting on the wall

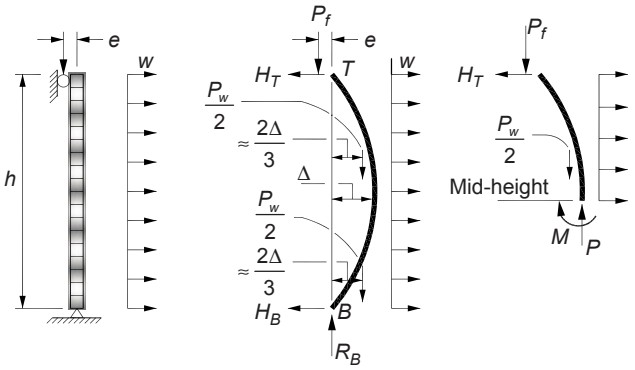


FIGURE 6.18 Wall support and free body diagrams.

P_w = weight of the wall

P_f = load at the roof line

e = eccentricity of the roof load

By summing moments about the wall mid-height, the relation for mid-height moment, M, is obtained.

$$M = \frac{wh^2}{8} + \frac{P_w\Delta}{2} + P_f\Delta + \frac{P_f e}{2}$$

6.6 SLENDER WALL DESIGN EXAMPLE

6.6.1 GENERAL

The design example given below considers a partially grouted 8 in. CMU wall. Note that a partial grouted, wall has lower lateral earthquake loads imposed on it as compared to a solid grouted wall.

The key to slender wall design is the assumption for the required steel reinforcement. The use of design aids will significantly reduce design time. Computer programs are also available which make slender wall design fast and simple.

EXAMPLE 6-H Strength Design of wall, *h/t* = 36.2.

Using the slender wall design method given in MSJC Code Section 3.3.5, design the reinforcing steel and check the wall for compliance to service load deflection and factored strength requirements.

Given: Partially grouted 8 in. nominal CMU wall, in SDC D.

ϕ = 0.9

f'_m = 1,500 psi

f_y = 60,000 psi

The wall spans 23 ft between lateral supports. The roof load is, P_f = 500 plf at an eccentricity of 7.3 in. and the seismic design service load is w_s = 15.9 psf acting horizontally perpendicular to the wall.

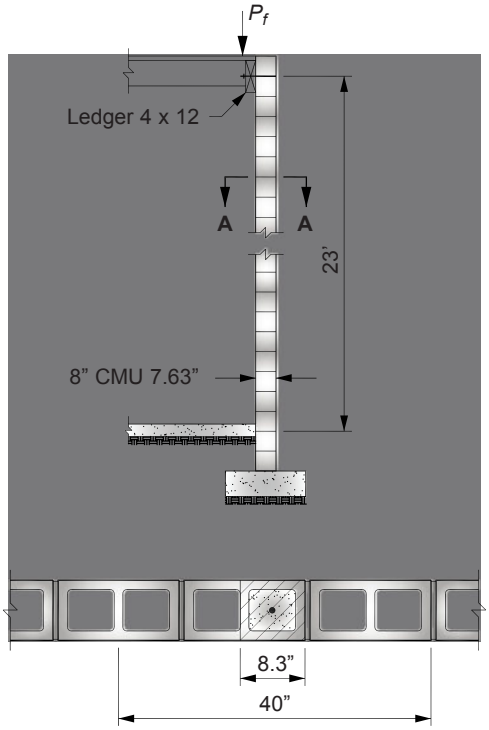


FIGURE 6.19 Slender wall cross-sections.

Solution 6-H Using the *P*-Δ Method

Assume steel is spaced 40 in. o.c. and grouted only at steel, this is based on the estimating curves given in the references.

1. Loads

   a) Weight of wall: assume medium weight CMU grouted at 40 in. o.c.; W_t = 53 psf from Table GN-3a

$$P_w = \frac{53\,(23)}{2} = 610 \text{ plf}$$

$$= \frac{610\,(40)}{12} = 2034 \text{ lbs / 40 in.}$$

$$\text{Roof load} = \frac{500\,(40)}{12} = 1667 \text{ lbs / 40 in.}$$

b) Lateral load

Seismic load, E, for Seismic Design Category D is (as provided in the problem statement):

$E = w_s$ = 15.9 psf (given)

c) Factored loads; $U = 0.9D + 1.0E$ (IBC Eq 16-7)

Factored wall load

P_{uw} = 0.9 (2034) = 1831 lbs/40 in.

Factored roof load

P_{uf} = 0.9 (1667) = 1,500 lbs/40 in.

Factored vertical loads

$P_u = P_{uf} + P_{uw}$

= 1,500 + 1,831

= 3,331 lbs/40 in.

Slenderness ratio:

$\frac{h}{t} = \frac{23(12)}{7.63} = 36.2 > 30$, thus the factored axial stress shall not exceed $0.05f'_m$ per MSJC Code Section 3.3.5.4

Check axial load limitation:

A_g = (40 -8.3)(2)(1.25) + (8.3)(7.63) = 142.6 in.^{2}

$\frac{P_u}{A_g} \le 0.05f'_m$ (MSJC Code Section 3.3.5.4)

$\frac{3351}{142.6} < 0.05(1500)$

23.5 psi < 75 psi OK

Factored seismic load

$w_u = 1.0E = 1.0\frac{(15.9)(40)}{12}$

= 53 lbs / 40 in. Assume the building is a standard occupancy; I = 1.0.

2. Assume vertical steel

The maximum amount of steel by MSJC Code Section 3.3.3.5 is determined as follows:

$\frac{c}{d} = \frac{\varepsilon_m}{\varepsilon_m + \varepsilon_s}$

$\varepsilon_s = 1.5\varepsilon_y = \frac{1.5(60{,}000)}{29{,}000{,}000} = 0.0031$

(This is for all walls with out-of-plane loading)

$\frac{c}{d} = \frac{0.0025}{0.0025 + 0.0031} = 0.4464$

c = 0.4464(3.81) = 1.7 in.

From equilibrium of $C = T$:

$A_{s\,max}$ (60,000) = 0.8 (1500) (0.8) (1.7) (40)

$A_{s\,max}$ = 1.088 in.^{2}

Assume #6 bars at 40 in. o.c., A_s = 0.44 in.^{2}

a) Gross steel ratio (see Table GN-24a)

$\rho_g = \frac{A_s}{bt} = \frac{0.44}{40(7.63)}$

= 0.00144

3. Determine E_m, n, f_r and I_g

a) Modulus of Elasticity, E_m

$E_m = 900f'_m$

= 900 (1500)

= 1,350,000 psi

b) Modulus ratio, n

$n = \frac{E_s}{E_m}$

$= \frac{29{,}000{,}000}{1{,}350{,}000} = 21.5$

c) Modulus of rupture, f_r (Table SD-24)

f_r = 63 psi for ungrouted and f_r = 163 psi for fully grouted (MSJC Code Table 3.1.8.2.1). Interpolation is allowed by MSJC Code based upon the percentage of partial grouting.

Thus, if one bar is placed every 40 in., then one cell in five is grouted, so an approximate percentage of grouting is 20%. Therefore, the interpolation gives:

f_r = 0.2 (163 - 63) + 63 = 83 psi.

d) Gross moment of Inertia, I_g

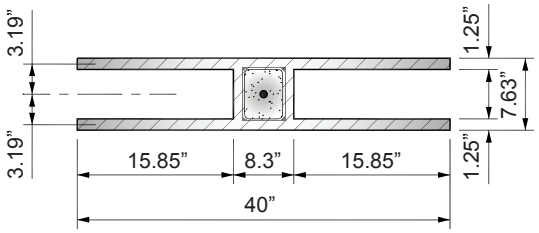


FIGURE 6.20 Moment of inertia of partial grouted wall.

$$I_g = \frac{8.3(7.63)^3}{12} + 2\left(\frac{31.7(1.25)^3}{12} + 31.7(1.25)(3.19)^2\right)$$

= 307.2 + 2(5.2 + 403.2)

= 1124 in.^{4} /40 in.

4. Moment at cracking, M_{cr}

$M_{cr} = Sf_r$

but $S = I/c$ where $c = t/2$

$$M_{cr} = \frac{2I_g f_r}{t} = \frac{2(1124)(83)}{7.63}$$

= 24,454 in. lbs / 40 in.

5. Cracked moment of Inertia

Calculate I_{cr} using the modular ratio, n, to transform the effective reinforcing steel into an equivalent area of masonry, and by using the expression:

$$I = \frac{bh^3}{12} + Ad^2$$

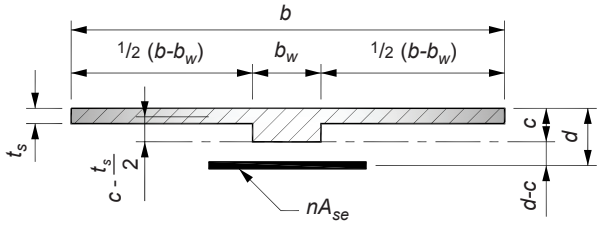


FIGURE 6.21 Moment of inertia of cracked partial grouted wall.

$$I_{cr} = \frac{(b - b_w)t_s^3}{12} + t_s(b - b_w)\left(c - \frac{t_s}{2}\right)^2 + \frac{b_w c^3}{3} + nA_{se}(d - c)^2$$

a) $$A_{se} = \frac{P_u + A_s f_y}{f_y}$$

$$= \frac{3331 + 0.44\,(60{,}000)}{60{,}000}$$

= 0.50 sq in. / 40 in.

b) Depth of rectangular stress block a

$$a = \frac{P_u + A_s f_y - 0.80f'_m\,(b - b_w)\,t_s}{0.80f'_m\,b_w}$$

$$= \frac{3331 + 0.44(60{,}000) - 0.80(1500)(40 - 8.3)(1.25)}{0.80(1500)\,8.3}$$

= -1.79

This results in a negative value. Therefore the stress block is completely in the shell.

$$a = \frac{P_u + A_s f_y}{0.80f'_m\,b}$$

$$= \frac{3331 + 0.44(60{,}000)}{0.80(1500)\,40}$$

= 0.62 in. < 1.25 (face shell thickness)

c) Distance to Neutral Axis, c

$$c = \frac{a}{0.80} = \frac{0.62}{0.80}$$

= 0.78 in.

Therefore - Cracked moment of inertia

$$I_{cr} = nA_{se}(d - c)^2 + \frac{bc^3}{3}$$

$$= 21.5(0.50)(3.81 - 0.78)^2 + \frac{40(0.78)^3}{3}$$

= 98.7 + 6.3

= 105 in.^{4} /40 in.

6. Calculate mid-height moment, M_s, and lateral deflection, δ_s, due the service loads by iteration method (w_s for service load is 15.9 psf assume to be factored)

   a) First iteration, assume δ_s = 0 in.

$$M_{s1} = \frac{wh^2}{8} + P_o\left(\frac{e}{2}\right) + (P_o + P_w)\delta_s$$

$$= \frac{15.9\left(\frac{40}{12}\right)(23)^2(12)}{8} + 1667\left(\frac{7.3}{2}\right) + 0$$

= 42,056 + 6085 + 0

= 48,141 in. lbs / 40 in. > M_{cr}

$$\delta_{s1} = \frac{5M_{cr}h^2}{48E_mI_g} + 5\frac{(M_{ser} - M_{cr})h^2}{48E_mI_{cr}}$$

(MSJC Code Eq 3-31)

$$\delta_{s2} = \frac{5(24{,}454)(23(12))^2}{48(1{,}350{,}000)(1{,}124)} + \frac{5(23(12))^2(48{,}141 - 24{,}454}{48(1{,}350{,}000)(105)}$$

= 0.128 + 1.326

= 1.454 in.

   b) Second iteration; Δ_s = 1.454 in.

M_{s2} = 42,056 + 6085 + (1667 + 2034) 1.454

= 53,522 in. lbs / 40 in. width

$$\delta_{s2} = \Delta_{s3} = 0.128 + \frac{5(23.33(12))^2(53{,}522 - 24{,}454)}{48(1{,}350{,}000)(105)}$$

= 0.128 + 1.627

= 1.755 in.

   c) Third iteration Δ_s = 1.755 in.

M_{s3} = 42,056 + 6085 + 3701(1.755)

= 54,637 in. lbs / 40 in. width

$$\delta_{s3} = \Delta_{s4} = 0.128 + \frac{5(23.33(12))^2(54{,}637 - 24{,}454)}{48(1{,}350{,}000)(105)}$$

= 0.128 + 1.690

= 1.818 in.

   d) Convergence based on deflection

$$\left(\frac{\Delta_{s3} - \Delta_{s2}}{\Delta_{s3}}\right)100 = \left(\frac{1.818 - 1.755}{1.818}\right)100$$

= 3.43% Satisfactory

7. Check lateral deflection allowance at service load

Allowable Δ_s = 0.007 h

= 0.007 (23) (12)

= 1.93 in.

Actual Δ_{s3} = 1.818 in. < 1.93 in. OK.

The service load deflection of 1.818 in. is less than the maximum allowable deflection of 1.93 inches. Therefore, the deflection criteria is satisfied. Although the deflection is not technically a story drift, the story drift limit provides important criteria in providing for separation requirements for structures and components of structures.

8. Strength calculation - based on a 40 in. width. Calculate mid-height moment under factored loads

$$M_u = \frac{w_u h^2}{8} + P_{uf}\left(\frac{e}{2}\right) + P_u\delta_u$$

(MSJC Code Eq 3-24) (with $\delta_u = \Delta_u$)

   a) First iteration; Assume Δ_u = 0

$$M_{u1} = \frac{15.9(40)(23)^2}{8} + \frac{1500\ (7.3)}{2} + 3331(0)$$

= 42,056 + 5475 + 0

= 47,531 in lbs / 40 in. width

$$\Delta_{u1} = \frac{5M_{cr}h^2}{48E_mI_g} + \frac{5(M_{u1} - M_{cr})h^2}{48E_mI_{cr}}$$

$$= \frac{5\ (24{,}454)\ [(23)(12)]^2}{48(1{,}350{,}000)(1124)} + \frac{5(47{,}531 - 24{,}454)[(23)(12)]^2}{48(1{,}350{,}000)(105)}$$

= 0.128 + 0.0000560 (47,531 - 24,454)

= 0.128 + 1.292

= 1.420 in.

b) Second iteration Δ_{u1} = 1.420 in.

M_{u2} = 42,056 + 5,475 + 3,331(1.420)

= 52,260 in. lbs / 40 in. width

Δ_{u2} = 0.128 + 0.0000560 (52,260 - 24,454)

= 0.128 + 1.557

= 1.685 in.

c) Third iteration Δ_{u2} = 1.685 in.

M_{u3} = 42,056 + 5,475 + 3,331(1.685)

= 53,142 in. lbs / 40 in. width

Δ_{u3} = 0.128 + 0.0000560 (53,142 - 24,454)

= 0.128 + 1.606

= 1.734 in.

d) Convergence based on deflection

$$\left(\frac{\Delta_{u3} - \Delta_{u2}}{\Delta_{u3}}\right)100 = \left(\frac{1.734 - 1.685}{1.734}\right)100$$

= 2.85% Satisfactory

9. Determine nominal strength of wall, M_n

$$M_n = 0.80 f'_m ab\left(d - \frac{a}{2}\right)$$

$$= 0.80 \text{ x } 1500 \text{ x } 0.62 \text{ x } 40\left(3.81 - \frac{0.62}{2}\right)$$

= 104,160 in. lbs / 40 in.

ϕM_n = 0.9 (104,160)

= 93,744 in. lbs / 40 in.

$\phi M_n > M_u$

93,744 in. lbs > 53,142 in. lbs

Therefore, the section is adequate for strength.

6.6.2 ALTERNATE METHOD OF MOMENT DISTRIBUTION

Moment and deflection calculations shown in Example 6-H are based on the MSJC Code equations given in Section 3.3.5.4 which assumes simple support conditions, top and bottom, with the maximum moment and deflection occurring at mid-height.

Other support and fixity conditions may be used and the resulting moments and deflections may be calculated using established principles of mechanics.

For instance, assume a wall is fully fixed at the bottom and designed as a pinned cantilever.

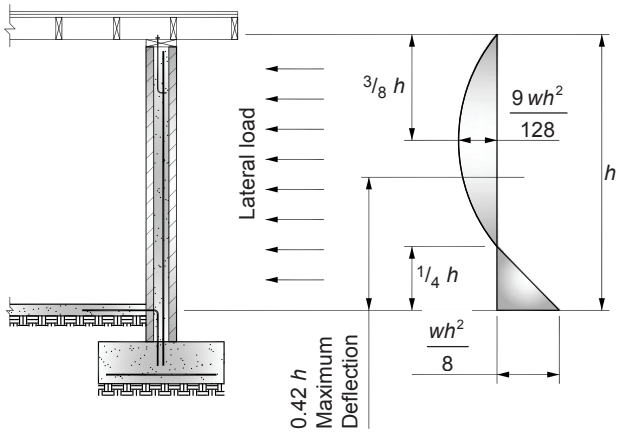


FIGURE 6.22 Slender wall fixed at bottom and pinned at top.

Under a uniform pressure, *w*, the moment at the base of the wall is $wh^2/8$. For this case the point of zero moment occurs at 0.25*h* and the maximum moment in wall is $9wh^2/128$ which occurs at 5*h*/8. The maximum deflection occurs at 0.4215*h* from the top and is determined by the equation.

$$\Delta_{max} = \frac{wh^4}{185EI}$$

This deflection is less than that of a simple span which is $5wh^4/48EI$ or about nineteen times as great.

Using this method the lower section of the wall can be reinforced for maximum moment while significantly less reinforcing steel is required in the upper part of the wall.

6.7 STRENGTH DESIGN OF SHEAR WALLS

6.7.1 GENERAL

Load bearing masonry walls support vertical and lateral loads. These loads create an interaction of load and moment on a wall. The strength design techniques for this condition are outlined in MSJC Code Section 3.3.6.

The IBC and MSJC Code provide appropriate load factors to be used and prescribe the conditions

for reinforcement, the hinge region and the required confinement for overturning steel.

Strength design procedures for shear walls allow masonry and reinforcing steel to resist shear forces even when the shear stress exceeds the capacity of the masonry. This reinforcement condition as prescribed in MSJC Code Section 3.3.6, refers back to shear strength computation given in MSJC Code Section 3.3.4.1.2 for the nominal shear strength as given by MSJC Code Equation 3-18. The reinforcement determined from the V_s computations needs to be coordinated with the shear wall types prescribed in the IBC and MSJC Code Section 1.14 for the various seismic design categories for shear walls resisting earthquake loads.

Phi, Φ, strength considerations

MSJC Code Section 3.1.4.3 provides for shear wall design strength, in terms of axial force, shear and moment, as the nominal strength multiplied by 0.80, the applicable strength reduction factor, Φ. For shear walls, MSJC Code Section 3.3.4.1.1 applies for axial and strength capacities. Thus, axial capacities given by MSJC Code Equations 3-16 and 3-17 apply.

For interaction diagrams, a balanced condition is needed.

For solid grouted walls, the value of P_b for the balanced condition may be calculated by:

$$P_b = 0.80\, f'_m b a_b$$

Where:

$$a_b = 0.80d\left(\frac{e_{mu}}{e_{mu} + \frac{f_y}{E_s}}\right)$$

The value of ϕ is 0.80 for any shear wall when the nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength for the factored-load combination. Maximum usable strain, e_{mu}, at the extreme masonry compression fiber is 0.0025 for concrete masonry and 0.0035 for clay masonry for design of beams, piers, columns and walls. The value of f'_m shall not be less than 1,500 psi or greater than 4,000 psi.

Reinforcement

Reinforcement requirements are:

1. Minimum reinforcement shall be provided in accordance with MSJC Code Section 1.14 for all seismic areas.
2. The amount of vertical reinforcement shall not be less than one half the horizontal reinforcement.
3. Other reinforcement provisions for shear walls are shown below (MSJC Code Section 1.14).

Terminology of the shear wall types in MSJC Code Section 1.14 are compatible with ASCE 7 and IBC. The five shear wall types are as follows:

- Ordinary plain (unreinforced) masonry shear walls,
- Detailed plain (unreinforced) masonry shear walls,
- Ordinary reinforced masonry shear walls,
- Intermediate reinforced masonry shear walls,
- Special reinforced masonry shear walls.

The resulting minimum prescriptive reinforcement in order to satisfy the above types are shown in Figures, 6.23, 6.24, 6.25 and 6.26:

Axial strength

The nominal axial strength of the shear wall supporting axial loads only without a reduction for slenderness effects can be calculated by:

$$P_n = 0.80 f'_m (A_e - A_s) + f_y A_s$$

However, MSJC Code Section 3.3.4.1.1 requires that slenderness also be taken into account. Thus, the axial load capacity is given by MSJC Code Equations 3-16 and 3-17 for the corresponding *h/r* limits as:

For members having an *h/r* ratio not greater than 99:

$$P_n = 0.80\left[0.80 f'_m (A_n - A_s) + f_y A_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

(MSJC Code Eq 3-16)

For members having an *h/r* ratio greater than 99:

$$P_n = 0.80\left[0.80 f'_m (A_n - A_s) + f_y A_s\right]\left(\frac{70r}{h}\right)^2$$

(MSJC Code Eq 3-17)

Axial design strength provided by the shear wall cross section shall satisfy:

$$P_u \le \phi P_n = 0.80 P_n$$

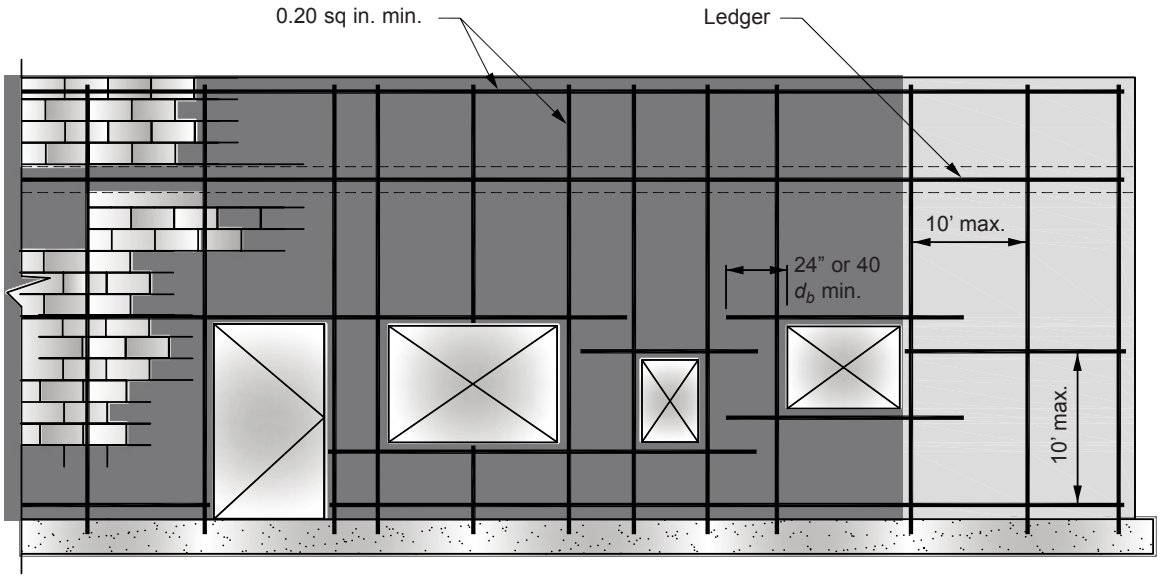


FIGURE 6.23 **Minimum reinforcement for Ordinary Reinforced Masonry Shear Wall - SDC C.**

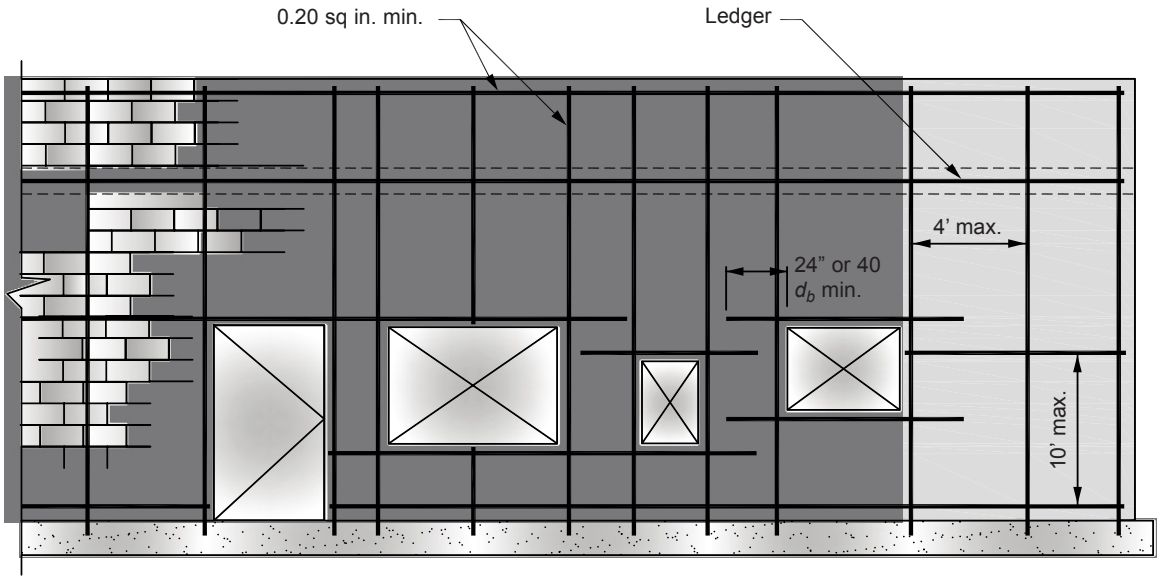


FIGURE 6.24 **Minimum reinforcement for Intermediate Reinforced Masonry Shear Wall - SDC C.**

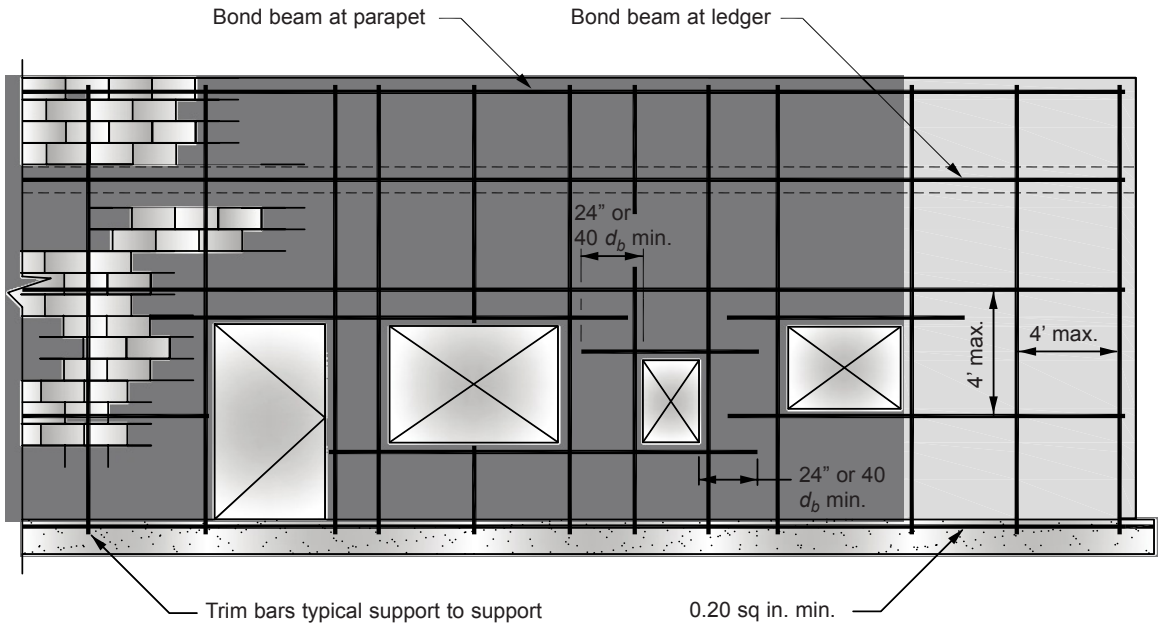


FIGURE 6.25 Minimum reinforcement for Special Reinforced Masonry Shear Wall - SDC C and above.

A_s = 0.0015 A_e in open ended bond beams for walls that are **not** part of the lateral-force-resisting system.

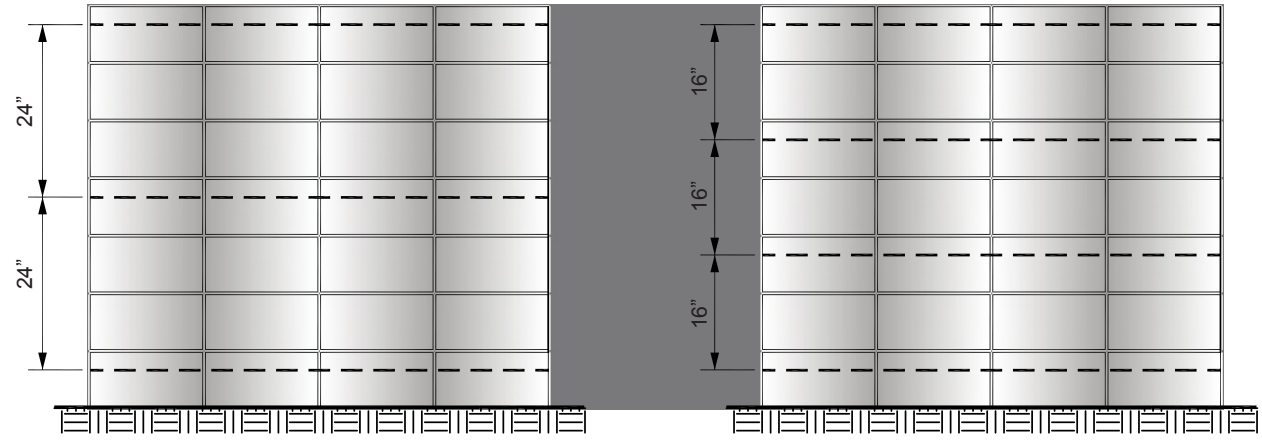
Maximum spacing of horizontal reinforcement should be 24".

Element should be solidly grouted and constructed of hollow open-end units or two wythes of solid units.

A_s = 0.0025 A_e in open ended bond beams for walls that are part of the lateral-force-resisting system.

Maximum spacing of horizontal reinforcement should be 16".

Element should be solidly grouted and constructed of hollow open-end units or two wythes of solid units.



a) *Minimum reinforcement for stack bond element that are **not** part of the lateral-force resisting system - SDC E.*

b) *Minimum reinforcement for stack bond element that are part of the lateral-force resisting system - SDC E.*

FIGURE 6.26 Minimum horizontal reinforcement in stack bond masonry walls - SDC E.

Shear strength

Shear strength shall be as follows:

The nominal shear strength is determined using:

$$V_n = V_m + V_s \qquad \text{(MSJC Code Eq 3-18)}$$

Maximum nominal shear strength values may be determined from Table SD-26. The nominal shear strength of the shear wall is determined from MSJC Code Equation 3-18, where V_n shall not exceed the following:

Where: $\frac{M_u}{V_u d_v} \le 0.25$:

$$V_n \le 6A_n\sqrt{f'_m}$$

Where: $\frac{M_u}{V_u d_v} > 1.00$:

$$V_n \le 4A_n\sqrt{f'_m}$$

The value of V_n for $M_u/V_u d_v$ between 0.25 and 1.00 may be interpolated.

Nominal masonry shear strength, V_m, is computed using:

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u$$

(MSJC Code Eq 3-21)

$M_u/V_u d_v$ need not be taken greater than 1.0, but it must be a positive number.

Nominal shear strength provided by shear reinforcement, V_s, is computed using:

$$V_s = 0.5\left(\frac{A_v}{s}\right)f_y d_v$$

Boundary elements

Boundary elements may be provided at the boundaries or extremities of shear walls when the maximum areas of flexural tensile reinforcement in the wall exceeds the provisions of MSJC Code Section 3.3.3.5. Special boundary elements are not required when the following conditions are met:

1. $P_u \le 0.10\ A_g f'_m$ for geometrically symmetrical wall sections and

   $P_u \le 0.05\ A_g f'_m$ for geometrically unsymmetrical wall sections; and either

2. $\frac{M_u}{V_u l_w} \le 1.0$ or

3. $V_u \le 3A_n\sqrt{f'_m}$ and $\frac{M_u}{V_u l_w} \le 3.0$

Based on MSJC Code Section 3.3.6.8, special boundary elements in walls bending in single curvature for which the flexural limit state response is governed by yielding at the base of the wall, are provided over portions of compression zones where:

$$c \ge \frac{l_w}{600\left(\frac{C_d\delta_{ne}}{h_w}\right)}$$

The term c is calculated for the P_u given by ASCE 7-02 Load Combination ($1.2D + 1.0E + L + 0.2S$) or the corresponding strength design load combination of the adopted building code, and the corresponding nominal moment strength, M_n, at the base critical section. Where required by MSJC Code Section 3.3.6.8, special boundary elements shall extend vertically from the critical section a distance not less than the larger of l_w or

$$\frac{M_u}{4V_u}$$

Shear walls not designed by MSJC Code Section 3.3.6.8 shall have special boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect, exceeds $0.2f'_m$. The special boundary element may be discontinued where the calculated compressive stress is less than $0.15f'_m$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in MSJC Code Section 1.9.4.2.3 is used.

Where special boundary elements are required, the following requirements must be satisfied with tests performed to verify the strain capacity of the element:

a. The special boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of (c-$0.1l_w$) and $c/2$.

b. In flanged sections, the special boundary element shall include the effective flange width in compression and shall extend at least 12 in. into the web.

c. Special boundary element transverse reinforcement at the wall base shall extend into the support a minimum of the development length of the largest longitudinal reinforcement in the boundary element unless the special boundary element terminates on a footing or mat, where special boundary element transverse reinforcement shall extend at least 12 in. into the footing or mat.

d. Horizontal shear reinforcement in the wall web shall be anchored to develop the specified yield strength, f_y, within the confined core of the boundary element.

6.8 DESIGN EXAMPLE – SHEAR WALL

This section provides a detailed design example based on strength design requirements of MSJC Code Section 3.3.6.

A shear wall computer program is suggested to estimate the location of the neutral axis and determine stresses, loads and moments. The Concrete Masonry Association of California and Nevada has a program available.

Tensile bond strength and modulus of rupture values for the unreinforced masonry is shown in Table 6.2.

EXAMPLE 6-I Shear Wall Design by Strength Methods; Vertical Load, Overturning and Shear.

A nominal 8 in. solid grouted concrete masonry shear wall carries a dead load of 4 kips/ft, live load of 1.5 kips/ft and a lateral force of 45 kips due to wind. (SDC D, but wind governs) f'_m = 1,500 psi, f_y = 60,000 psi.

Determine the required tension and shear steel using factored loads and strength design procedures developed above.

Solution 6-I

Determine factored loads.

$U = 0.9D + 1.6W + 1.6H$ (H = 0 in this case)
(IBC Section 1605.2.1, Equation 16-6)

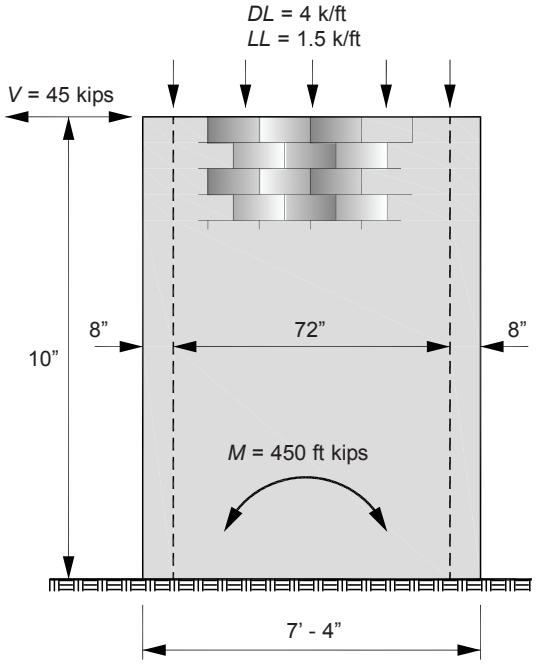


FIGURE 6.27 Load condition example 6-I.

$P_u = 0.9D = 0.9\ (4)\ 7.33$

$= 26.4$ kips

Factored overturning moment

$M = 1.6W = 1.6\ (45)\ 10$

$= 720$ ft kips

Solve for length of stress block a

Determine the constants for the coefficients for the quadratic equations

$\mathbf{a} = 0.4f'_m t$

$= 0.4\ (1.5)\ 7.63 = 4.58$

$\mathbf{b} = -0.80f'_m td$

$= -0.80\ (1.5)\ 7.63\ (80) = -732.5$

$$\mathbf{c} = P\left(\frac{l}{2} - d_1\right) + M$$

$$= 26.4\left(\frac{88}{2} - 8\right) + 720\ (12)$$

$= 9{,}590$

Solve for length of stress block a

$$a = \frac{-\mathbf{b} - \sqrt{\mathbf{b}^2 - 4\mathbf{ac}}}{2\mathbf{a}}$$

$$a = \frac{-(-732.5) - \sqrt{(-732.5)^2 - 4(4.58)9{,}590}}{2(4.58)}$$

$= 14.4$ in.

Compression forces

$C = 0.80 f'_m a t$

= 0.80(1.5) (14.4) (7.63)

= 131.7 kips

Tension force

$T = C - P$

= 131.7 - 26.4

= 105.3 kips

Area of overturning tension steel for in-plane flexure

$\phi = 0.9$

$$A_s = \frac{T}{\phi f_y} = \frac{105.3}{0.9\,(60)}$$

= 1.95 sq in.

Use 2 - #9 bars each side (A_s = 2.0 sq in.)

Check strain requirements of MSJC Code Section 3.3.3.5. Since shear walls of Seismic Design Category D must be designed as special reinforced masonry shear walls, MSJC Code Section 3.3.3.5.3 applies and the strain in the extreme fibers must be 4 times the yield strain. Based on the neutral axis, the strain in the extreme fiber can be computed using similar triangles:

$$\varepsilon_s = (d - c)\frac{\varepsilon_m}{c} = \left(80 - \frac{14.4}{0.8}\right)\frac{(0.0025)}{\left(\frac{14.4}{0.8}\right)}$$

= 0.008611

$$\frac{\varepsilon_s}{\varepsilon_y} = \frac{0.008611}{0.002069}$$

= 4.16 > 4 OK

Satisfies MSJC Code Section 3.3.3.5.3.

Shear design

$V_n = V_m + V_s$ (MSJC Code Eq 3-18)

Consider various options in the design.

Factored lateral load

V_u = 1.6 (45) = 72.0 kips

ϕ = 0.80 (MSJC Code Section 3.1.4.3)

$$V_n = \frac{V_u}{\phi} = \frac{72}{0.8} = 90 \text{ kips}$$

d_v = 7 ft 4 in. - 8 in. = 80 in.

Option 1 - Consider nominal masonry shear strength

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u$$

(MSJC Code Eq 3-21)

$$= \left[4.0 - 1.75\left(\frac{720(12)}{72(80)}\right)\right](7.63)(88)\frac{\sqrt{1500}}{1000} + 0.25(26.4)$$

= 42.4 kips

Therefore V_s required is

$V_s = V_n - V_m$ = 90 - 42.36 = 47.6 kips

$$V_s = 0.5\left(\frac{A_v}{s}\right) f_y d_v$$ (MSJC Code Eq 3-22)

$$A_v = \frac{2 s V_s}{f_y d_v}$$

$$= \frac{2s(47.6)}{60(80)} = 0.0198s \text{ in.}^2$$

(area for spacing s)

Try 16 in. spacing.

A_s = 0.0198(16) = 0.3168 in.^{2}
(#5 bars, A_s = 0.31 in.^{2}, 2% overstress)

Consider minimum steel 0.0013bt.

A_s = 0.0013 (16) (7.63)

= 0.16 in.^{2} < 0.31 in.^{2}

Use #5 at 16 in. o.c. in horizontal bond beams.

Option 2 - Assume wall is in critical hinge area, all shear to be resisted by reinforcing steel.

$$V_s = 0.5\left(\frac{A_v}{s}\right) f_y d_v$$ (MSJC Code Eq 3-22)

$$A_v = \frac{2 s V_s}{f_y d_v}$$

$$= \frac{2s(90)}{60(80)} = 0.0375s \text{ in.}^2 \quad \text{(area for spacing } s\text{)}$$

Try 16 in. spacing.

A_s = 0.0375 (16) = 0.60 in.^{2} (#7 bars @ 16 in.)

Consider steel in primary direction 0.0013*bt*.

A_s = 0.0013 (16) (7.63)

= 0.16 in.^{2} < 0.60 in.^{2}

Use #7 at 16 in. o.c. in horizontal bond beams

Vertical (transverse) shear reinforcement between OTM jamb steel:

MSJC Code Section 3.3.6.2 requires vertical reinforcement of at least half the horizontal reinforcement. Therefore, vertical reinforcement is at least 0.5(0.60) = 0.30 in.^{2}/16 in. (#5 bar A_s = 0.31).

Consider minimum steel 0.0007*bt*.

A_s = 0.0007 (16) (7.63)

= 0.85 in.^{2} < 0.31 in.^{2}

Use #5 bars @ 16 in. spacing between the #9 bars

A_s = 4(1.0) + 4(0.31) = 5.24 sq in.

However, if this same wall is located in SDC D, then MSJC Code Section 1.14.6.3 requires a minimum of 0.002 times the gross section for combined vertical and horizontal reinforcement:

A_s = 0.002*bt*

Considering horizontal and vertical ratio requirements:

(0.002 - 0.0007)*bt* = 0.0013*bt* = 0.0013 (88) (7.63) = 0.87 in.^{2}

0.0007*bt* = 0.0007 (88) (7.63) = 0.47 in.^{2} [which is less than the area of 0.85 in.^{2}]

If the shear wall is required to have plastic hinge considerations, then the masonry component should not be considered as part of the shear strength, V_m.

Additional examples are provided with respect to in-plane shear for shear walls and the use of the criteria based upon *M/Vd* as per MSJC Code Section 3.3.6.

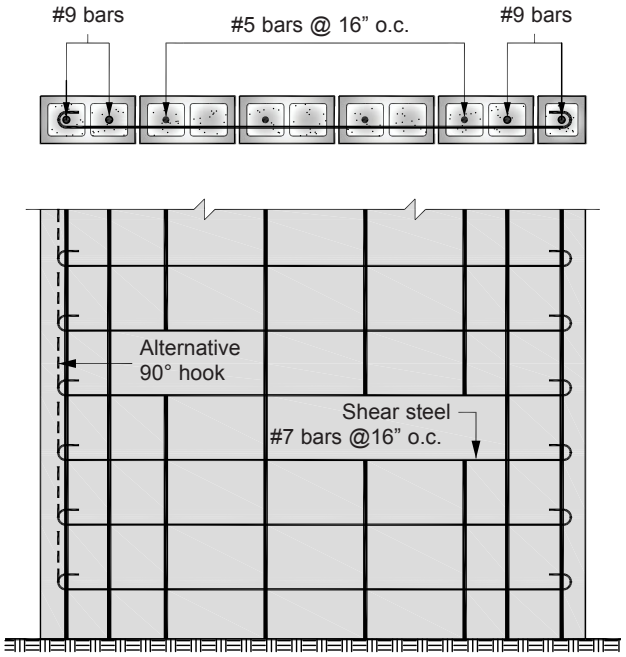


FIGURE 6.28 Layout of final design steel in shear wall (considering plastic hinge criteria).

EXAMPLE 6-J Strength Design of a Shear Wall.

Determine the reinforcing steel for the overturning moment, axial load and shear force on the solid grouted 8 in. concrete masonry wall shown. Type S portland cement-lime mortar is specified. Verify that the wall meets the requirements of MSJC Code Section 3.3.6.

V = 110 kips	(Earthquake Load)
P = 200 kips	(Dead Load)
M = 1100 ft kips	(Earthquake Load)

Wall properties

8 in. CMU = 7.625 in. actual

Given f'_m = 3,000 psi; f_y = 60,000 psi

$E_m = 900f'_m$ = 2,700,000 psi; n = 10.7

From Table SD-24:

Modulus of rupture = 163 psi

Maximum usable masonry strain, e_{mu} = 0.0025 in./in.

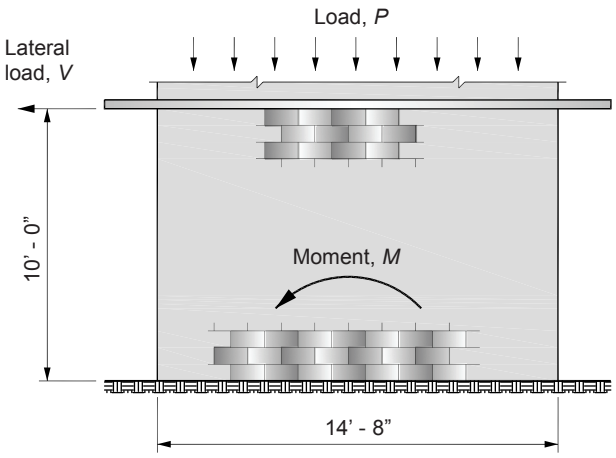


FIGURE 6.29 Masonry shear wall subjected to combined loading and moment.

Load factors (other factors for snow, rain, wind and/or contributory area could apply) from sample combinations:

$U = 1.4D$

$U = 1.2D + 1.6L$

$U = 0.9D + (1.0E \text{ or } 1.6W)$ (Assume that E controls over W for this example)

Strength reduction factors, ϕ

ϕ = 0.9 Axial load and moment (MSJC Code Section 3.1.4.1)

ϕ = 0.80 Shear (MSJC Code Section 3.1.4.3)

Estimate vertical steel requirement for overturning moment (neglecting axial force for this trial determination of an area of steel). For seismic strength design the preferred distribution of steel is uniform distribution at a 16 in. or 24 in. spacing. Thus, a distribution of reinforcement represented in Figure 6.30 is preferred.

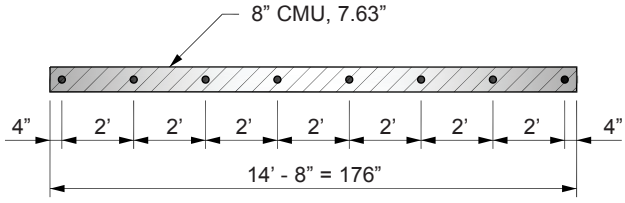


FIGURE 6.30 Shear wall reinforcement locations.

To estimate the reinforcement required compute the area of the bar size required from:

$$A_s = \frac{M}{(\sum d_i) f_y}$$

$$= \frac{1100(1.0)(12)}{(168 + 144 + 120 + 96 + 72 + 48 + 24)(60)}$$

= 0.33 in.^{2} (8 - #5 bars, A_s = 2.62 sq in.)

This value is close to the size of a #5 bar (0.31 in.^{2}), but since the combined stresses including axial loading requires the next larger size, use #6 bars (0.44 in.^{2}).

Try 8 - # 6 bars (A_s = 3.52 sq in. > 2.62 sq in.)

Analyze the shear wall by:

1. Plotting the interaction diagram for the wall.
2. Determining the cracking moment, $M_n \geq M_{cr}$.
3. Checking loading conditions for vertical load and moment.
4. Checking the requirements for boundary elements and confinement.
5. Determining the shear reinforcement.
6. Comparing the design to wall designed by the allowable stress method.

Solution 6-J

1. Plot interaction diagram

Where

P_o = Nominal axial strength

P_u = ϕ times the nominal axial strength or the factored axial load on the wall

M_n = Nominal moment strength

M_u = ϕ times the nominal moment strength or the factored moment on the wall

P_b = Balanced axial strength

P_{bu} = ϕ times the balanced axial strength

M_b = Balanced moment strength

M_{bu} = ϕ times the balanced moment strength

a) Nominal axial load P_o

$P_o = 0.80 f'_m (A_n - A_s) + f_y A_s$

A_n = 7.625 (176) = 1,342 in.^{2}

A_s = 8 (0.44) = 3.52 in.^{2}

P_o = 0.80 (3) (1,342 - 3.52) + 60 (3.52)

= 3,212 + 211 = 3,424 kips

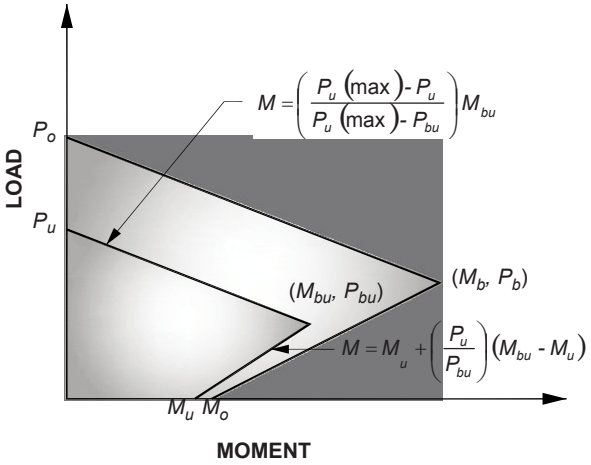


FIGURE 6.31 Simplified generic interaction diagram.

Nominal maximum axial compressive strength for the upper limit of axial force:

Where P_o is the theoretical upper limit; however, the upper permissive axial force is governed by MSJC Code Equations 3-16 or 3-17 depending upon the *h/r* ratio of greater or less than 99.

Thus, from Table GN-8b, horizontal section properties for solid grouted masonry spanning vertically, the radius of gyration, *r*, is 2.21 in., so the *h/r* ratio is determined by:

$$\frac{h}{r} = \frac{10\,(12)}{2.21} = 54.3 \le 99$$ therefore MSJC Code Equation 3-16 for $h/r \le 99$ applies

$$P_n = 0.80\left[0.80f'_m\,(A_n - A_s) + f_y A_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

(MSJC Code Eq 3-16)

P_n = 0.80[0.80(3)(1,342-3.52) + 60(3.52)]

$$\left[1 - \left(\frac{10(12)}{140(2.21)}\right)^2\right]$$

= 2,324 kips

Therefore, the P_n obtained from the above equation used with $P_u = \phi P_n$ provides the upper limit to the final axial load capacity for the interaction curves.

$P_u = \phi P_n$ = 0.90 (2,324) = 2,091 kips

b) Factored axial load, P_u

P_u = 1.4 (200) = 280 k

Check $P_u \le \phi P_n$ (conservatively check for bearing with ϕ = 0.6 in MSJC Code Section 3.1.4.5)

280 = 0.6 (2,324)

280 ≤ 1,394 kips O.K.

c) Nominal moment strength, M_n

Solve for location of the neutral axis (NA) so that sum of vertical forces equals zero.

Assume location for NA; *c* = 10 in., use trials based upon strain compatibility and equilibrium.

Use maximum allowable CMU strain = 0.0025

Solution by iteration.

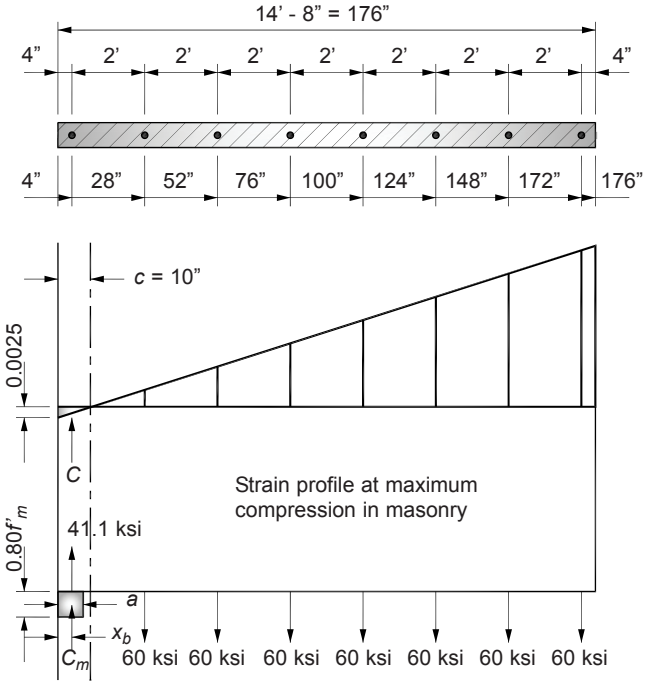


FIGURE 6.32 Steel location, strain condition and force equilibrium diagrams.

Take the sum of the moments about the extreme compression fiber at the end of wall.

a = Depth of compression stress block

= 0.80 c = 0.80 (10) = 8.0 in.

x_b = 4.0 < 8.0 in.

Tension force

$T = A_s f_y$

= 0.44 (7) (60)

= 184.8 kips

Compression force

$$f_s = E_s\left(\frac{c-4}{c}\right)\varepsilon_{mu} \text{ (by proportion)}$$

$$= 29{,}000\left(\frac{10-4}{10}\right)0.0025$$

= 43.5 ksi

$C = A_s f_s + 0.80f'_m ba$

= 0.44 (43.5 - 0.80 (3))
+ 0.80 (3) (7.625) (8.0)

= 18.1 + 146.4 = 164.5 kips

T - C = 184.8 - 164.5 = 20.3 kips (need more C force, therefore, try c = 11 in.)

a = 0.80 (11) = 8.8 in.

$$f_s = 29{,}000\left(\frac{11-4}{11}\right)0.0025 = 46.14 \text{ ksi}$$

$C = A_s f_s + 0.85f'_m ba$

= 0.44 (46.14 - 0.80(3)) + 0.80 (3) (7.625) (8.8)

= 19.2 + 161.0 = 180.2 kips (just shy by 4.6 kips, try c = 11.3 in.)

a = 0.80(11.3) = 9.04 in.

$$f_s = 29{,}000\left(\frac{11.3-4}{11.3}\right)0.0025 = 46.84 \text{ ksi}$$

$C = A_s f_s + 0.85f'_m ba$

= 0.44 (46.84 - 0.80(3))
+ 0.80 (3) (7.625) (9.04)

= 19.6 + 165.4 = 185.0 kips - reasonably close

Thus, use c = 11.3 in.

Nominal bending moment, M_n

Sum of moments about left edge of wall

M_n = T (moment arm) - C (moment arm)

= $A_s f_y$ (moment arm) - [0.80f'_m ba (moment arm) + $A_s f_s$ (moment arm)]

= 0.44[(60)(28 + 52 + 76 + 100
+ 124 + 148 + 172)]
- [0.80(3)(7.625)(9.04)(4.52)]
- 0.44[46.84 - 0.8(3)](4)

= 18,480 - 747.8 - 78.2

= 17,654 in. kips = 1,471 ft kips

d) Design bending moment, M_u

$M_u = \phi M_n$ = 0.90 (1,471)

= 1,324 ft kips

e) Nominal balanced design axial load, P_b

Compression capacity, $C_m = 0.80f'_m ba_b$

Where balanced stress block, $a_b = 0.80c$

$$c_b = \left(\frac{e_{mu}}{e_{mu} + \frac{f_y}{E_s}}\right)d$$

$$= \left(\frac{0.0025}{0.0025 + \frac{60{,}000}{29{,}000{,}000}}\right)d = 0.547d$$

c_b = 0.547 (172) = 94.1 in. (neutral axis for balanced design)

$a_b = 0.80c_b = 0.80(0.547)d$

= 0.438d

a_b = 0.438(172)

= 75.3 in., depth of compression stress block

$$x_b = \frac{176}{2} - \frac{75.3}{2} = 50.35 \text{ in.}$$

Tension force

$T = A_s f_s$

$= 0.44(4.5 + 23 + 41.5 + 60)$

$= 0.44\ (129) = 57$ kips

Compression force

$C = A_s f_s + 0.80 f'_m\, ba_b$

$= 0.44\ [(14.0 + 32.4 + 50.9 + 60) - 4\ (0.80)\ (3)]$
$+ 0.80(3)(7.625)(75.3)$

$= 65 + 1{,}378 = 1{,}443$ kips

Sum of vertical forces

$P_b = C - T$

$= 1{,}443 - 57$

$= 1{,}386$ kips

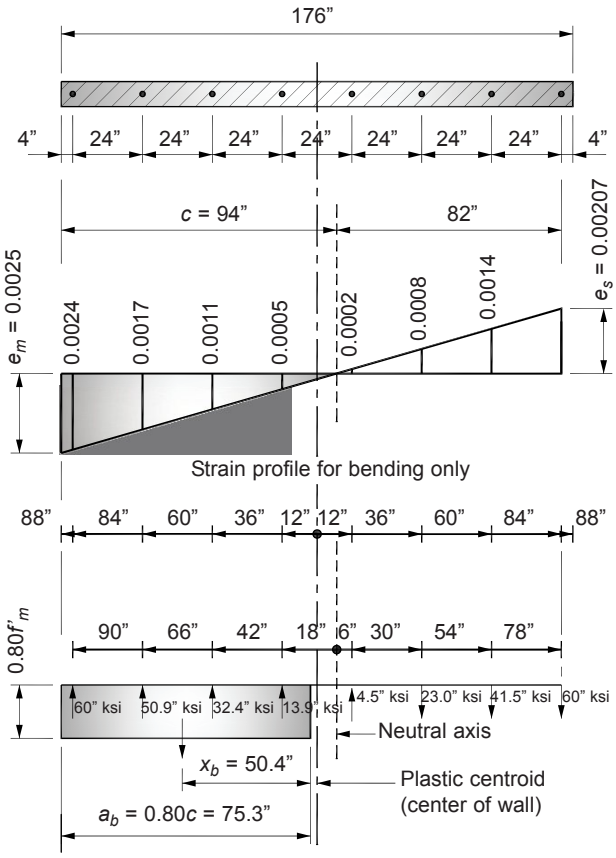


FIGURE 6.33 Balanced design load condition.

f) Design balanced axial load, P_{bu}

$P_{bu} = \phi P_b$

$= 0.9\ (1386) = 1247$ kips

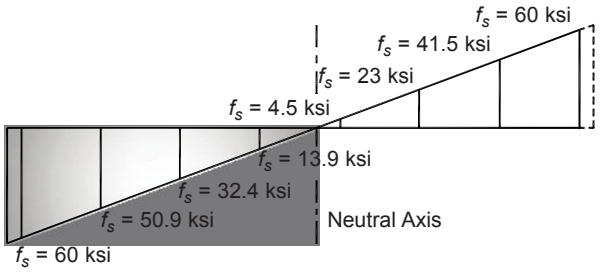


FIGURE 6.34 Stress distribution.

g) Nominal balanced design moment strength, M_b

Take moments about plastic centroid which is the center of the wall as it is symmetrical for masonry and steel

$M_b = A_s f_s$ (moment arm) $+ 0.80 f'_m a_b b x_b$

$= 0.44\ [60(84) + 41.5\ (60) + 23.0\ (36)$
$+ 4.5(12) + 13.9(12) + 32.4(36)$
$+ 50.9(60) + 60(84)]$
$+ 0.80(3)(75.3)(7.625)(50.35)$

$= 7{,}650 + 69{,}380$

$= 77{,}030$ in. kips $= 6{,}419$ ft kips

h) Design balanced moment strength, M_{bu}

$M_{bu} = \phi M_b$

$= 0.9(6{,}419) = 5{,}777$ ft kips

i) Plot the interaction diagram

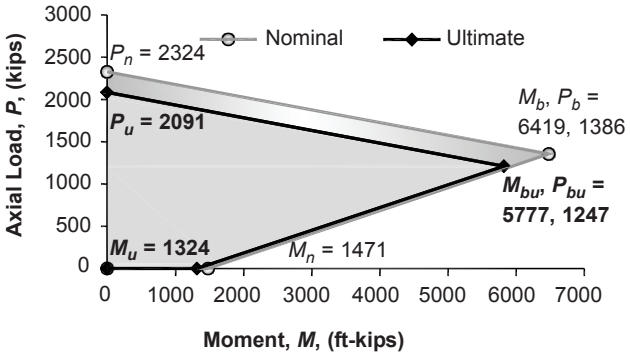


FIGURE 6.35 Interaction diagram for wall, Example 6-J for the assumed reinforcing steel.

2. Cracking moment, M_{cr}

Using gross section properties and linear elastic theory:

$$f_r = \frac{M_{cr}}{S} - \frac{P}{A}$$

Where

A = area of cross-section, bl

= 7.625 (176) = 1,342 sq in.

S = section modulus = $\frac{bl^2}{6}$

$$= \frac{7.625(176)^2}{6} = 39{,}365 \text{ in.}^3$$

from MSJC Code Table 3.1.8.2.1, f_r = 163 psi

P = dead load = 200 kips

$$M_{cr} = S\left(\frac{P}{A} + f_r\right)$$

$$= 39{,}365\left(\frac{200{,}000}{1{,}342} + 163\right)\frac{1}{1{,}000}$$

= 12,283 in. kips = 1,024 ft kips

3. Analyze two loading conditions for combined loading, vertical load and moment

a) The load condition for dead load is:

$U = 1.4D$

From Table GN-3a for a fully grouted normal weight 8 in. concrete masonry wall, the wall dead load is 84 psf. The ultimate axial load is:

$P_u = 1.4P_{DL}$

$P_{DL} = P + hl$ (wall weight per sq ft surface area)

$$P_u = 1.4\left[200 + \frac{10(14.67)(84)}{1{,}000}\right]$$

= 297.3 kips < P_{bu}

b) The load condition for dead load plus seismic load is:

$U = 0.9D + 1.0E$

$P_u = 0.9\ P_{DL}$

$$P_u = 0.9\left[200 + \frac{10(14.67)(84)}{1{,}000}\right]$$

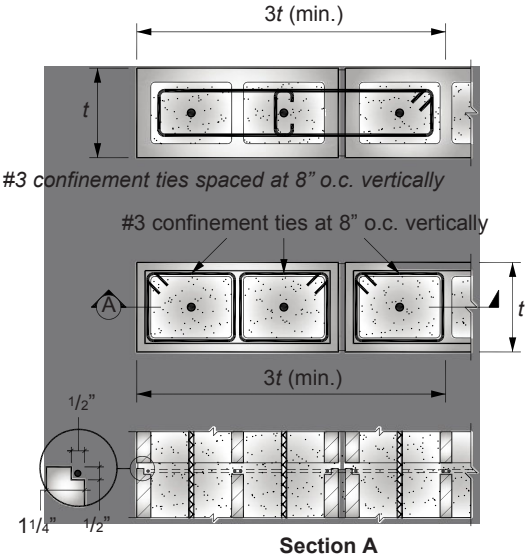
= 191.1 kips

M_u = 1.0 (1,100)

= 1,100 ft kips and the M_n is greater than the M_{cr} (Controlling load condition)

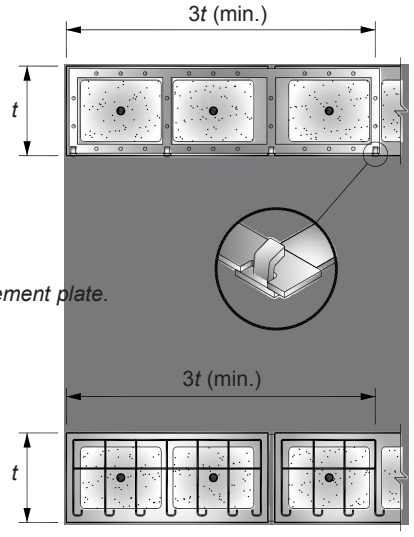
4. Check requirements for boundary and confinement condition.

For this example assume that confinement of vertical steel is not required, but the designer may specify confinement devices in boundary elements for 32 in. on each side at 8 in. vertical spacing.

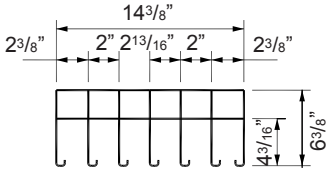


a) #3 confinement ties spaced at 8" o.c. vertically

b) #3 confinement ties spaced at 8" o.c. vertically. (Detail of confinement ties used on the 28 story Excalibur Hotel, Las Vegas, Nevada.



c) Confinement plate.



Reinforcement detail

d) Open wire mesh bed joint reinforcement.

FIGURE 6.36 Confinement devices for masonry boundary members.

5. Shear Design

a) Shear requirement from controlling load condition

$U = 0.9D + 1.0E$ $V_u = 1.0\ V_{service}$

= 1.0 (110) = 110 kips

b) Shear strength of wall is determined by:

$V_n = V_m + V_s$ (MSJC Code Eq 3-18)

Shear strength of masonry only:

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25P_u$$

(MSJC Code Eq 3-21)

where in the above equation the term $M_u/V_u d_v$ need not be taken greater than 1.0

d_v = 172 in.

M_u = 1,100 ft kips and V_u = 110 kips

$$\frac{M_u}{V_u d_v} = \frac{1{,}100(12)}{110(172)} = 0.698$$

$A_n = bl$ = 7.625(176) = 1,342 in.^{2}

From Table SD-26 and Diagram SD-26,

for $\frac{M_u}{V_u d_v} = 0.698$ and f'_m = 3,000 psi

v_m = 152 psi

$V_m = v_m A_n + 0.25P_u$

Where:

$$v_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right]\sqrt{f'_m}$$

V_m = 152(1342) + 0.25(191.1)

= 204,032 lbs

= 204 kips > 110 kips

From $C_d = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right]$

or C_d = [4 - 1.75(0.698)] = 2.78

$$V_m = \left[(C_d)(A_n)\sqrt{f'_m}\ \right] + 0.25P_u$$

$$V_m = 2.78(1{,}342)\sqrt{3{,}000} + 0.25(191{,}100)$$

= 204,342 + 47,775 = 252,117 lbs (or 252.1 kips)

ϕV_m = 0.80(2252.1) = 201.7 kips > V_u = 110 kips

Check design strength requirement of MSJC Code Section 3.1.3

The design shear strength shall exceed the shear corresponding to 125% of the nominal flexural strength, in order to provide an overstrength factor for the critical shear capacity of the wall over the flexural capacity of the wall during a seismic event.

$$\phi V_n \geq 1.25V_{Mn} = 1.25\left(\frac{M_n}{h}\right) = 1.25\left(\frac{1{,}471}{10}\right)$$

= 183.9 kips < 201.7 kips OK

Note that the V_n computed from the nominal flexural strength need not exceed 2.5 times the required shear strength, such that:

$2.5\phi V_u > \phi V_n > 1.25V_{Mn}$

Shear reinforcement is not required, except for the nominal prescriptive reinforcement required by MSJC Code Section 1.14 depending upon shear wall type.

6.9 WALL FRAMES

6.9.1 GENERAL

Masonry walls are normally considered solid elements with few openings.

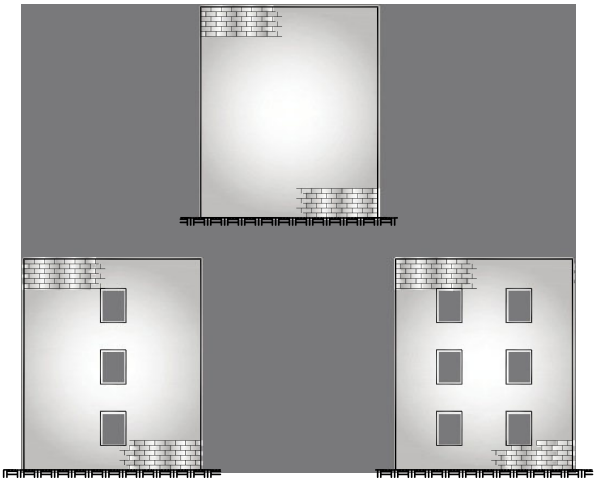


FIGURE 6.37 Shear walls with few small openings.

As openings in walls increase in size, a system of vertical load carrying elements (columns) and horizontal spandrel elements (beams) is created. As the proportions of the piers and connecting elements are changed, the system approaches the concept of a building wall frame.

Research conducted by Dr. Nigel Priestly at the University of Canterberry in Christ Church, New Zealand and at the University of California, San Diego justifies the capability of masonry wall frames. As a result of this research, requirements have been formulated and have been incorporated into some building codes, most predominately the Uniform Building Code.

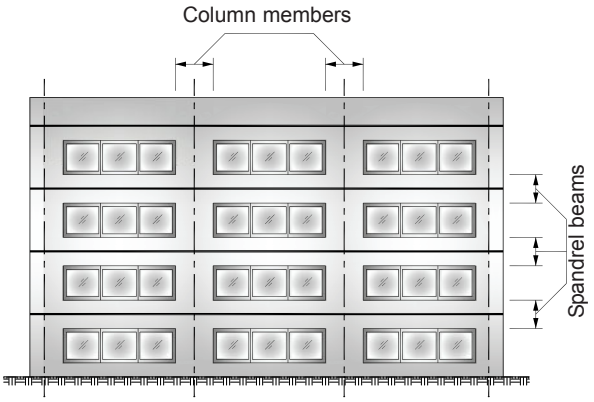


FIGURE 6.38 Elevation of a four story wall frame building.

Masonry wall frames have demonstrated ability to transmit shear and moment. They function in a ductile manner when properly proportioned and detailed. The system must be under-reinforced based on strength design requirements and the concept of a strong column and weak beam is used. This concept is to insure a ductile mechanism forming in the beam and maintaining a strong column to support vertical load.

The masonry frame must be solid grouted using open-end concrete or clay units. The masonry "wall frame" terminology was contained in the UBC. The codes cover the general beams and columns as the conventional provisions to make up the concept of a wall frame.

6.9.2 PROPORTION REQUIREMENTS

Proportional suggestions for the spandrel beam; strong column-weak beam principle are shown in Figure 6.39

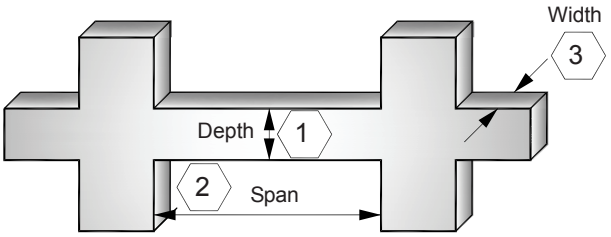


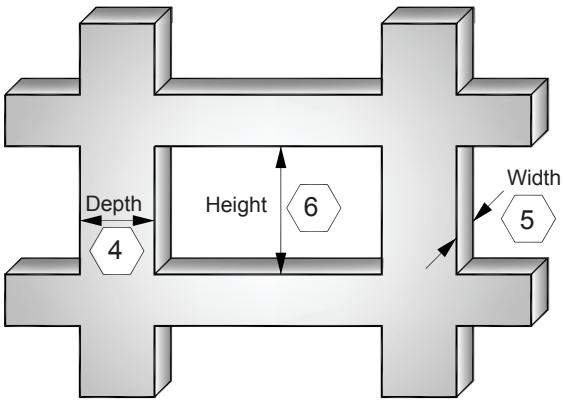
FIGURE 6.39 Horizontal spandrel beam framing member.

1. Depth of spandrel, horizontal beam between columns not less than 16 in. or two masonry units which ever is greater. The nominal depth to width ratio should be 6 or less.

2. The clear span for the beam should be two times its depth or more.

3. The nominal width should be 8 in. or $^1/_{26}$ of the clear span whichever is greater.

The pier or vertical column proportional requirements are shown in Figure 6.40.



4. The nominal depth of the column should not be more than 96 in. nor less than 32 in. or two full units, whichever is greater.

5. The nominal width of the column should not be less than the nominal width of the beam and not less than eight in. or $^1/_{14}$ of the clear height between beam faces whichever is greater.

6. The clear height to depth ratio should not exceed five.

FIGURE 6.40 Vertical column/pier framing member.

6.9.3 ANALYSIS OF MASONRY WALL FRAMES

The design and analysis of masonry wall frames uses strength design requirements and load factors to determine the cross-section size and reinforcing steel requirements. It takes into consideration the relative stiffness of columns and beams including the stiffening influence of the joints and the contribution of floor slab reinforcement, if any.

6.9.4 DESIGN STRENGTH REDUCTION FACTOR, ϕ

All members must have a strength greater than the required strength. The design strength for flexure, shear and axial load shall be the nominal strength multiplied by the strength reduction factor, ϕ.

Flexural reduction factor with or without axial load is:

ϕ = 0.90 (MSJC Code Section 3.1.4.1)

Shear reduction factor is:

ϕ = 0.80 (MSJC Code Section 3.1.4.3)

6.9.5 REINFORCEMENT DETAILS

6.9.5.1 GENERAL

a. The design shear strength, ϕV_n, must exceed the shear corresponding to the development of 1.25 times the nominal moment strength, ϕM_n of, a member, except that the nominal shear strength, V_n need not exceed 2.5 times the required shear strength, V_u.

b. Lap splices are defined in MSJC Code Section 3.3.3.4.

c. Welded splices and mechanical connections must conform to MSJC Code Sections 3.3.3.4(b) and 3.3.3.4(c).

d. Bundling of reinforcing bars is not permitted, as per MSJC Code Section 3.3.3.6.

e. MSJC Code Section 3.3.3.1 requires that reinforcing bars shall not be larger than a No. 9. The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimensions of the cell, course, or collar joint in which it is placed (Figure 6.43). The area of the reinforcing bars in a cell or course is limited to 4 percent of the cell area.

6.9.6 SPANDREL BEAMS

These suggestions apply to beams proportioned primarily to resist flexure. Factored axial compression force on the beam designed primarily to resist flexure shall not exceed 0.05 $A_n f'_m$, in accordance with MSJC Code Section 3.3.4.2.1.

6.9.6.1 LONGITUDINAL REINFORCEMENT

a. At any section of a beam, each masonry unit through the beam depth normally contains longitudinal reinforcement. Self-supporting lintel beams must contain reinforcement at the bottom one or two courses enabling the self-supporting system. Lintel beams generally contain reinforcement in either or both of the bottom two courses.

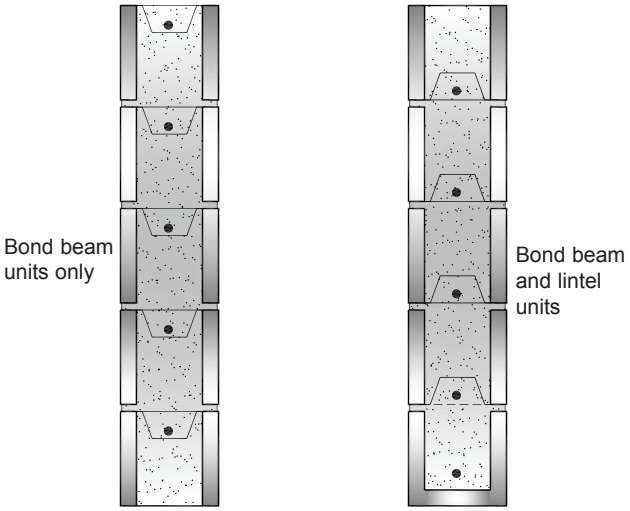


FIGURE 6.41 Uniform distribution of steel throughout the depth of the spandrel beam.

b. Minimum reinforcement ratio calculated over the gross cross section is not specified in MSJC Code, but generally, the minimum amount is at least 0.002.

c. Maximum reinforcement ratio is calculated depending upon the R greater than or less than 1.5 (MSJC Code Section 3.3.3.5.4).

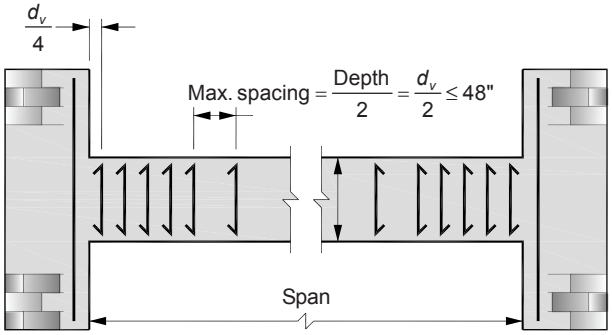


FIGURE 6.42 Maximum spacing of transverse shear reinforcement in spandrel beam.

6.9.6.2 TRANSVERSE REINFORCEMENT – BEAMS

The following are suggestions for transverse reinforcement, unless other provisions are specifically required:

a. Transverse reinforcement shall be hooked around top and bottom longitudinal bars with a standard 180-degree hook and shall be single pieces (MSJC Code Section 3.3.4.2.3(a)).

b. Within an end region extending one beam depth from pier faces and at any region at which beam plastic hinges may form during seismic or wind loading, maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the beam, d_v. The first transverse bar shall be not more than 1/4 of the beam depth, d_v, from the end of the beam. (MSJC Code Section 3.3.4.2.3(d)).

c. The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the beam and also not exceed 48 in. (MSJC Code Section 3.3.4.2.3(e)).

d. Minimum area of reinforcement shall be $0.0007bd_v$, (MSJC Code Section 3.3.4.2.3(c)).

6.9.7 PIERS SUBJECTED TO AXIAL FORCE AND FLEXURE

These requirements apply to piers proportioned to resist flexure in conjunction with axial load.

6.9.7.1 LONGITUDINAL REINFORCEMENT

a. The maximum factored axial compression force shall not exceed $0.3A_n f'_m$ (MSJC Code Section 3.3.4.3.1).

b. Longitudinal reinforcement for piers subjected to in-plane moment reversals shall be placed symmetrically about the neutral axis of the pier and comply with the following: (MSJC Code Section 3.3.4.3.2).

- Flexural reinforcement shall be essentially uniformly distributed across the member depth (MSJC Code Section 3.3.4.3.2(c)).
- The minimum area of the longitudinal reinforcement shall be $0.0007bd$ (MSJC Code Section 3.3.4.3.2(b)).
- One bar shall be placed in the end cells (MSJC Code Section 3.3.4.3.2(a)).

c. The following dimensional limits apply (MSJC Code Section 3.3.4.3.3).

- The nominal thickness of a pier shall not exceed 16 in.
- The distance between lateral supports of a pier shall not exceed 25 times the nominal thickness, except when the design is based on the provisions of MSJC Code Section 3.3.5.
- The nominal length of a pier shall not be less than three times its nominal thickness nor greater than six times its nominal thickness. The clear height shall not exceed five times its length, unless the factored axial force is less than $0.05f'_m A_g$, in which case the length may be equal to the thickness of the pier.

Other provisions for piers apply from shear, flexure and compression requirements.

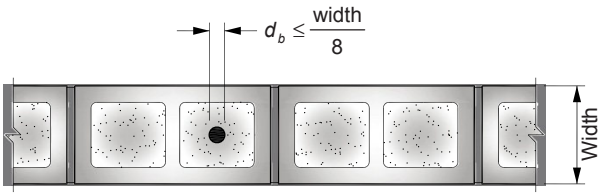


FIGURE 6.43 Masonry bar size limitation.

6.9.7.2 TRANSVERSE REINFORCEMENT

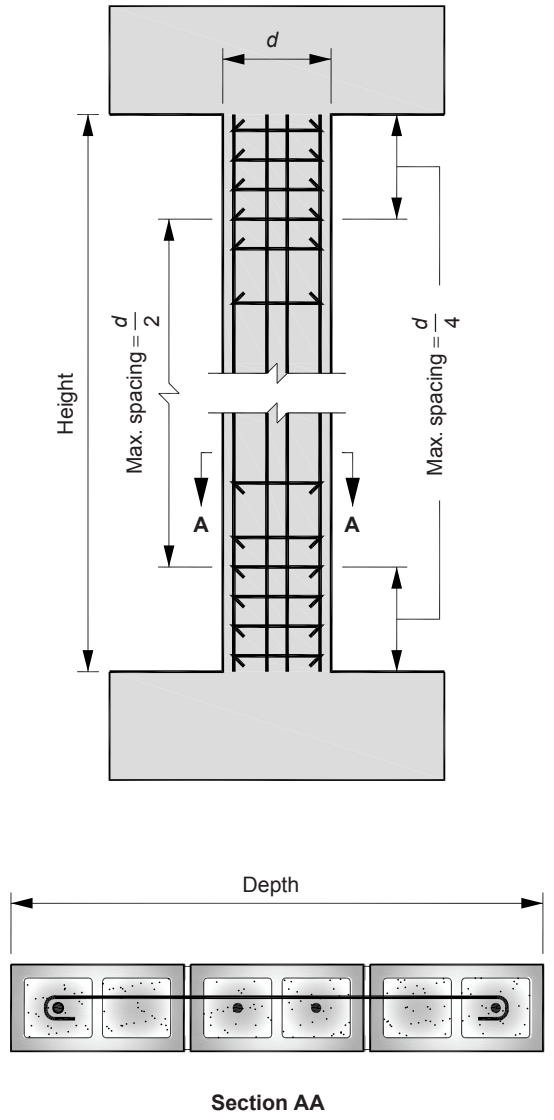


FIGURE 6.44 Spacing of transverse steel in pier.

The following provides recommendations for transverse reinforcement for piers; however, other shear, flexure and compression requirements may supercede the items below.

a. Transverse reinforcement shall be hooked around the extreme longitudinal bars with standard 180-degree hook.

b. Within an end region extending one pier depth from the end of the beam, and at any region at which plastic hinges may form during seismic or wind loading, the maximum spacing of transverse reinforcement should not exceed one fourth of the nominal depth of the pier.

c. The maximum spacing of transverse reinforcement should not exceed one-half the nominal depth of pier.

d. Minimum transverse reinforcement ratio should be 0.0015.

6.9.8 PIER DESIGN FORCES

Design of piers follows the other flexure, shear, and compression requirements in MSJC Code, except for the items previously noted. That is, for example, the shear capacity is

$$\phi V_n = \phi(V_m + V_s)$$

where these shear capacities are determined for shear in MSJC Code and likewise for flexure and compression and the special items for seismic and other provisions as required.

6.10 THE CORE METHOD OF DESIGN

6.10.1 CORE METHOD

Grouting between masonry wythes provides a vertical element, called a core, which is concrete. This concrete core can be considered the structural member which resists both vertical and lateral loads due to wind, earthquake, or more commonly, earth pressure for a retaining wall. The clay or concrete masonry serves as a form for the concrete grout and also provides the color, texture and architectural features of the wall.

There are concrete masonry face shell units specifically designed to act as forms and provide the look of masonry. Figures 6.46, 6.47 and 6.48 show how the shells are tied together with rectangular 9 gauge wire. The walls can be made to any desired width. These components are lightweight or medium weight concrete units conforming to ASTM C55 with a minimum strength of 2500 psi and may be specified for higher strengths such as 3750 psi. The components provide a 4-hour fire rating when used in 8 in. walls. Since the face shells are separate until tied in the wall, different units may be used on each side of the wall.

The system can have the units laid in mortar allowing the full width to be used in calculating masonry stresses. Both the masonry and the

concrete core can be designed based on strength design methods. When the design is based using only the concrete core, the requirements of conventional reinforced concrete apply.

The prime advantage of this method of construction and design is that high strength concrete can be utilized and/or special reinforcement such as welded wire fabric (WWF) grids.

After the units are laid, the core is filled with masonry grout or concrete. The wall thickness for concrete design purposes is measured from inside face to inside face. Ties are commonly made for walls 6 to 24 in. thick in 1 in. increments.

Component or expandable units are ideal for subterranean walls, retaining walls, and shear walls. They are also very useful when there is congestion of reinforcing steel such as at the end of shear walls.

FIGURE 6.45 Component units used where there is steel congestion.

To add texture to exposed portions of walls, split face or patterned units can be used or standard units may be sandblasted.

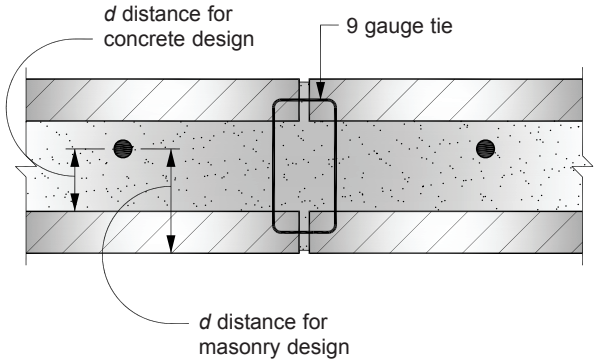


FIGURE 6.46 Component wall showing tie and *d* distance for either concrete or masonry design calculations.

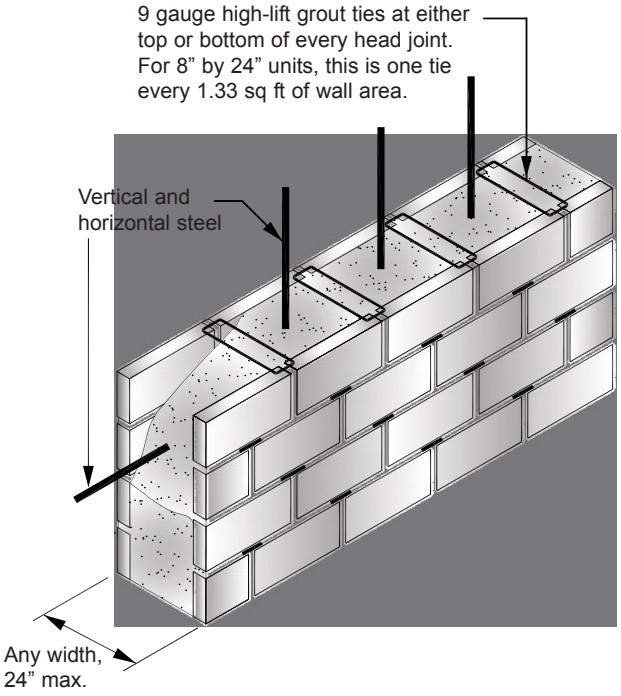


FIGURE 6.47 Wires that tie the masonry components together.

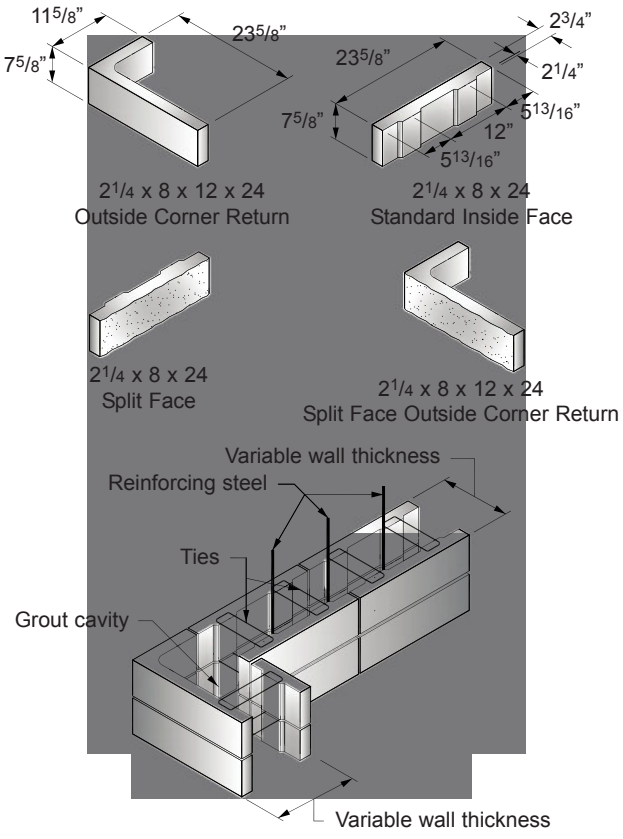


FIGURE 6.48 Typical component units.

Grout in the core space between wythes must have a minimum strength f'_g = 2,000 psi. The core may be considered as a concrete member and designed by the strength design methods of IBC Chapter 19.

The use of strength design whether for a masonry section or concrete section varies only in the coefficients. Load factors are the same for each material and the flexural strength reduction factor is 0.90 for masonry and 0.90 for concrete, for tension-controlled sections, but the provisions of IBC Chapter 19 and ACI 318 lowers the reduction factor for compression-controlled sections. Additionally, the limitation on the maximum allowable steel ratio differs between concrete and masonry.

6.10.2 COMPARISON OF THE DESIGN OF A WALL SECTION WITH COMPONENT UNITS USING MASONRY DESIGN AND CONCRETE CORE DESIGN

EXAMPLE 6-K Component Design.

Compare the cross-section requirements, *d* distance, and area of steel for a 12 ft high cantilever retaining wall using form or component units which are held in position by wire ties. Use a) allowable stress design method for masonry; b) strength design method for masonry; c) strength design method for the concrete core.

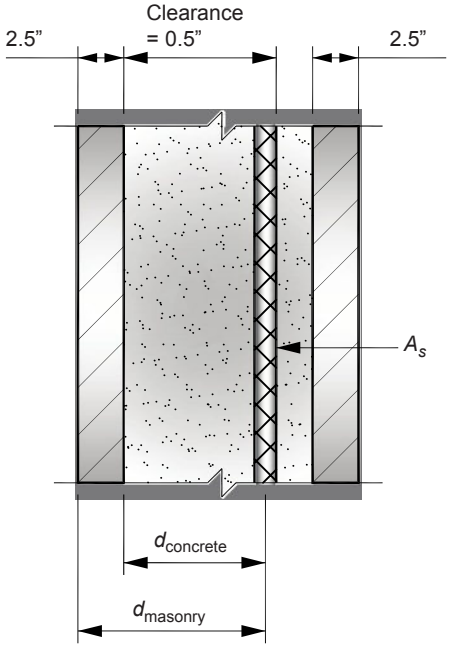


FIGURE 6.49 Masonry reinforcement clearances.

Assume f'_m = 1,500 psi, $f_g = f'_c$ = 3,000 psi and Grade 60 reinforcement.

Given:

Backfill is on a slope of 3 to 1, equivalent fluid pressure, EFP = 38 pcf.

$$\text{Moment} = \frac{1}{6}wh^3$$

$$= \frac{1}{6}(38)(12)^3$$

= 10,944 ft lbs/ft

Note that IBC Section 1806 requires a factor of safety of 1.5 against sliding and overturning stated as follows:

IBC Section 1806 Retaining Walls

1806.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Retaining walls shall be designed for a safety factor of 1.5 against lateral sliding and overturning.

This example develops the masonry size and reinforcement to resist the flexure of the retaining wall, not sliding or overturning.

6.10.2.1 MASONRY – ALLOWABLE STRESS DESIGN

Assume solid grouted

f'_m = 1,500 psi, f_s = 24,000 psi; n = 21.5

From Table ASD-24b

Balanced K_f = 69.3, ρ = 0.00322

$$bd^2 = \frac{M}{K} = \frac{10{,}944(12)}{69.3}$$

bd^2 = 1,895

b = 12 in.

d^2 = 158

d = 12.6 ≈ 13 in.

Total thickness = 13 + 0.5 (clearance)
+ 0.5 (to center of bar)
+ 2.5 (shell thickness)

= 16.5 in.

Space units for d = 13 in.

$$K = \frac{M}{bd^2} = \frac{10{,}944(12)}{12(13)^2}$$

$$= 64.8$$

From Table ASD-24b for K = 64.8

$$\rho = 0.00300$$

$$A_s = \rho bd$$

$$= 0.00300\ (12)\ (13)$$

$$= 0.47 \text{ in.}^2\text{/ft}$$

Use #9 at 24 in. o.c. (A_s = 0.50 in.^{2}/ft)

Horizontal steel = $0.0007bt$

$$= 0.0007(12)\ (16.5)$$

$$= 0.139 \text{ in.}^2\text{/ft}$$

Use #5 @ 24 in. o.c. (A_s = 0.15 in.^{2}/ft)

6.10.2.2 MASONRY – STRENGTH DESIGN

f'_m = 1500 psi, f_y = 60,000 psi

Load factor = 1.6

Strength reduction Factor ϕ = 0.9

Maximum steel ratio per MSJC Code Section 3.3.3.5

Factored moment, M_u = 1.6 (10,944)

$$= 17{,}510 \text{ ft lbs/ft}$$

The balanced ratio for strength design for concrete masonry is:

$$\rho_b = \frac{0.350f'_m}{f_y}$$

$$= 0.350\ (1500) / 60{,}000$$

$$= 0.0088 \qquad \text{(Table 6.1)}$$

There are many acceptable combinations for the selection of size and amount of reinforcement. As indicated above for the first part of this example, select b = 12 inches. That leaves A_s and d as the other two unknown variables. If the full amount for a balanced steel ratio is selected, the most economical selection is probably not going to be accomplished; however, the combinations need to be considered for efficiency of constructability and material costs.

A selection of 35 to 70 percent of the balanced steel ratio is reasonable for the first trial. Assume 50 percent, which gives 0.5(0.0088) = 0.0044.

Or, from the equations earlier in this chapter:

$$M_u \le \phi bd^2 f'_m q\ (1 - 0.62q)$$

for ρ = 0.0044,

q = 0.0044 (60,000)/1500 = 0.1760, and

$$17{,}510\ (12) \le 0.8\ (12)\ d^2$$
$$(1500)\ (0.1760)\ (1 - 0.625\ (0.1760))$$

$$d^2 = \frac{17{,}510(12)}{0.8(12)(1{,}500)(0.1760)(1 - 0.625(0.1760))} = 93.15$$

$$d = 9.7 \text{ in.}$$

Total thickness = 9.7 + 1 + 2.5 = 13.2 in.
- round to 13 in., so that d = 13 - 1 - 2.5 = 9.5 in.

Or, using tables to solve,

From Table SD-2, obtain K_u for ρ = 0.0044,

$$K_u = 187.0$$

$$bd^2 = \frac{M_u}{K_u} = \frac{17{,}510(12)}{187.0} = 1{,}124$$

Where b = 12 in.

$$d^2 = 93.64$$

d = 9.7 in. (same as above), use b = 12, t = 13, and d = 13 - 1 - 2.5 = 9.5 in.

Using actual b and d values, solve for the A_s:

$$\frac{M_u}{\phi bd^2 f'_m} = q(1 - 0.62q)$$

$$\frac{M_u}{\phi bd^2 f'_m} = \frac{17{,}510(12)}{0.9(12)(9.5)^2(1{,}500)} = 0.1437 = q(1 - 0.62q)$$

Thus, from Table SD-12, q = 0.1596

From $q = \rho f_y / f'_m$

$$\rho = 0.1596\ (1500) / 60{,}000 = 0.0040$$

$$A_s = 0.0040\ (12)\ (9.5) = 0.45 \text{ in.}^2$$

Or, from Table SD-2 for q = 0.1596, K_u = 172.8 read ρ = 0.004

$A_s = \rho bd = 0.0040(12)(9.5)$

$= 0.45$ in.^{2}/ft

Use #7 at 16 in. o.c. ($A_s = 0.45$ in.^{2}/ft)

Horizontal steel = $0.0007bt$

$= 0.0007\ (12)\ (13)$

$= 0.109$ in.^{2}/ft

Use #5 @ 32 in. o.c. ($A_s = 0.116$ in.^{2} /ft)

6.10.2.3 CONCRETE STRENGTH DESIGN

$f'_c = 3{,}000$ psi, $f_y = 60{,}000$ psi

Load factor = 1.6

ϕ factor = 0.9

Maximum ρ from a strain gradient of 0.003 in compression and not less than 0.005 in tension for a "tension-controlled" section is assumed. The balanced reinforcement ratio for concrete is based upon a strain of 0.003 in the compression side and a yield strain of the traditional amount of $\varepsilon_y = f_y/E_s$. Based upon this balanced strain gradient and performing the same derivation for the balanced ratio for concrete compression block of $0.85f'_c$ gives the following equation based upon ε_y = 60,000/29,000,000 = 0.00207 in./in.:

$$\rho_b = \frac{0.85(\beta_1)f'_c}{f_y}\left(\frac{87{,}000}{87{,}000 + 60{,}000}\right)$$

where, $\beta_1 = 0.85$ for f'_c up to 4,000 psi. Above 4,000 psi, reduce β_1 by 0.05 per 1,000 psi above the 4,000 psi, but β_1 must be greater than or equal 0.65.

For this example, this equation gives a ρ_b = 0.0214. If (like the case above), the first trial reinforcement is taken as 50 percent of the balanced condition, $\rho = 0.5(0.0214) = 0.0107$

$$K_u = \phi\rho f_y\left[1 - 0.59\left(\frac{\rho f_y}{f'_c}\right)\right]$$

(Note, the coefficient of 0.59 applies for reinforced concrete based upon the compressive strain of 0.003 and the stress block of 0.85)

$$K_u = 0.9(0.0107)(60{,}000)\left[1 - 0.59\left(\frac{0.0107(60{,}000)}{3{,}000}\right)\right]$$

$= 505$

$$bd^2 = \frac{M_u}{K_u} = \frac{17{,}510(12)}{505} = 416$$

Where:

b = 12 in.

d^2 = 34.7

d = 5.9 in.,

Total thickness = 2(2.5) + 5.9 + 0.5 + 0.5

= 11.9 in. - round to 12 in.

actual d = 12 - 2(2.5) - 0.5 - 0.5 = 6 in.

Solve for the ρ and A_s for the actual dimensions:

$$\frac{M_u}{\phi bd^2 f'_c} = \frac{17{,}510(12)}{0.9(12)(6.0)^2(3{,}000)} = 0.1801 = q(1 - 0.59q)$$

$q = 0.2049$

From $q = \rho\dfrac{f_y}{f'_m}$

$\rho = 0.2049\ (3{,}000) / 60{,}000 = 0.0102$

$A_s = \rho bd = 0.0102\ (12)\ (6.0)$

$= 0.734$ in.^{2}/ft

Use # 8 at 12 in. o.c.

Check the tension strain for the required gradient to be a tension-controlled member:

$$a = \frac{A_s f_y}{0.85f'_c\, b} = \frac{(0.734)60}{0.85(3)(12)} = 1.44 \text{ in.}$$

$c = a/\beta_1 = 1.44/0.85 = 1.69$ in.

Similar triangles shows:

$$\frac{0.003}{1.69} = \frac{0.003 + \varepsilon_s}{6.0}$$

$\varepsilon_s = 0.0076 > 0.005$ okay, therefore satisfies the tension-controlled member requirement.

Horizontal steel = $0.002bt$

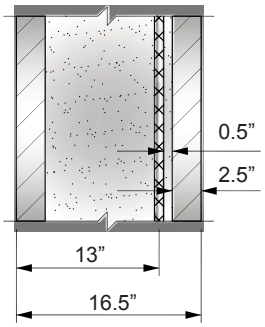
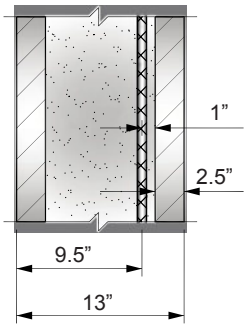
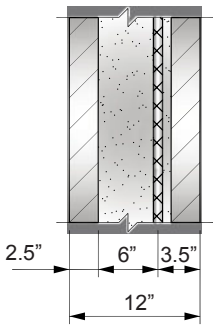
$= 0.002\ (12)\ (12)$

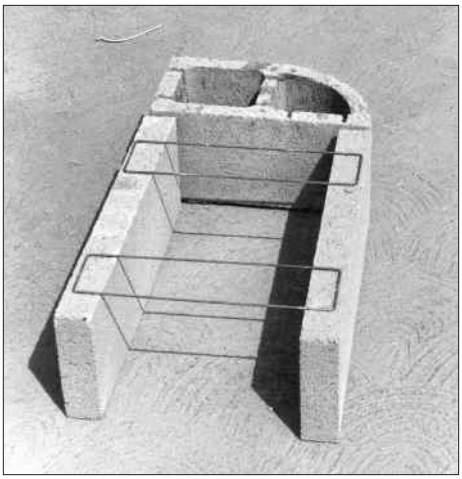
$= 0.288$ in.^{2}/ft

Use #5 @ 12 in. o.c. ($A_s = 0.31$ in.^{2}/ft), or

Use #7 @ 24 in. o.c. ($A_s = 0.30$ in.^{2}/ft)

TABLE 6.4 Summary of Comparison of Designs for Moment = 10.9 ft kips/ft

	Masonry ASD	Masonry SD	Concrete SD
			
f'_m or f_c; psi	1500	1500	3000
Depth *d*, in. **Total Thickness *t*, in.**	13.0 16.5	9.5 13.0	6.0 12.0
Vertical Reinforcement in.²/ft	#9 @ 24 in. 0.50	#7 @ 16 in. 0.45	#8 @ 12 in. 0.79
Horizontal Reinforcement in.²/ft	#5 @ 24 in. 0.13	#5 @ 32 in. 0.12	#7 @ 24 in. 0.30



Shrine Auditorium garage built with concrete component units, 6 levels, 645 car capacity – Los Angeles, CA.

FIGURE 6.50 Shrine Auditorium Garage, Los Angeles, California.

6.11 LIMIT STATE

6.11.1 GENERAL

Today the term "limit state" is used two ways. One way the term "limit state" refers to the behavior under question or to the state of the design criteria being applied. The other way the term "limit state" is used is to refer to the limit states design criteria, which in turn refers to the ultimate controlling failure of the system. The latter definition can take on many forms, including analysis, excessive deformations, unacceptable performance criteria.

Design of masonry is based on several states that limit its use or stress conditions. The qualification of these limit states may be based on the loading, the stress or the strain conditions imposed on either the reinforcing steel or masonry or on the deflection of the members.

The concept of limit state conditions were recognized by the 1963 ACI Code in a minor way and were later stated in the 1971 ACI Code as moment redistribution. The 1971 Code included the concept of changing moment pattern, stress conditions, curvature and deflection conditions.

The ultimate limit states design is predicated upon the actual material strengths, as opposed to idealized or modeled material strengths. For example, the yield strength of steel is typically taken as f_y = 60 ksi for Grade 60 reinforcement and the behavior is modeled as an idealized bilinear stress-strain curve as shown in Figure 6.52. However, the average statistical yield strength is assumed to be 72 ksi. Thus, a true limit states design is based upon the statistical variation of the actual material strength whereas factors related to the statistical variation are selected for each material to reflect the level of desired predictability of the system.

For a properly proportioned reinforced masonry structural member subjected to an ever increasing bending moment, there are three distinctive limit states that may be considered as the moment on the section changes.

The following subsections outline these basic limit states as shown in Figure 6.51.

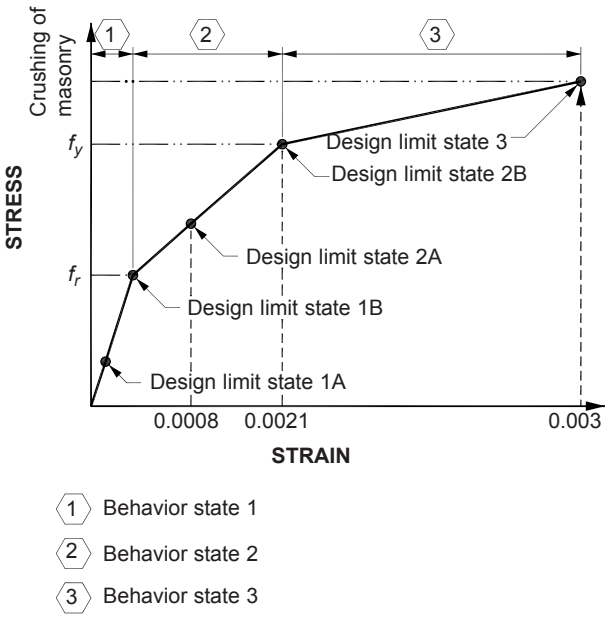


FIGURE 6.51 Limit and behavior states of a flexural member.

6.11.2 BEHAVIOR STATE 1 – UNCRACKED CONDITION

Within this behavior state, the masonry system is not cracked. The mortar joint, the bond between mortar and unit, and the masonry unit itself resist the tensile forces caused by moment on the section. The tension stresses in the masonry range from zero to less than the modulus of rupture.

The limit of behavior State 1 is reached when the moment on the section stresses the masonry in tension to the modulus of rupture.

6.11.2.1 DESIGN LIMIT STATE 1A

At the design limit State 1A, the tensile stress of the masonry is limited, based on Table SD-24 (MSJC Code Table 3.1.8.2.1) which forms the basis for the design of unreinforced masonry systems.

6.11.2.2 DESIGN LIMIT STATE 1B

At design limit State 1B, the modulus of rupture is reached and the section cracks. The modulus of rupture value has reached one of the values shown in the MSJC Code Table 3.1.8.2.1 for out-of-plane bending or in-plane bending except for grouted stack bond masonry which is based only on the continuous horizontal grout section which has reached a

maximum of 250 psi for in-plane bending, as per MSJC Code Sections 3.1.8.2.1. The cracking moment strength of the wall is determined by the equation:

$$M_{cr} = Sf_r$$

Where:

S = section modulus, in.^{3}

f_r = modulus of rupture Table SD-24 (MSJC Code Table 3.1.8.2.1 for out-of-plane and in-plane bending, or 250 psi for in-plane bending made with stack bond masonry based only on the grout section).

6.11.3 BEHAVIOR STATE 2 – CRACKED ELASTIC RANGE

When the moment on the section exceeds the modulus of rupture, the masonry will crack and behavior State 2 is reached. The reinforcing steel in the system resists the tensile forces and the masonry resists the compression forces. This is the basis for reinforced masonry.

6.11.3.1 DESIGN LIMIT STATE 2A

At design limit State 2A the stresses or strains in the steel and the masonry are limited to maximum values as given in MSJC Code Chapter 2. Initially, the values of moment occur well within the elastic range of materials. As flexure demand is increased, eventually the limit of these maximum values is reached at the stage of 2B.

6.11.3.2 DESIGN LIMIT STATE 2B

As the moment on the section increases, the stresses in the reinforcing steel and masonry increase.

To assure a ductile failure of a member, the reinforcing steel ratio is limited so that it *will* yield well before the masonry crushes. Limit State 2B occurs at the point where the steel first reaches its yield strength. For example, the steel properties for Grade 60 are shown and included in Figure 6.52.

f_y = 60,000 psi specified min.

< 78,000 psi actual max.

e_y = 0.0021 in./in. for f_y = 60 ksi

e_{sh} = 0.008 in./in. for f_y = 60 ksi

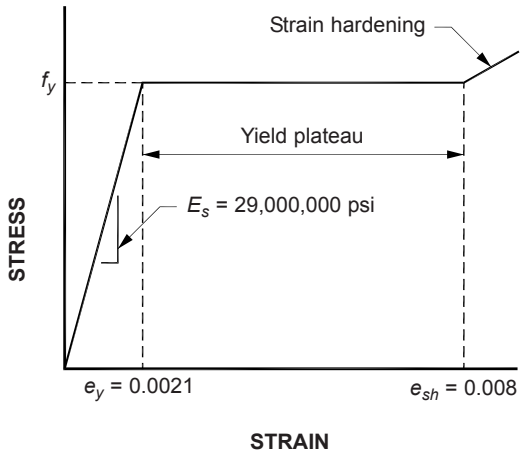


FIGURE 6.52 Stress-strain relationship for Grade 60 reinforcing steel.

6.11.4 BEHAVIOR STATE 3 – STRENGTH NONLINEAR CONDITION

After limit State 2 is achieved, the reinforcing steel stretches without significantly increasing the moment on the section. The strain in the masonry increases throughout behavior State 3 until the limiting strain in the masonry is exceeded at which point the masonry will fail in compression. The limit state for the maximum masonry compression strain ranges from 0.0025 to 0.005, as shown in Figure 6.53. Building codes, however, limit the maximum masonry compression strain to 0.0025 in./in. and 0.0035 in./in., respectively for concrete and clay masonry (MSJC Code Section 3.3.2.c).

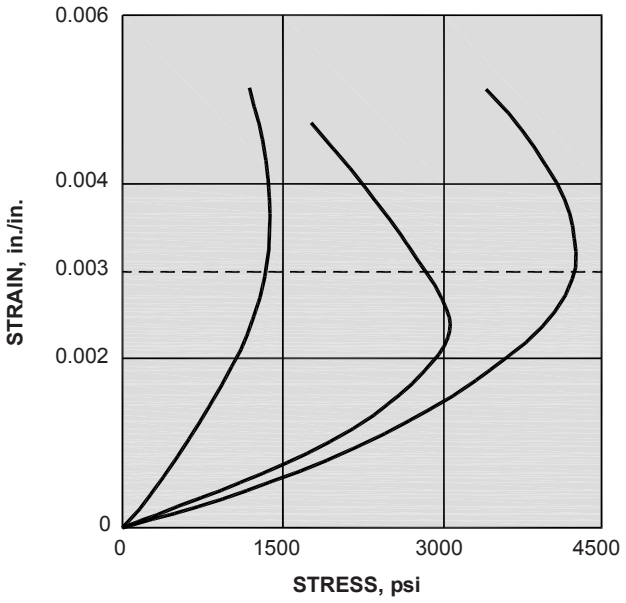


FIGURE 6.53 Stress-strain relationship for masonry.

6.11.4.1 LIMIT STATE 3

At limit State 3, the steel is at yield stress and the masonry reaches its crushing strain which is defined as 0.0025 in./in. (concrete masonry) or 0.0035 in./in. (clay masonry), and the compressive and tension force conditions along with the strain compatibility are given in MSJC Code Section 3.3.2, as shown below:

MSJC Code Section 3.3.2

3.3.2 *Design assumptions*

The following assumptions apply to the design of reinforced masonry:

a) There is strain continuity between the reinforcement, grout, and masonry such that all applicable loads are resisted in a composite manner.

b) The nominal strength of reinforced masonry cross-sections for combined flexure and axial load shall be based on applicable conditions of equilibrium.

c) The maximum usable strain, ε_{mu}, at the extreme masonry compression fiber shall be assumed to be 0.0035 for clay masonry and 0.0025 for concrete masonry.

d) Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis.

e) Compression and tension stress in reinforcement shall be taken as E_s times the steel strain, but not greater than f_y.

f) The tensile strength of masonry shall be neglected in calculating flexural strength but shall be considered in calculating deflection.

g) The relationship between masonry compressive stress and masonry strain shall be assumed to be defined by the following:

Masonry stress of 0.80 f'_m shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = 0.80\ c$ from the fiber of maximum compressive strain. The distance c from the fiber of maximum strain to the neutral axis shall be measured perpendicular to that axis.

These above conditions are the basis for strength design procedures of a member for strength design limit state.

6.11.4.2 PROPOSED MASONRY LIMIT STATES

Design Standards

MSJC Code developed a proposed Limit State Design Standard; however, the current design standards are based on limit State 3 including serviceability limits, and strength limits. The discussion above for Limit State 3 was centered around the flexural reinforced masonry requirements; however, limit states are also included for:

- shear (in-plane and out-of-plane);
- limits amount of reinforcement to provide for levels of ductility;
- bearing;
- development of reinforcement;
- splices;
- drift limits;
- deflection conditions;
- anchors;
- reinforcement limits for size and spacing;
- criteria for beams, piers, and columns;
- shear wall prescriptive reinforcement per each shear wall type;
- slender wall design; and
- transverse reinforcement criteria.

The above-listed criteria were limit states for reinforced masonry. There are also strength limit states for unreinforced or plain masonry. Both the reinforced and unreinforced strength criteria make up Chapter 3 of the MSJC Code. However, Chapter 3 is written in terms of the traditional strength design criteria as opposed to true limit states. The true limit states is based upon the expected true material strength values as opposed to specified strengths that are reduced from the true values. Thus, the design values of the masonry design in accordance with IBC or MSJC Code are predicated upon nominal strength values instead of limit states values.

6.12 QUESTIONS AND PROBLEMS

6-1 You wish to use 8 in. concrete masonry units for a 24 ft high bearing wall. Explain how you would do this in order to comply with the code.

6-2 An 8 in. thick non-load bearing concrete masonry wall is 20 ft high. Design the vertical and horizontal reinforcing steel if the wind load is 20 psf, f_y = 60,000 psi, f'_m = 2000 psi. Use strength design procedures.

6-3 A 6 in. nominal (5½ in. actual) hollow clay masonry beam has an overall depth of 36 in. The beam is continuous at the supports and has a clear span of 24 ft. f'_m = 2,500 psi, f_y = 60,000 psi, *LL* = 1000 lbs/ft, *DL* = 740 lbs/ft

plus the weight of the beam. Use strength design methods to determine tension steel and if necessary compression steel. Check if shear reinforcement is required.

6-4 What is the live load capacity for a 8 in. CMU solid grouted beam spanning 16 ft if it is 32 in. deep with d = 26 in., A_s = 2.00 sq. in.; d' = 3 in., A'_s = 0.62 sq. in., f'_m = 1500 psi; f_y = 60,000 psi; LL = 1200 plf, DL = 800 lbs/ft. Use strength design methods.

6-5 Design a 10 ft high reinforced 6 in. clay block wall for a vertical load of 4 kips/ft and a moment perpendicular to the plane of the wall of 2 ft kip/ft. Assume that the wall is fixed at the bottom and pinned at the top. Use f'_m = 3000 psi and f_y = 60,000 psi. Specify reinforcement size and spacing.

6-6 Using the slender wall method of design, check the adequacy of an 8 in. concrete masonry wall having vertical reinforcing steel of #5 @ 24 in. Assume that the wall is grouted at @ 24 in. o.c. and is located in Seismic Design Category C, wind pressure 12 psf.

The wall is 20 ft between pinned lateral supports at the floor and roof diaphragm.

The axial dead load on the wall is 450 plf, f'_m = 1500 psi, and f_y = 60 ksi.

6-7 A solid grouted reinforced clay masonry wall is 26 ft high between the lateral supports of the floor and roof diaphragm. It is located in Seismic Design Category C where the wind pressure is 20 psf. It supports a roof live load of 370 plf with an eccentricity of 7 in. to the center of the wall.

Given:

t = 10 in. d = 5 in. f_y = 60,000 psi

f'_m = 2500 psi

Determine the reinforcing steel size and spacing, and check for adequacy using the slender wall method of design. Wall is assumed pinned at the top and bottom.

6-8 Given a nominal 8 in. hollow clay masonry shear wall, solid grouted. Wall is 12 ft high; 8 ft-6 in. long, f'_m = 2500 psi; f_y = 60,000 psi; Units are 3 1/2 in. x 7 1/2 in. x 11 1/2 in.

Lateral seismic shear	V	= 60 kips
Vertical dead load	P_{DL}	= 100 kips
Vertical live load	P_{LL}	= 90 kips
Seismic Moment	M	= 720 ft kips

It is reinforced with 9 - #8 bars.

Plot the interaction diagram and determine if the wall and reinforcement is adequate for the loads and moments imposed? Try for nominal moment strength, M_n, neutral axis at 18.5 in.; for *D.I*, load condition 1, *N.A.* = 57 in. For load condition 2, *N.A.* = 37 in.

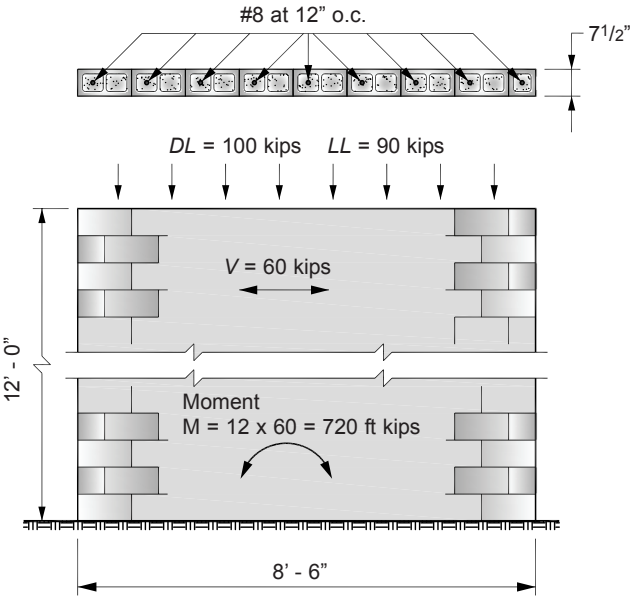


FIGURE 6.54 Problem 6-8 wall diagram.

6-9 Compare the moment capacity of a component wall system by strength design and allowable stress design methods. f'_m = 2,000 psi; f_g = 3,000 psi; f_s = 24,000 psi; f_y = 60,000 psi.

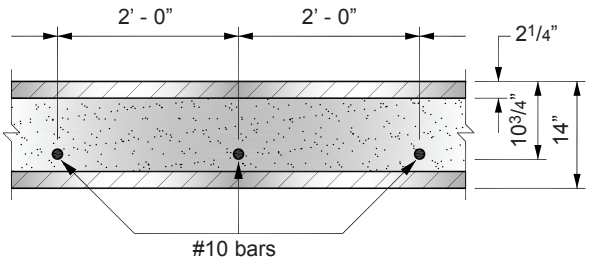


FIGURE 6.56 Problem 6-9 masonry reinforcement layout.

6-10 Using the cross-section and material properties of Problem 6-9 compare the moment capacity for d = 10.75 in. using masonry allowable stress design to d = 8.75 in. using concrete strength design, ϕ = 0.9.

CHAPTER 7

DETAILS OF REINFORCING STEEL AND CONSTRUCTION

7.1 MINIMUM REINFORCING STEEL

As part of the design process, the Structural Engineer must be aware of the minimum prescriptive reinforcement requirements and how the different elements can fit inside of a masonry wall. The convenience of hiding conduits and pipes inside a wall often competes with the structural elements of reinforcing steel and grout. While these components may theoretically fit inside the wall, unless grout adequately surrounds the reinforcing steel, the masonry will not perform as designed. This chapter provides guidance on detailing of reinforcing steel that not only complies with code requirements but also is constructable.

Prescriptive requirements for the minimum area of steel to be used in masonry depends on the seismic design category under which the structure is to be constructed. The categories are designated as Seismic Design Categories A, B, C, D, E and F. These categories are defined in ASCE 7, as adopted by the IBC and the MSJC Code provisions. Reinforcement must be placed in grout as stated in MSJC Code Section 1.13.1, with the cell dimensions and grout pour heights conforming to MSJC Code Section 1.16. For reinforcement, MSJC Code Section 1.13.2.1 limits the maximum bar size to a number 11 with the diameter limited to one-half the least cell dimension, collar joint, or bond beam in which the reinforcement is placed. For joint reinforcement, the longitudinal and cross wires must have minimum wire size of W1.1 (11 gage) and the wire must not be more than one-half the mortar joint thickness.

A more precise determination of the minimum area of steel should be based upon the section of masonry between bars of main longitudinal reinforcement to ensure that the quantity of reinforcement is sufficient to carry the flexure of the section between the main reinforcing bars. Thus, the maximum distance between bars could be based upon the modulus of rupture of the section in flexure between the bars. Or, the minimum reinforcement would be that amount needed to carry the moment on the section between the bars of the main longitudinal reinforcement. This calculation could be determined for each case, if needed.

Minimum steel area requirements are somewhat arbitrary and are an outgrowth of the minimum requirements initially used for reinforced concrete. Concrete requires a fairly large amount of minimum steel because it is cast in a plastic state and is subject to significant shrinkage during hydration. Masonry units, on the other hand, are for the most part, dimensionally stable when the wall is constructed. Only plastic mortar and grout are added to the masonry structure. Because there is far less material to shrink in a masonry wall than in a concrete wall, the minimum steel requirements have been set at half that of required concrete.

Minimum requirements for reinforced masonry shear walls are dependent upon both the Seismic Design Category of the structure and how the wall is classified for the purpose of seismic design. Reinforced masonry wall types are:

- Ordinary reinforced masonry shear walls,
- Intermediate reinforced masonry shear walls, and
- Special reinforced masonry shear walls.

TABLE 7.1 MSJC Code Minimum Seismic Reinforcement Requirements Summary

Shear Wall Type	Permitted Seismic Design Category	Minimum Reinforcement (MSJC Code Reference)		Other (MSJC Code Reference)
		Horizontal	Vertical	
Ordinary	A, B	#4 @ 120" (1.14.2.2.2.1)	#4 @ 120" (1.14.2.2.2.1)	If reinforcing required to resist shear loads, max spacing is reduced to horizontal @ 48", vertical @ 96" (2.3.5.3.1 & 2.3.5.3.2)
	C	#4 @ 48" (1.14.5.2.3)	#4 @ 120" (1.14.5.2.3)	If reinforcing required to resist shear loads, maximum vertical spacing is reduced to 96" (2.3.5.3.2)
Intermediate	A, B	#4 @ 120" (1.14.2.2.2.1)	#4 @ 48" (1.14.2.2.4)	If reinforcing requried to resist shear loads, maximum horizontal spacing is reduced to 48" (2.3.5.3.1)
	C	#4 @ 48" (1.14.5.2.3)	#4 @ 48" (1.14.2.2.4)	
Special	A, B, C	#4 @ 48" (1.14.2.2.5)	#4 @ 48" (1.14.2.2.5)	
	D	#4 @ 48" (1.14.2.2.5)	#4 @ 48" (1.14.2.2.5)	If stack bond, maximum spacings are reduced to 24" (1.14.6.3)
	E, F	#4 @ 48" (1.14.2.2.5)	#4 @ 48" (1.14.2.2.5)	If stack bond, maximum spacings are reduced to 16" (1.14.7.3)

Coordinating the requirements of shear wall types, reinforcement requirements and seismic design categories provide reinforcement requirements. These requirements must be coupled with the strength requirements for the component structure to resist imposed loads and the capacity requirements calculated by design.

MSJC Code Section 1.14.2.2 provides for prescriptive minimum reinforcement for each of the above shear wall types and connections. For the Ordinary Plain and Detailed Plain Shear Walls, the following applies:

MSJC Code Section 1.14.2.2

1.14.2.2.1 *Ordinary plain (unreinforced) masonry shear walls* — Design of ordinary plain (unreinforced) masonry shear walls shall comply with the requirements of Section 2.2, Section 3.2, or Chapter 4.

1.14.2.2.2 *Detailed plain (unreinforced) masonry shear walls* — Design of detailed plain (unreinforced) masonry shear walls shall comply with the requirements of Section 2.2 or Section 3.2, and shall comply with the requirements of Sections 1.14.2.2.2.1 and 1.14.2.2.2.2.

1.14.2.2.2.1 *Minimum reinforcement requirements* — Vertical reinforcement of at least 0.2 in.^{2} (129 mm^2) in cross-sectional area shall be provided at corners, within 16 in. (406 mm) of each side of openings, within 8 in. (203 mm) of each side of movement joints, within 8 in. (203 mm) of the ends of walls, and at a maximum spacing of 120 in. (3048 mm) on center.

Reinforcement adjacent to openings need not be provided for openings smaller than 16 in. (406 mm) in either the horizontal or vertical direction, unless the spacing of distributed reinforcement is interrupted by such openings.

Horizontal joint reinforcement shall consist of at least two wires of W1.7 (MW11) spaced not more than 16 in. (406 mm) on center, or bond beam reinforcement shall be provided of at least 0.2 in.^{2} (129 mm^2) in cross-sectional area spaced not more than 120 in. (3048 mm) on center. Horizontal reinforcement shall also be provided at the bottom and top of wall openings and shall extend not less than 24 in. (610 mm) nor less than 40 bar diameters past the opening, continuously at structurally connected roof and floor levels, and within 16 in. (406 mm) of the top of walls.

1.14.2.2.2.2 *Connections* — Connectors shall be provided to transfer forces between masonry walls and horizontal elements in accordance with the requirements of Section 2.1.8. Connectors shall be designed to transfer horizontal design forces acting either perpendicular or parallel to the wall, but not less than 200 lb per lineal ft (2919 N per lineal m) of wall. The maximum spacing between connectors shall be 4 ft (1.22 m).

For Ordinary Reinforced Shear Walls, the following applies.

MSJC Code Section 1.14.2.2.3

1.14.2.2.3 *Ordinary reinforced masonry shear walls* — Design of ordinary reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3, and shall comply with the requirements of Sections 1.14.2.2.2.1 and 1.14.2.2.2.2.

For Intermediate Reinforced Shear Walls:

MSJC Code Section 1.14.2.2.4

1.14.2.2.4 *Intermediate reinforced masonry shear walls* — Design of intermediate reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3. Design shall also comply with the requirements of Sections 1.14.2.2.2.1 and 1.14.2.2.2.2, except that the spacing of vertical reinforcement shall not exceed 48 in. (1219 mm).

For Special Reinforced Shear Walls:

MSJC Code Section 1.14.2.2.5

1.14.2.2.5 *Special reinforced masonry shear walls* — Design of special reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3. Design shall also comply with the requirements of Sections 1.14.2.2.2.1, 1.14.2.2.2.2, 1.14.6.3, and the following:

(a) The maximum spacing of vertical and horizontal reinforcement shall be the smaller of one- third the length of the shear wall, one-third the height of the shear wall, or 48 in. (1219 mm).
(b) The minimum cross-sectional area of vertical reinforcement shall be one-third of the required shear reinforcement.
(c) Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.

Reinforcement details are also prescribed for Seismic Design Category A, B, C, D, and E.

7.1.1 SEISMIC DESIGN CATEGORY A

The MSJC Code contains seismic requirements for masonry shear walls based on wall type and other items, such as lateral connections between floors and walls. SDC A, however, imposes no additional reinforcement detailing requirements. Provisions for Seismic Design Category A are:

MSJC Code Section 1.14.3.1

1.14.3.1 Structures in Seismic Design Category A shall comply with the requirements of Chapter 2, 3, 4, or 5. AAC masonry structures in Seismic Design Category A shall comply with the requirements of Appendix A.

1.14.3.2 *Drift limits* — The calculated story drift of masonry structures due to the combination of design seismic forces and gravity loads shall not exceed 0.007 times the story height.

1.14.3.3 *Anchorage of masonry walls* — Masonry walls shall be anchored to the roof and all floors that provide lateral support for the walls. The anchorage shall provide a direct connection between the walls and the floor or roof construction. The connections shall be capable of resisting the greater of a seismic lateral force induced by the wall or 1000 times the effective peak velocity-related acceleration, lb per lineal ft of wall (14,590 times, N/m).

Exception: AAC masonry walls shall comply with the requirements of Section 1.14.4.3.

7.1.2 SEISMIC DESIGN CATEGORY B

In Seismic Design Category B, there are no additional reinforcement detailing requirements.

MSJC Code Section 1.14.4.1

1.14.4.1 Structures in Seismic Design Category B shall comply with the requirements of Seismic Design Category A and with the additional requirements of Section 1.14.4. AAC masonry structures shall comply with the requirements of 1.14.4.3.

1.14.4.2 *Design of elements that are part of the lateral force-resisting system* — The lateral force-resisting system shall be designed to comply with the requirements of Chapter 2, 3, or 4. Masonry shear walls shall comply with the requirements of ordinary plain (unreinforced) masonry shear walls, detailed plain (unreinforced) masonry shear walls, ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, or special reinforced masonry shear walls.

1.14.4.3 *Anchorage of floor and roof diaphragms in AAC masonry structures* — Floor and roof diaphragms in AAC masonry structures shall be surrounded by a continuous grouted bond beam reinforced with at least two longitudinal reinforcing bars, having a total cross-sectional area of at least 0.4 $in.^2$ (260 mm^2).

7.1.3 SEISMIC DESIGN CATEGORY C

In Seismic Design Category C masonry structures must be reinforced in accordance with the requirements of the application, part or not part of the lateral force-resisting system.

MSJC Code Section 1.14.5

1.14.5.1 Structures in Seismic Design Category C shall comply with the requirements of Seismic Design Category B and with the additional requirements of Section 1.14.5.

1.14.5.2 *Design of elements that are not part of the lateral force-resisting system*

1.14.5.2.1 Load-bearing frames or columns that are not part of the lateral force-resisting system shall be analyzed as to their effect on the response of the

system. Such frames or columns shall be adequate for vertical load carrying capacity and induced moment due to the design story drift.

1.14.5.2.2 Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.

1.14.5.2.3 *Reinforcement requirements* — Masonry elements listed in Section 1.14.5.2.2, except AAC masonry elements, shall be reinforced in either the horizontal or vertical direction in accordance with the following:

(a) *Horizontal reinforcement* — Horizontal joint reinforcement shall consist of at least two longitudinal W1.7 (MW11) wires spaced not more than 16 in. (406 mm) for walls greater than 4 in. (102 mm) in width and at least one longitudinal W1.7 (MW11) wire spaced not more 16 in. (406 mm) for walls not exceeding 4 in. (102 mm) in width; or at least one No. 4 (M #13) bar spaced not more than 48 in. (1219 mm). Where two longitudinal wires of joint reinforcement are used, the space between these wires shall be the widest that the mortar joint will accommodate. Horizontal reinforcement shall be provided within 16 in. (406 mm) of the top and bottom of these masonry walls.

(b) *Vertical reinforcement* — Vertical reinforcement shall consist of at least one No. 4 (M #13) bar spaced not more than 120 in. (3048 mm) for Seismic Design Category C and not more than 48 in. (1219 mm) for Seismic Design Category D, E, and F. Vertical reinforcement shall be located within 16 in. (406 mm) of the ends of masonry walls.

1.14.5.3 *Design of elements that are part of the lateral force-resisting system* — Design of masonry columns and shear walls shall comply with the requirements of 1.14.5.3.1 and 1.14.5.3.2. Design of ordinary reinforced AAC masonry structures shall comply with the requirements of 1.14.5.3.3.

1.14.5.3.1 *Connections to masonry columns* — Connectors shall be provided to transfer forces between masonry columns and horizontal elements in accordance with the requirements of Section 2.1.8. Where anchor bolts are used to connect horizontal elements to the tops of columns, anchor bolts shall be placed within lateral ties. Lateral ties shall enclose both the vertical bars in the column and the anchor bolts. There shall be a minimum of two No. 4 (M #13) lateral ties provided in the top 5 in. (127 mm) of the column.

1.14.5.3.2 *Masonry shear walls* — Masonry shear walls shall comply with the requirements for ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, or special reinforced masonry shear walls.

1.14.5.3.3 *Anchorage of floor and roof diaphragms in AAC masonry structures* — Lateral load between floor and roof diaphragms and AAC masonry shear walls shall be transferred through connectors embedded in grout in accordance with Section 2.1.8. Connectors shall be designed to transfer horizontal design forces acting either parallel or perpendicular to the wall but not less than 200 lb per lineal ft (2919 N per lineal m) of wall. The maximum spacing between connectors shall be 4 ft (1.2 m).

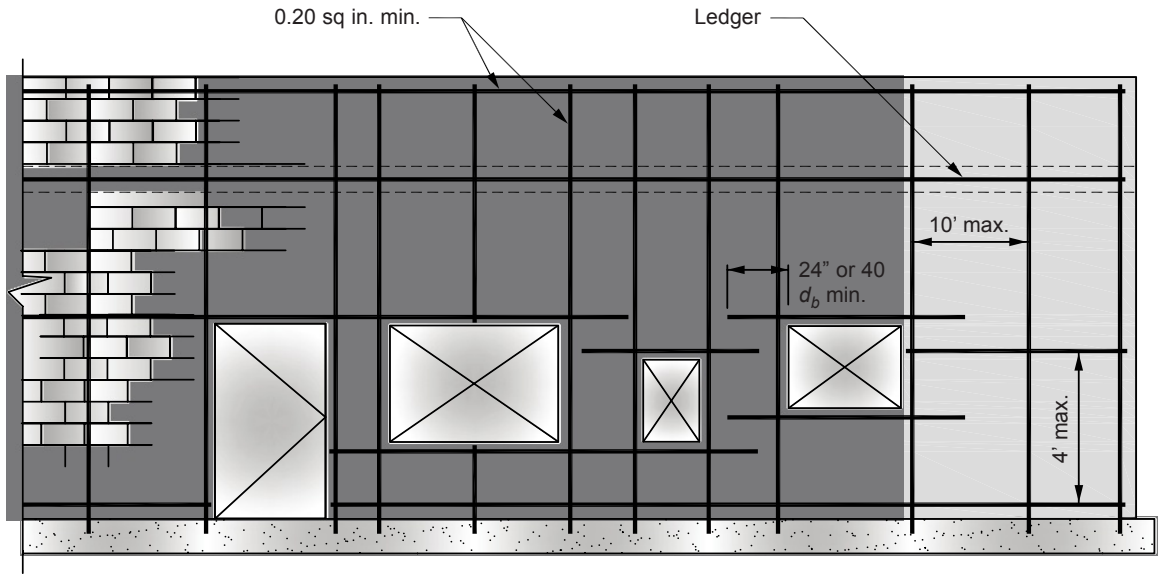


FIGURE 7.1 Minimum deformed reinforcement for Seismic Design Category C elements that are not part of the lateral force-resisting system.

Continuous reinforcement at the top and bottom of openings may be used in determining the maximum spacing specified in the above requirements. Figure 7.1 provides the layout of the wall reinforcement as indicated in the requirements for elements that are not part of the lateral force-resisting system in SDC C.

7.1.4 SEISMIC DESIGN CATEGORY D

The MSJC Code provisions for Category D are :

MSJC Code Section 1.14.6.1

1.14.6.1 Structures in Seismic Design Category D shall comply with the requirements of Seismic Design Category C and with the additional requirements of Section 1.14.6.

Exception: AAC masonry elements shall comply with the requirements of 1.14.5.

1.14.6.2 *Design requirements* — Masonry elements, other than those covered by Section 1.14.5.2.2, shall be designed in accordance with the requirements of Sections 2.1 and 2.3, Chapter 3, Chapter 4 or Appendix A.

1.14.6.3 *Minimum reinforcement requirements for masonry walls* — Masonry walls other than those covered by Section 1.14.5.2.2, and other than AAC masonry, shall be reinforced in both the vertical and horizontal direction. The sum of the cross-sectional area of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall, and the minimum cross-sectional area in each direction shall be not less than 0.0007 times the gross cross-sectional area of the wall, using specified dimensions. Reinforcement shall be uniformly distributed. The maximum spacing of reinforcement shall be 48 in. (1219 mm), except for stack bond masonry. Wythes of stack bond masonry shall be constructed of fully grouted hollow open-end units, fully grouted hollow units laid with full head joints, or solid units. Maximum spacing of reinforcement for walls with stack bond masonry shall be 24 in. (610 mm).

1.14.6.4 *Masonry shear walls* — Masonry shear walls shall comply with the requirements for special reinforced masonry shear walls.

1.14.6.5 *Minimum reinforcement for masonry columns* — Lateral ties in masonry columns shall be spaced not more than 8 in. (203 mm) on center and shall be at least 3/8 in. (9.5 mm) diameter. Lateral ties shall be embedded in grout.

1.14.6.6 *Material requirements* — Neither Type N mortar nor masonry cement shall be used as part of the lateral force-resisting system.

1.14.6.7 *Lateral tie anchorage* — Standard hooks for lateral tie anchorage shall be either a 135-degree standard hook or a 180-degree standard hook.

See Figure 7.3 for the minimum prescriptive reinforcement requirements for SDC D.

FIGURE 7.2 Reinforcement in a concrete masonry wall located in Seismic Design Category D.

7.1.5 SEISMIC DESIGN CATEGORIES E AND F

Below are the requirements for Seismic Design Categories E and F. See Figure 7.3 for the minimum prescriptive reinforcement for walls for SDC E and F.

MSJC Code Section 1.14.7.1

1.14.7.1 Structures in Seismic Design Categories E and F shall comply with the requirements of Seismic Design Category D and with the additional requirements of Section 1.14.7.

1.14.7.2 *Minimum reinforcement for stack bond elements that are not part of the lateral force-resisting system* — Stack bond masonry that is not part of the lateral force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0015 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 24 in. (610 mm). These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

1.14.7.3 *Minimum reinforcement for stack bond elements that are part of the lateral force-resisting system* — Stack bond masonry that is part of the lateral force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0025 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 16 in. (406 mm). These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

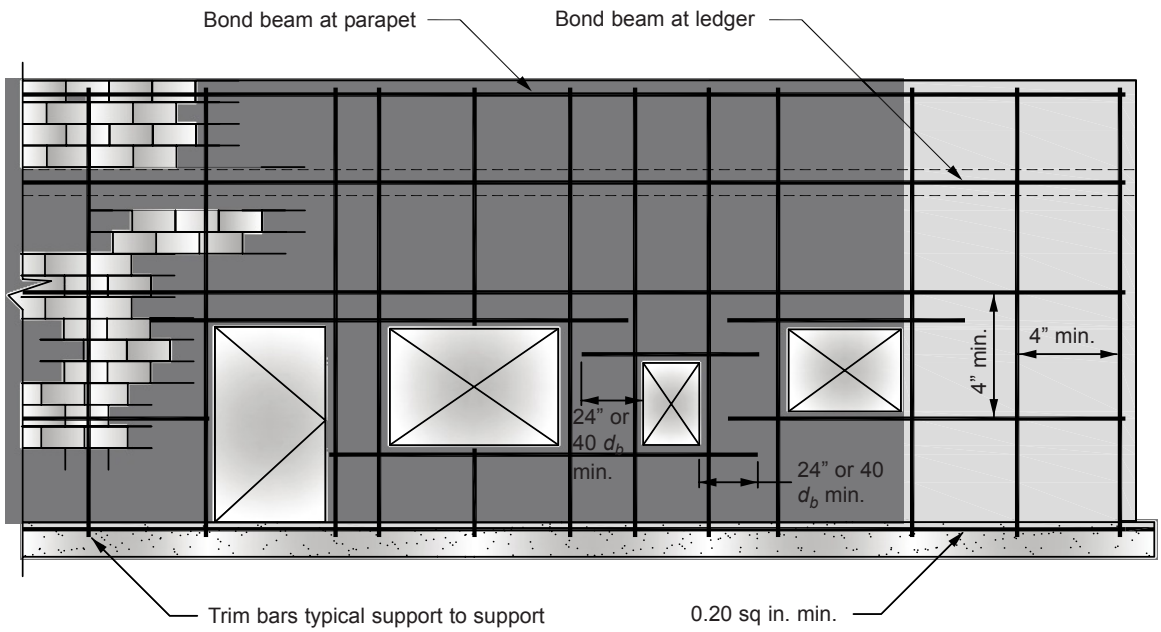


FIGURE 7.3 Minimum wall reinforcement for Seismic Design Category D, E, and F.

7.1.6 CALCULATION OF MINIMUM STEEL AREA

MSJC Code Section 1.14.6.3 states that a wall must be reinforced both vertically and horizontally with a required minimum amount of reinforcing. The minimum area of reinforcement for Seismic Design Categories D, E and F, in one direction, either vertically or horizontally, may not be less than 0.0007 times the gross cross-sectional area of the wall. The sum of the horizontal and vertical reinforcement must be at least 0.002 time the gross cross-sectional area.

The gross cross-sectional area is the width of the wall times a given length.

EXAMPLE 7-A Minimum Areas of Steel.

Based on the 2005 MSJC Code, determine the minimum size and spacing of reinforcing steel required in each direction for:

(a) 9 in. solid grouted double-wythe brick wall in SDC D.
(b) 8 in. concrete block wall in SDC E.

Solution 7-A

MSJC Code Section 1.14.6.3 requires at least $A_s = 0.0007bt$ in both directions with a minimum total area of steel of $0.002bt$ for all reinforced masonry structures located in Seismic Design Categories D. Generally, $0.0007bt$ is placed in the wall opposite of the direction the wall spans. The balance of the reinforcement ($0.002bt - 0.0007bt = 0.0013bt$) is placed in the direction the wall is principally spanning.

(a) 9 in. Solid Grouted Brick Wall

Total reinforcement required:

$A_s = 0.0020bt = 0.216$ sq in./ft

Area of reinforcement required in weak direction:

$A_s = 0.0007bt = 0.076$ sq in./ft

Choose #5 @ 48 in. o.c. in weak direction ($A_s = 0.075$ sq in./ft)

Area of reinforcement required in strong direction:

A_s (required total)	0.216
A_s (in weak direction)	0.076
A_s (principal direction)	0.140

Choose #5 @ 26 in. o.c. in the principal (strong) direction ($A_s = 0.139$ sq in./ft)

(b) 8 in. Solid Concrete Block Wall

Total reinforcement required:

$A_s = 0.0020bt = 0.183$ sq in./ft

Area of reinforcement required in weak direction:

A_s = 0.0007*bt* = 0.064 sq in./ft

Choose #5 @ 48 in. o.c. in weak direction (A_s = 0.075 sq in./ft)

Area of reinforcement required in strong direction:

A_s (required total)	0.183
A_s (in weak direction)	0.076
A_s (principal direction)	0.107

Choose #5 @ 32 in. o.c. in the principal (strong) direction (A_s = 0.116 sq in./ft)

EXAMPLE 7-B Minimum Steel Requirements Utilizing Joint Reinforcement.

Select the minimum vertical and horizontal reinforcement for an 8 in. block wall which spans 12 ft between the foundation and the roof bond beam. The wall is located in Seismic Design Category D.

Solution 7-B

For SDC D, use A_s = 0.0013*bt* vertically and A_s = 0.0007*bt* horizontally to satisfy the requirements of MSJC Code Section 1.14.6.3.

Therefore:

Vertical reinforcement, A_s = 0.119 sq in./ft (Table GN-21b).

Minimum horizontal A_s = 0.064 sq in./ft (Table GN-21a).

From Table GN-21b, choose vertical reinforcement of #5 @ 32 in. o.c. (A_s = 0.116 sq in./ft)

To find the additional horizontal area of steel required to meet the A_s = 0.064 sq in./ft, the contribution of the joint reinforcement, if used, must first be determined.

Total required horizontal steel, A_s = 0.064 x 12

= 0.769 sq in.

Place the joint reinforcement in every other mortar joint or at 16 in. o.c.

Therefore:

$$\text{The number of joints reinforced} = \frac{12 \text{ ft x } 12 \text{ in./ft}}{16 \text{ in.}} - 1$$

= 8 joints

From Table GN-20c, the area of 2 - #9 longitudinal joint reinforcing wires is 0.035 sq in. Therefore, the area of steel provided by the joint reinforcement is:

A_s = 0.035 x 8 joints reinforced = 0.28 in^2

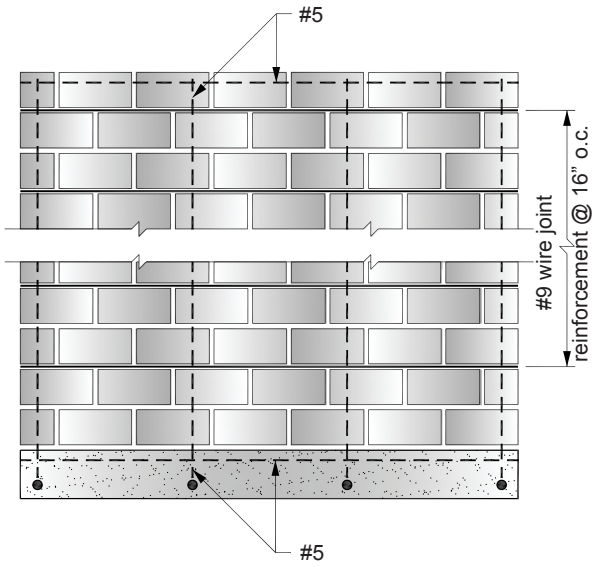


FIGURE 7.4 Wall with joint reinforcement.

Area of steel needed in the bond beam and the top of the footing is:

$$\frac{0.769 - 0.28}{2} = 0.24 \text{ in.}^2$$

Use #5 bar in the bond beam and top of the footing.

The general practice is for the principal steel which resists the design stresses in SDC D or higher, to be the larger amount of steel, (A_s = 0.0013*bt*), and perpendicular to it would be the minimum amount of steel (A_s min. = 0.0007*bt*). Thus, if a wall spans vertically, between floors, or between the floor and the roof, the principal steel would be vertical and would be 0.0013*bt* or, as required by engineering calculations. The minimum horizontal steel could then be 0.0007*bt* as required.

Many times, however, the same amount of steel is used both vertically and horizontally. In that case, the area of steel would be 0.001*bt* placed in both directions.

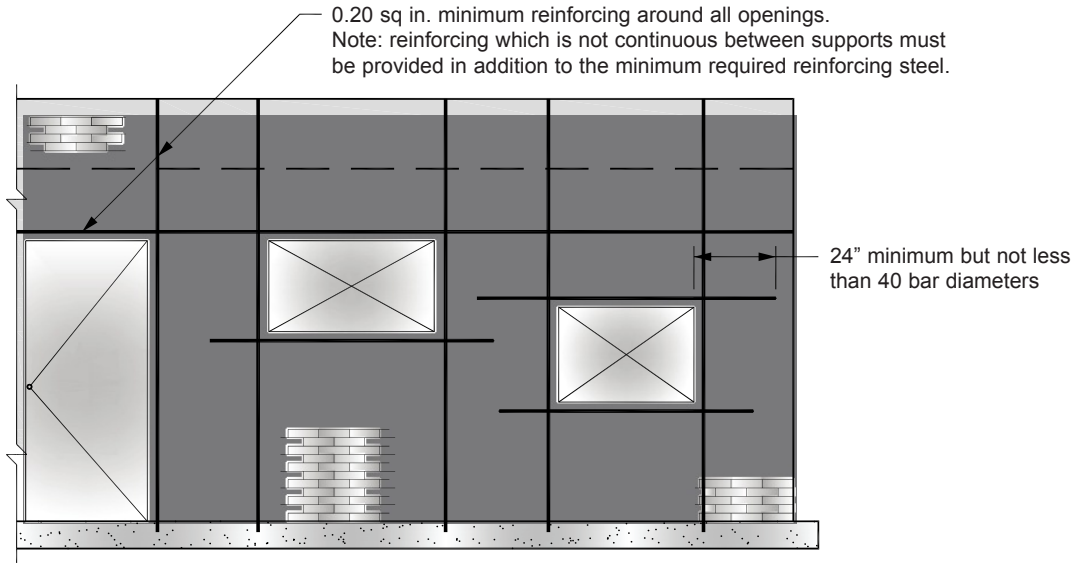


FIGURE 7.5 Typical reinforcing steel around opening (Coordinate this figure with Figure 7.1 and 7.3 for minimum wall reinforcement requirements).

7.2 REINFORCING STEEL AROUND OPENINGS

In reinforced masonry, walls containing openings may require perimeter reinforcement. For example, there should be not less than one #4 bar or two #3 bars on all sides of, and adjacent to, every opening which exceeds 16 inches in either direction. These bars should extend at least 40 diameters, but in no case less than 24 in., beyond the corner of the opening. These bars should be provided in addition to the minimum reinforcement, unless they are continuous throughout the length of the wall.

7.3 PLACEMENT OF STEEL

7.3.1 POSITIONING OF STEEL

Placement of reinforcing bars should conform to the recommended practice of placing reinforcing bars in concrete. Principal steel should be properly located and secured in position so that it will resist the forces for which it was designed. This is particularly important in elements such as cantilever retaining walls, beams and columns.

There is no code requirement for spacing of reinforcing bar supports, but as a point of reference, the Uniform Building Code required that vertical bars be held in place at top and bottom and at intervals not exceeding 200 diameters of the reinforcement to insure correct location of principal steel.

Vertical dowels out of position may be bent at a slope of 1 to 6 for proper alignment (Figure 7.6). This is based on ACI 318-05, Section 7.8.1.1. As a practical matter, bars larger than #5 should not be field-bent without the approval of the structural engineer.

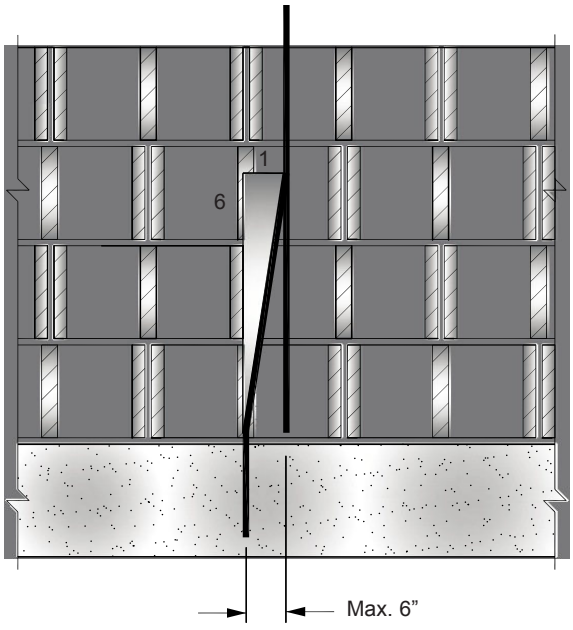


FIGURE 7.6 Slope for bending reinforcing steel into position.

7.3.2 TOLERANCES FOR PLACEMENT OF STEEL

For reinforced masonry to perform as designed, reinforcement, wall ties, and anchors must be in the proper location.

The proper placement of reinforcing steel is governed by MSJC Code Section 1.13.3 and MSJC Specification Article 3.4. Project drawings must include the locations of reinforcement, wall ties, and anchors along with the associated sizes, types detailed locations.

MSJC Specification Article 3.4 B

3.4 B. *Reinforcement*

1. Support and fasten reinforcement together to prevent displacement beyond the tolerances allowed by construction loads or by placement of grout or mortar.
2. Completely embed reinforcing bars in grout in accordance with Article 3.5.
3. Maintain clear distance between reinforcing bars and any face of masonry unit or formed surface, but not less than 1/4 in. (6.4 mm) for fine grout or 1/2 in. (12.7 mm) for coarse grout.
4. Splice only where indicated on the Project Drawings, unless otherwise acceptable. When splicing by welding, provide welds in conformance with the provisions of AWS D 1.4.
5. Unless accepted by the Architect/Engineer, do not bend reinforcement after it is embedded in grout or mortar.
6. Place joint reinforcement so that longitudinal wires are embedded in mortar with a minimum cover of 1/2 in. (12.7 mm) when not exposed to weather or earth and 5/8 in. (15.9 mm) when exposed to weather or earth.
7. *Placement tolerances*
   a. Tolerances for the placement of steel in walls and flexural elements shall be ± 1/2 in. (12.7 mm) when the distance from the centerline of steel to the opposite face of masonry, *d*, is equal to 8 in. (203 mm) or less, ± 1 in. (25.4 mm) for d equal to 24 in. (610 mm) or less but greater than 8 in. (203 mm), and ± 1 1/4 in. (31.8 mm) for *d* greater than 24 in. (610 mm).
   b. Place vertical bars within 2 in. (50.8 mm) of the required location along the length of the wall.
   c. If it is necessary to move bars more than one bar diameter or a distance exceeding the tolerance stated above to avoid interference with other reinforcing steel, conduits, or embedded items, notify the Architect/Engineer for acceptance of the resulting arrangement of bars.

The wall tie placement criteria are contained in:

MSJC Specification Article 3.4 C:

3.4 C. *Wall ties*

1. Embed the ends of wall ties in mortar joints. Embed wall tie ends at least 1/2 in. (13 mm) into the outer face shell of hollow units. Embed wire wall ties at least 1 1/2 in. (38.1 mm) into the mortar bed of solid masonry units or solid grouted hollow units.
2. Unless otherwise required, bond wythes not bonded by headers with wall ties as follows:

   The maximum spacing between ties is 36 in. (914 mm) horizontally and 24 in. (610 mm) vertically.

Wire size	*Minimum number of wall ties required*
W.17 (MW11)	One per 2.67 ft² (0.25 m²)
W2.8 (MW18)	One per 4.50 ft² (0.42 m²)

3. Unless accepted by the Architect/Engineer, do not bend wall ties after being embedded in grout or mortar.
4. Unless otherwise required, install adjustable ties in accordance with the following requirements:
   a. One tie for each 1.77 ft² (0.16 m²) of wall area.
   b. Do not exceed 16 in. (406 mm) horizontal or vertical spacing.
   c. The maximum misalignment of bed joints from one wythe to the other is 1 1/4 in. (31.8 mm).
   d. The maximum clearance between connecting parts of the ties is 1/16 in. (1.6 mm).
   e. When pintle legs are used, provide ties with at least two legs made of wire size W2.8 (MW18).
5. Install wire ties perpendicular to a vertical line on the face of the wythe from which they protrude. Where one-piece ties or joint reinforcement are used, the bed joints of adjacent wythes shall align.
6. Unless otherwise required, provide additional unit ties around openings larger than 16 in. (406 mm) in either dimension. Space ties around perimeter of opening at a maximum of 3 ft (0.91 m) on center. Place ties within 12 in. (305 mm) of opening.
7. Unless otherwise required, provide unit ties within 12 in. (305 mm) of unsupported edges at horizontal or vertical spacing given in Article 3.4 C.2.

Allowable placement tolerances for reinforcement are shown in Figure 7.7 and in Table 7.2.

TABLE 7.2 Tolerances for Placing Reinforcement

Distance, *d*, from face of CMU to the center of Reinforcing	Allowable tolerance (in.)
$d \leq 8$ in.	$\pm 1/2$
8 in. $< d \leq 24$ in.	± 1
$d > 24$ in.	$\pm 1\,1/4$

7.3.3 CLEARANCES

7.3.3.1 CLEARANCE BETWEEN REINFORCING STEEL AND MASONRY UNITS

To be effective, reinforcing steel must be surrounded by grout. Reinforcing steel bars must have a minimum of 1/4 in. of grout between the steel and the masonry when fine (sand) grout is used. When coarse (pea gravel) grout is used, the clearance between the steel and the masonry units must be at least 1/2 in. This assures proper bond so that stresses may be transferred between the steel and the masonry as shown in Figure 7.8. The above clearances are not subject to placement tolerances, that is, after the reinforcing steel is placed, clearance must be present so that grout can completely surround the reinforcement.

7.3.3.2 CLEAR SPACING BETWEEN REINFORCING BARS

The clear distance between parallel bars, except in columns, must be at least the nominal diameter of the bars or 1 in., except that bars in a splice may be in contact. This clear distance requirement applies to the clear distance between a contact splice and adjacent splices or bars. In columns and pilasters, the clear spacing between bars must be 1 1/2 bar diameters, but not less than 1 inch.

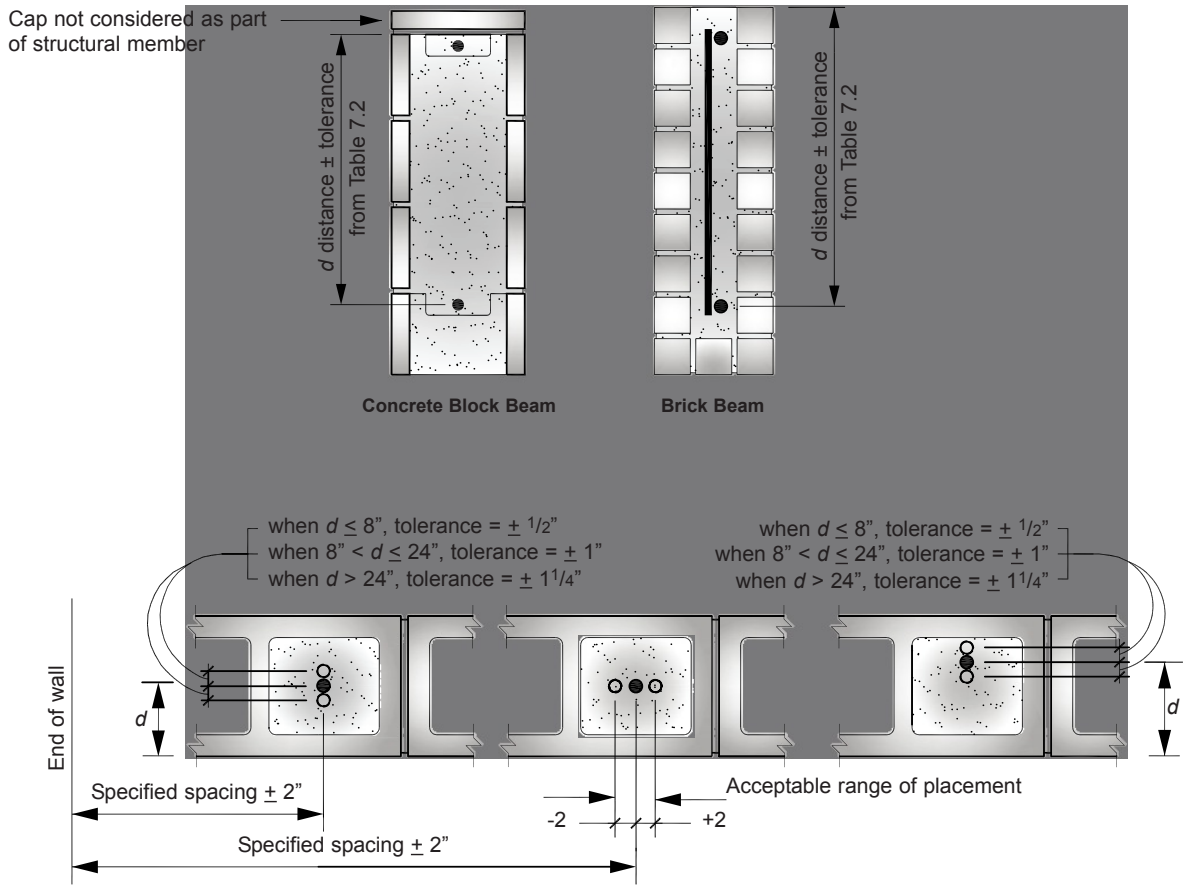


FIGURE 7.7 Illustration of tolerances for steel placement.

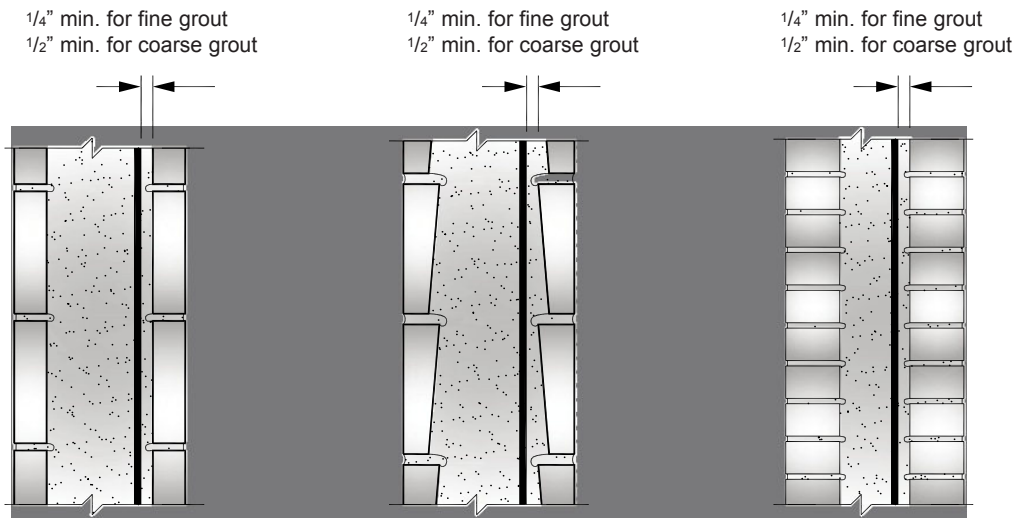


FIGURE 7.8 Reinforcing steel clearances.

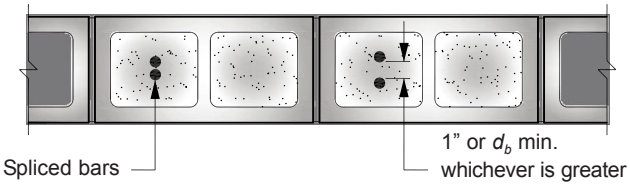


FIGURE 7.9 Minimum spacing of vertical reinforcement in cell.

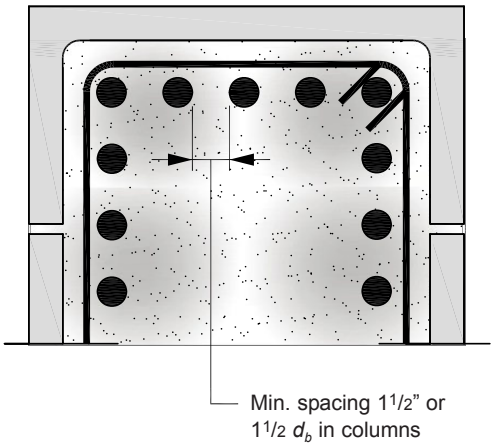
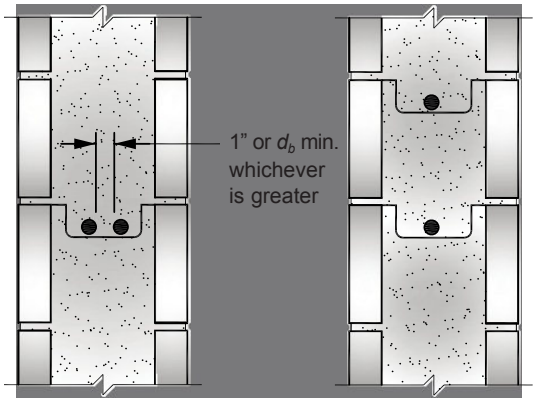


FIGURE 7.10 Minimum clearance between bars in a column.



d_b = Bar diameter

Alternate when approved by Architect/Engineer

FIGURE 7.11 Spacing of horizontal reinforcement in a concrete masonry wall.

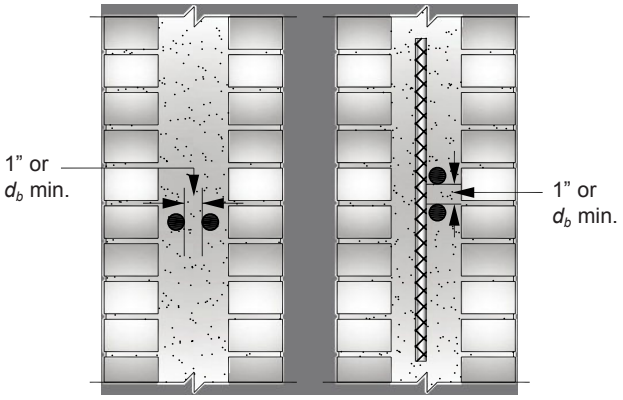


FIGURE 7.12 Spacing of horizontal reinforcement in a brick wall.

7.3.4 COVER OVER REINFORCEMENT

7.3.4.1 STEEL BARS

Reinforcement in masonry requires the following protective cover:

- 2 in. for bars larger than #5 and 1 1/2 in. for #5 bars and smaller, when exposed to earth or weather.
- 1 1/2 in. for all bars not exposed to earth or weather.

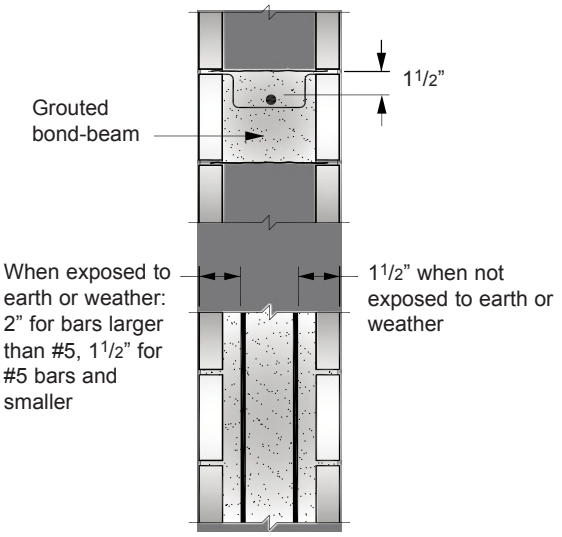


FIGURE 7.13 Minimum cover over reinforcing steel.

7.3.4.2 COVER FOR JOINT REINFORCEMENT AND TIES

Joint reinforcement and ties embedded in mortar require a 1/2 in. protective cover when not exposed to earth or weather and a 5/8 in. protective cover when exposed to earth or weather. MSJC Code Section 1.13.2.3 requires that joint reinforcement not exceed one-half the mortar joint thickness.

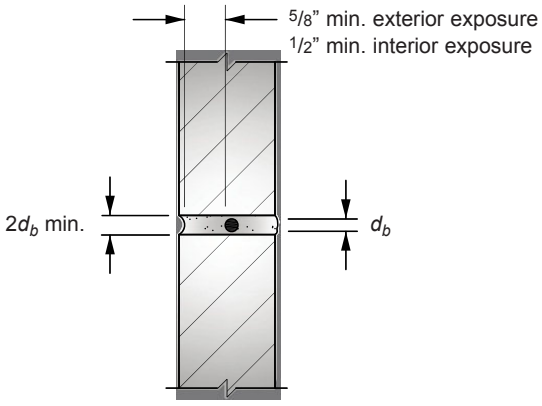


FIGURE 7.14 Cover over joint reinforcement.

7.3.4.3 COVER FOR COLUMN REINFORCEMENT

Lateral ties and longitudinal bars in columns must be placed with the same protective cover as noted in Section 7.3.4.1. Longitudinal bars are typically placed with at least 1 1/2 in. but usually not more than 5 in. (the limitation in previous UBC criteria) from the surface of the column.

7.4 EFFECTIVE DEPTH, *d*, IN A WALL

Determination of the *d* distance to the reinforcing steel perpendicular to the plane of the wall is given in Tables 7.3, 7.4, 7.5 and 7.6:

7.4.1 HOLLOW MASONRY UNIT WALLS

TABLE 7.3 Steel in Center of Cell, Block

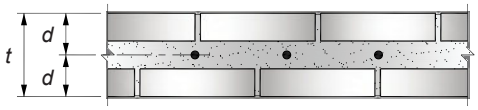
Hollow Masonry Units		
Nominal Thickness (in.)	**Actual Thickness (*t*) (in.)**	***d*** **(in.)**
6	5 5/8	2.8
8	7 5/8	3.8
10	9 5/8	4.8
12	11 5/8	5.8

TABLE 7.4 Steel Placement for Maximum *d*, Block

Nominal Thickness (in.)	Actual Thickness (*t*) (in.)	***d*** **(in.)**
8	7 5/8	5.25
10	9 5/8	7.25
12	11 5/8	9.00
16	15 5/8	13.00

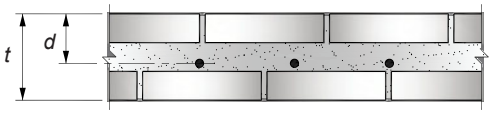
7.4.2 MULTI-WYTHE BRICK WALLS

TABLE 7.5 Steel in Center of Grout Space, Brick



Thickness, *t* (in.)	*d* (in.)
10.5 11.0 11.5	5.25 5.50 5.75
12.0 12.5 13.0	6.00 6.25 6.50
14.0 15.0 16.0	7.00 7.50 8.00

TABLE 7.6 Steel Placed for Maximum *d*, Brick



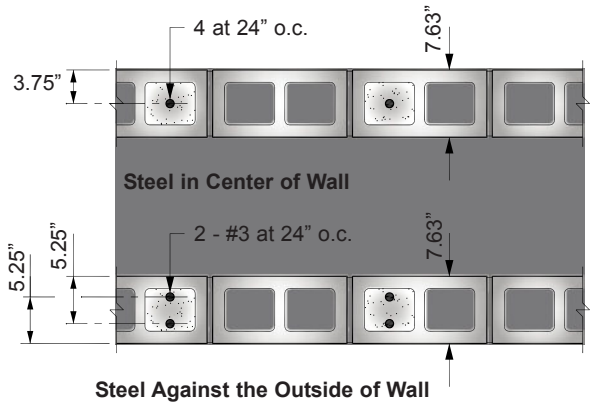
Actual Thickness, *t* (in.)	*d* (in.)	Actual Thickness, *t* (in.)	*d* (in.)
9.0 9.5 10.0	5.00 5.25 5.50	12.5 13.0 14.5	8.00 8.50 9.50
10.5 11.0	6.00 6.50	15.0 16.0	10.50 11.50
11.5 12.0	7.00 7.50	18.0 20.0	13.50 15.50

7.4.3 EFFECT OF *d* DISTANCE IN A WALL (LOCATION OF STEEL)

If a wall is subjected to lateral forces from either face, generally the more economical approach is to place the steel in the center of the wall rather than 1/2 the amount of steel against each outside face.

EXAMPLE 7-C Moment Capacity vs. *d* Distance.

Assume: f'_m = 1500 psi; n = 21.5 (concrete masonry).



Steel in Center of Wall

A_s = #4 at 24 in. o.c. = 0.10 sq in./ft

From Table GN-23b for d = 3.75 in.; ρ = 0.0022

From Table ASD-24b for f'_m = 1500 psi

ρ = 0.0022; K_f = 48.1

Moment = $K_f bd^2$

= 48.1 x 12 x 3.75^2

= 8117 in. lbs/ft

Steel Placed for Maximum *d* Distance

A_s = #3 at 24 in. o.c. each face; = 0.11 sq in./ft

From Table GN-23c for d = 5.25 in., ρ = 0.0009 (#3 at 24 in.)

From Table ASD-24b for f'_m = 1500 psi

ρ = 0.0009, K_f = 20.3

Moment = $K_f bd^2$

= 20.3 x 12 x 5.25^2 = 6714 in. lbs/ft

Although the moment capacity is greater with the steel against the outside face, twice the number of bars are required and increased steel placement costs plus the congestion with added steel, thereby negating the benefit of increased moment.

A_s = #5 at 24 in. o.c. = 1.55 sq in./ft

From Table GN-23b for d = 3.75; ρ = 0.0034

From Table ASD-24b for f'_m = 1500 psi

for ρ = 0.0034, K_f = 70.6

Moment = $K_f bd^2$

= 70.6 x 12 x 3.75^2

= 11,914 in. lbs/ft

Thus, a faster and more economical construction is to place one bar of steel in the center rather than a bar of steel on each side.

7.5 ANCHORAGE OF REINFORCING STEEL

7.5.1 DEVELOPMENT LENGTH, BOND

To develop a reinforcing bar, adequate development length, l_d, is required. The development length is based on the bond stress, the bar diameter, and the stress to be developed in the steel bar. The development lengths are based on either Allowable Stress Design (ASD) or Strength Design (SD) criteria. Table 7.7 gives the values for both ASD and SD based upon an allowable stress for Grade 60 reinforcement of F_s = 24,000 psi. The difference is that SD limits the development length at 72d_b.

TABLE 7.7 Development Length, l_d (in.)[1] Grade 60, F_s = 24,000 psi

Bar Size		l_d (in.) for Deformed Bars	
No.	Diameter, d_b (in.)	Bars in Tension[2,3]	Bars in Compression[2]
3	0.375	18	18
4	0.500	24	24
5	0.625	30	30
6	0.750	36	36
7	0.875	42	42
8	1.000	48	48
9	1.128	54	54
10	1.270	61	61
11	1.410	68	68

1. Based on MSJC Code Sections 2.1.10 and 3.3.3.3.

2. $l_d = \dfrac{0.13 d_b^{\,2} f_y \gamma}{K\sqrt{f'_m}}$ (MSJC Code Eqs 3-9 and 3-15)

3. For epoxy-coated bars increase by 50%

where:

K shall not exceed the lesser of masonry cover, spacing between adjacent reinforcement, nor 5 times d_b.

γ = 1.0 for No. 3 (M#10) through No. 5 (M#16) bars

γ = 1.3 for No. 6 (M#19) through No. 7 (M#22) bars

γ = 1.5 for No. 8 (M#25) through No. 11 (M#36) bars

7.5.2 HOOKS

A hook has the benefit of developing stress within a very short distance. When combined with a straight length of bar, the hook allows reinforcement to be fully developed over a shorter length than would be possible for a straight bar.

A standard 90 degree and 180 degree hook has a tension equivalent development of 11.25 d_b for ASD and 13 d_b for SD in accordance with MSJC Code Section 2.10.5 and 3.3.3.2 respectively.

According to 2005 MSJC Code Section 1.13.5, a 'standard hook' is defined as one of the following:

1. A 180-degree turn plus extension of at least 4 bar diameters but not less than 2 1/2 in. at free end of bar.

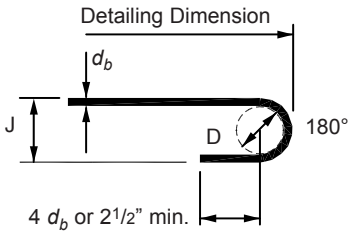


FIGURE 7.15a Standard 180° hook.

2. A 90-degree turn plus extension of at least 12 bar diameters at free end of bar.

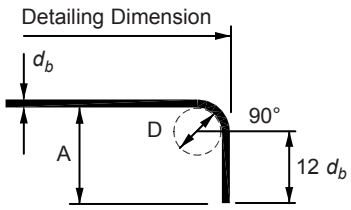


FIGURE 7.15b Standard 90° hook.

3. For stirrup and tie anchorage only, either a 90-degree or a 135-degree turn, plus an extension of at least 6 bar diameters.

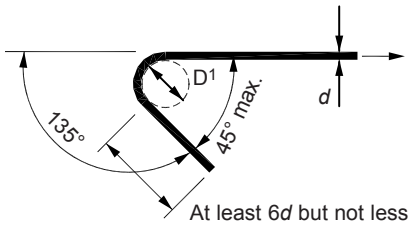


FIGURE 7.15c Standard 135° stirrup hook.

TABLE 7.8 Standard Hook and Bend

d_b = Bar Diameter D = Finish inside bend diameter		D = $5d_b$ for #3 through #7, Grade 40 D = $6d_b$ for #3 through #8, Grade 50/60 D = $8d_b$ for #9 through #11, Grade 50/60		
			Dimensions of Standard 180° Hooks	Dimensions of Standard 90° Hooks
Bar Size	Grade	D	J	A
#3	40	1 7/8"	2 5/8"	5.5"
#4	40	2 1/2"	3 1/2"	7.3"
#5	40	3 1/8"	4 3/8"	9.1"
#6	40	3 3/4"	5 1/4"	10.9"
#7	40	4 3/8"	6 1/8"	12.7"
#3	50/60	2 1/4"	3"	5.7"
#4	50/60	3"	4"	7.5"
#5	50/60	3 3/4"	5"	9.4"
#6	50/60	4 1/2"	6"	11.3"
#7	50/60	5 1/4"	7"	13.2"
#8	50/60	6"	8"	15.0"
#9	50/60	9"	11 1/4"	18.0"
#10	50/60	10"	12 1/2"	20.0"
#11	50/60	11"	13 3/4"	22.0"

TABLE 7.9 Minimum Diameters of Bend[1]

Bar Size	Grade	Minimum Diameter
No. 3 thru No. 7	40	5 bar diameters
No. 3 thru No. 8	60	6 bar diameters
No. 9 thru No. 11	60	8 bar diameters

1. MSJC Code Section 1.13.6.

The diameter of bend measured on the inside of the bar, including stirrups and ties, shall not be less than values specified in Table 7.8.

Hooks should not be placed in the tension portion of any beam, except at the ends of simple or cantilever beams or at the freely supported end of continuous or restrained beams.

MSJC Code Commentary Section 2.1.10.5.1 states that hooks should not be assumed to carry a load which would produce a tensile stress in the bar greater than 7500 psi.

Hooks are not effective in adding to the compressive resistance of bars.

Any mechanical device capable of developing the strength of the bar without damage to the masonry may be used in lieu of a hook. Data should be presented to show the adequacy of such devices.

MSJC Code Chapter 2 contains specific requirements for hooks and development shear reinforcement:

MSJC Code Section 2.1.10.5

2.1.10.5 *Hooks*

2.1.10.5.1 Standard hooks in tension shall be considered to develop an equivalent embedment length, l_e, equal to 11.25 d_b.

2.1.10.5.2 The effect of hooks for bars in compression shall be neglected in design computations.

2.1.10.6 *Development of shear reinforcement*

2.1.10.6.1 *Bar and wire reinforcement*

2.1.10.6.1.1 Shear reinforcement shall extend to a distance *d* from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends for its calculated stress.

2.1.10.6.1.2 The ends of single leg or U-stirrups shall be anchored by one of the following means:

(a) A standard hook plus an effective embedment of 0.5 l_d. The effective embedment of a stirrup leg shall be taken as the distance between the middepth of the member, *d*/2, and the start of the hook (point of tangency).

(b) For No. 5 bar (M #16) and D31 (MD200) wire and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of 0.33 l_d. The 0.33 l_d embedment of a stirrup leg shall be taken as the distance between middepth of member, *d*/2, and start of hook (point of tangency).

2.1.10.6.1.3 Between the anchored ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

2.1.10.6.1.4 Longitudinal bars bent to act as shear reinforcement, where extended into a region

of tension, shall be continuous with longitudinal reinforcement and, where extended into a region of compression, shall be developed beyond middepth of the member, *d*/2.

2.1.10.6.1.5 Pairs of U-stirrups or ties placed to form a closed unit shall be considered properly spliced when length of laps are 1.7 l_d. In grout at least 18 in. (457 mm) deep, such splices with $A_v f_y$ not more than 9,000 lb (40 032 N) per leg shall be permitted to be considered adequate if legs extend the full available depth of grout.

2.1.10.6.2 *Welded wire fabric*

2.1.10.6.2.1 For each leg of welded wire fabric forming simple U-stirrups, there shall be either:

(a) Two longitudinal wires at a 2-in. (50.8-mm) spacing along the member at the top of the U, or

(b) One longitudinal wire located not more than *d*/4 from the compression face and a second wire closer to the compression face and spaced not less than 2 in. (50.8 mm) from the first wire. The second wire shall be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than $8d_b$.

2.1.10.6.2.2 For each end of a single leg stirrup of welded smooth or deformed wire fabric, there shall be two longitudinal wires spaced a minimum of 2 in. (50.8 mm) with the inner wire placed at a distance at least *d*/4 or 2 in. (50.8 mm) from middepth of member, *d*/2. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

2.1.10.7 *Splices of reinforcement* — Lap splices, welded splices, or mechanical connections are permitted in accordance with the provisions of this section. All welding shall conform to AWS D1.4.

Likewise, Chapter 3 also has a specific provision for standard hooks:

MSJC Code Section 3.3.3.2

3.3.3.2 *Standard hooks* — The equivalent embedment length to develop standard hooks in tension, l_e , shall be determined by Eq. (3-14):

$$l_e = 13 d_b \qquad (3\text{-}14)$$

7.6 DEVELOPMENT LENGTH IN CONCRETE

For bars anchored in concrete, the development length and splices are based on ACI 318-05 Chapter 12.

ACI 318 - Section 12.1

12.1 — Development of reinforcement — General

12.1.1 — Calculated tension or compression in reinforcement at each section of structural concrete members shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks shall not be used to develop bars in compression.

12.1.2 — The values of $\sqrt{f'_c}$ used in this chapter shall not exceed 100 psi.

12.2 — Development of deformed bars and deformed wire in tension.

12.2.1 — Development length for deformed bars and deformed wire in tension, ℓ_d, shall be determined from either 12.2.2 or 12.2.3, but shall not be less than 12 in.

12.2.2 — For deformed bars or deformed wire, ℓ_d shall be a follows:

	No. 6 and smaller bars and deformed wires	**No. 7 and larger bars**
Clear spacing of bars or wires being developed or spliced not less than d_b, clear cover not less than d_b, and stirrups or ties throughout ℓ_d not less than the code minimum or Clear spacing of bars or wires being developed or spliced not less than $2d_b$ and clear cover not less than d_b	$\left(\frac{f_y \psi_t \psi_e \lambda}{25\sqrt{f'_c}}\right) d_b$	$\left(\frac{f_y \psi_t \psi_e \lambda}{20\sqrt{f'_c}}\right) d_b$
Other cases	$\left(\frac{3 f_y \psi_t \psi_e \lambda}{50\sqrt{f'_c}}\right) d_b$	$\left(\frac{3 f_y \psi_t \psi_e \lambda}{40\sqrt{f'_c}}\right) d_b$

12.2.3 — For deformed bars or deformed wire, ℓ_d shall be:

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s \lambda}{\frac{c_b + K_{tr}}{d_b}} \right) d_b \qquad (12\text{-}1)$$

in which the term $(c_b + K_{tr})/d_b$ shall not be taken greater than 2.5, and

$$K_{tr} = \frac{A_{tr} f_{yt}}{1500sn} \qquad (12\text{-}2)$$

where n is the number of bars or wires being spliced or developed along the plane of splitting. It shall be permitted to use $K_{tr} = 0$ as a design simplification even if transverse reinforcement is present.

12.2.4 — The factors used in the expressions for development of deformed bars and deformed wires in tension in 12.2 are as follows:

(a) Where horizontal reinforcement is placed such that more than 12 in. of fresh concrete is cast below the development length or splice, $\psi_t = 1.3$. For other situations, $\psi_t = 1.0$.

(b) For epoxy-coated bars or wires with cover less than $3d_b$, or clear spacing less than $6d_b$, $\psi_e = 1.5$. For all other epoxy-coated bars or wires, $\psi_e = 1.2$. For uncoated reinforcement, $\psi_e = 1.0$.

However, the product $\psi_t \psi_e$ need not be greater than 1.7.

(c) For No. 6 and smaller bars and deformed wires, $\psi_s = 0.8$. For No. 7 and larger bars, $\psi_s = 1.0$.

(d) Where lightweight concrete is used, $\lambda = 1.3$. However, when f_{ct} is specified, λ shall be permitted to be taken as $6.7\sqrt{f'_c}/f_{ct}$ but not less than 1.0. Where normalweight concrete is used, $\lambda = 1.0$.

12.2.5 — Excess Reinforcement

Reduction in ℓ_d shall be permitted where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for f_y is specifically required or the reinforcement is designed under provisions of 21.2.1.4 ..... (A_s required)/(A_s provided).

12.3 — Development of deformed bars and deformed wire in compression

12.3.1 — Development length for deformed bars and deformed wire in compression, ℓ_{dc}, shall be determined from 12.3.2 and applicable modification factors of 12.3.3, but ℓ_{dc} shall not be less than 8 in.

12.3.2 — For deformed bars and deformed wire, ℓ_{dc} shall be taken as the larger of $\left(0.02 f_y/\sqrt{f'_c}\right)d_b$ and $(0.0003 f_y)d_b$, where the constant 0.0003 carries the unit of in.^{2}/lb.

12.3.3 — Length ℓ_{dc} in 12.3.2 shall be permitted to be multiplied by the applicable factors for:

(a) Reinforcement in excess of that required by analysis.....(A_s required)/(A_s provided).

(b) Reinforcement enclosed within spiral reinforcement not less than 1/4 in. diameter and not more than 4 in. pitch or within No. 4 ties in conformance with 7.10.5 and spaced at not more than 4 in. on center.....0.75.

7.7 LAP SPLICES FOR REINFORCING STEEL

In general, a reinforced masonry wall cannot be built using a single continuous length of reinforcing steel. Instead, the steel is placed using bars cut to manageable lengths. For these shorter lengths of steel to function as continuous reinforcement, they must be connected in some fashion.

The usual method is to lap bars at specified lengths. IBC Allowable Stress Design requires that reinforcing bars in tension or compression have a lapped length of 40 bar diameters for Grade 40 (300) Steel and 48 bar diameters for Grade 60 (420) steel based on Equation 21-2. Additional lap requirements are contained in the applicable sections of MSJC Code Section 2.1.10.7.

IBC Section 2107.5

2107.5 ACI 530/ASCE 5/TMS 402, Section 2.1.10.7.1.1, lap splices. Modify Section 2.1.10.7.1.1 as follows:

2.1.10.7.1.1 The minimum length of lap splices for reinforcing bars in tension or compression, l_d, shall be:

$l_d = 0.002 d_b f_s$ **(Equation 21-2)**

For SI: $l_d = 0.29 d_b f_s$

but not less than 12 inches (305 mm). In no case shall the length of the lapped splice be less than 40 bar diameters.

where:

d_b = Diameter of reinforcement, inches (mm).

f_s = Computed stress in reinforcement due to design loads, psi (MPa).

In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tension stress, F_s, the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase shall be permitted.

Where epoxy coated bars are used, lap length shall be increased by 50 percent.

2107.6 ACI 530/ASCE 5/TMS 402, Section 2.1.10.7, splices of reinforcement. Modify Section 2.1.10.7 as follows:

2.1.10.7 Splices of reinforcement. Lap splices, welded splices or mechanical splices are permitted in accordance with the provisions of this section. All welding shall conform to AWS D1.4. Reinforcement larger than No. 9 (M #29) shall be spliced using mechanical connections in accordance with Section 2.1.10.7.3.

MSJC Code Section 2.1.10.7.1

2.1.10.7.1 *Lap splices*

2.1.10.7.1.2 Bars spliced by noncontact lap splices shall not be spaced transversely farther apart than one-fifth the required length of lap nor more than 8 in. (203 mm).

2.1.10.7.2 *Welded splices* — Welded splices shall have the bars butted and welded to develop in tension at least 125 percent of the specified yield strength of the bar.

2.1.10.7.3 *Mechanical connections* — Mechanical connections shall have the bars connected to develop in tension or compression, as required, at least 125 percent of the specified yield strength of the bar.

2.1.10.7.4 *End-bearing splices*

2.1.10.7.4.1 In bars required for compression only, the transmission of compressive stress by bearing of square cut ends held in concentric contact by a suitable device is permitted.

2.1.10.7.4.2 Bar ends shall terminate in flat surfaces within $1^1/_2$ degree of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

2.1.10.7.4.3 End-bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

TABLE 7.10 ASD Length of Lap (in.)[1]

Bar Size		Laps for Grade 40 (Tension or Compression)		Laps for Grade 60 (Tension or Compression)	
No.	Dia., d_b (in.)	40 d_b Min. (in.)	60 d_b Min.[2,3] (in.)	48 d_b Min. (in.)	72 d_b Min.[2,3] (in.)
3	0.375	15	23	18	27
4	0.500	20	30	24	36
5	0.625	25	38	30	45
6	0.750	30	45	36	54
7	0.875	35	53	42	63
8	1.000	40	60	48	72
9[4]	1.128	45	68	54	81

1. Based on IBC Section 2107.5
2. 50% lap splice increase for regions of moment where design tensile stresses are greater than 80% of the allowable steel tension stress (IBC Section 2107.5)
3. 50% lap splice increase where epoxy coated bars are used (IBC Section 2107.5)
4. Bars larger than #9 must be mechanically spliced or welded (IBC Section 2107.6)

Strength Design splice requirements are given in MSJC Code Sections 3.3.3.3.1 and 3.3.3.4:

MSJC Code Sections 3.3.3.3.1 and 3.3.3.4

3.3.3.3.1 Bars spliced by noncontact lap splices shall not be spaced farther apart than one-fifth the required length of lap nor more than 8 in. (203 mm).

and

3.3.3.4 *Splices* — Reinforcement splices shall comply with one of the following:

(a) The minimum length of lap for bars shall be 12 in. (305 mm) or the development length determined by Eq. (3-15), whichever is greater.

$$l_d = \frac{0.13 d_b^2 f_y \gamma}{K\sqrt{f'_m}} \qquad (3\text{-}15)$$

K shall not exceed the lesser of the masonry cover, clear spacing between adjacent reinforcement, nor 5 times d_b.

$\gamma = 1.0$ for No. 3 (M#10) through No. 5 (M#16) bars;
$\gamma = 1.3$ for No. 6 (M#19) through No. 7 (M#22) bars; and
$\gamma = 1.5$ for No. 8 (M#25) through No. 9 (M#29) bars.

When epoxy-coated reinforcing bars are used, development length determined by Eq. (3-15) shall be increased by 50 percent.

(b) A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required.

(c) Mechanical splices shall have the bars connected to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required.

Splices should be at certain locations as indicated on the project drawings and in such a manner that the structural strength of the member will not be reduced. The designer may consider detailing staggered laps even though this is not a code requirement.

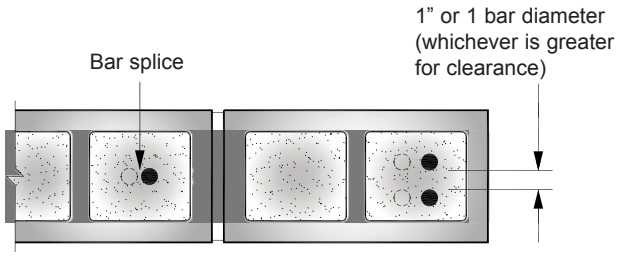


FIGURE 7.16 Lap splice of steel in cell.

Although the 2005 MSJC Code does not require lap splices for joint reinforcement, historic codes have required a nominal lap splice. For reference, the 2008 MSJC Code requires a 6 in. lap splice for joint reinforcement.

7.8 ANCHOR BOLTS

7.8.1 ANCHOR BOLTS IN MASONRY

Anchor bolts are used to tie masonry to structural supports and to transfer loads from masonry attachments such as ledgers, and sill plates. Some examples where anchor bolts may be used are connections between masonry walls, roofs, floors, ledger beams, and signs.

Conventional embedded anchor bolts are commonly specified as bent bar anchor bolts, plate anchor bolts and headed anchor bolts. They are available in standard sizes (diameters and lengths) or can be fabricated to meet specific project requirements.

Anchor bolts are commonly embedded at:

1. The surface of walls – for connecting relief angles and wood or steel ledger beams to the walls.
2. The top of walls – for attaching sill plates and base plates to the walls.
3. The top of columns – for anchoring steel bearing plates onto the columns.

Anchor bolts are generally divided into two categories:

1. Embedded anchor bolts which are placed and grouted during construction, and
2. Drilled-in anchors which are placed after construction of the masonry.

Anchor bolts are subjected to shear and tension forces resulting from gravity loads, earthquakes, wind forces, differential movements, dynamic vibrations, etc. The magnitude of these loads vary significantly.

The values for shear and tension given in the code are generalized and in some cases very conservative. Tables ASD-7a, ASD-7b, and ASD-8a give allowable shear and tension capacities of typical size anchor bolts based on MSJC Code Section 2.1.4.2.

Note that anchor bolts subjected to combined shear and tension forces must be designed by MSJC Code Section 2.1.4.2.4, Equation 2-7:

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \leq 1.0$$

Where:

b_a = total applied design axial force on an anchor bolt
B_a = allowable axial force on an anchor bolt
b_v = total applied design shear force on an anchor bolt
B_v = allowable shear force on an anchor bolt

MSJC Code Section 2.1.4 provides the details for ASD anchor bolt design:

MSJC Code Section 2.1.4

2.1.4 *Anchor bolts solidly grouted in masonry*

2.1.4.1 *Test design requirements* — Except as provided in Section 2.1.4.2, anchor bolts shall be designed based on the following provisions.

2.1.4.1.1 Anchors shall be tested in accordance with ASTM E 488 under stresses and conditions representing intended use, except that a minimum of five tests shall be performed.

2.1.4.1.2 Allowable loads shall not exceed 20 percent of the average tested strength.

2.1.4.2 *Plate, headed, and bent-bar anchor bolts* — The allowable loads for plate anchors, headed anchor

bolts, and bent-bar anchor bolts (J or L type) embedded in masonry shall be determined in accordance with the provisions of Sections 2.1.4.2.1 through 2.1.4.2.4.

2.1.4.2.1 The minimum effective embedment length shall be 4 bolt diameters, but not less than 2 in. (50.8 mm).

2.1.4.2.2 The allowable load in tension shall be the lesser of that given by Eq. (2-1) or Eq. (2-2).

$$B_a = 0.5A_p\sqrt{f'_m} \quad (2\text{-}1)$$

$$B_a = 0.2A_b f_y \quad (2\text{-}2)$$

2.1.4.2.2.1 The area A_p shall be the lesser of Eq. (2-3) or Eq. (2-4). Where the projected areas of adjacent anchor bolts overlap, A_p of each bolt shall be reduced by one-half of the overlapping area. That portion of the projected area falling in an open cell or core shall be deducted from the value of A_p calculated using Eq. (2-3) or (2-4).

$$A_p = \pi l_b^2 \quad (2\text{-}3)$$

$$A_p = \pi l_{be}^2 \quad (2\text{-}4)$$

2.1.4.2.2.2 The effective embedment length of plate or headed bolts, l_b, shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the plate or head of the anchor bolt.

2.1.4.2.2.3 The effective embedment length of bent anchors, l_b, shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the bent end minus one anchor bolt diameter.

2.1.4.2.3 The allowable load in shear, where l_{be} equals or exceeds 12 bolt diameters, shall be the lesser of that given by Eq. (2-5) or Eq. (2-6).

$$B_v = 350\sqrt[4]{f'_m A_b} \quad (2\text{-}5)$$

$$B_v = 0.12A_b f_y \quad (2\text{-}6)$$

Where l_{be} is less than 12 bolt diameters, the value of B_v in Eq. (2-5) shall be reduced by linear interpolation to zero at an l_{be} distance of 1 in. (25.4 mm).

Likewise, MSJC Code Section 3.1.6 provides for anchor bolts using Strength Design:

MSJC Code Section 3.1.6

3.1.6 *Headed and bent-bar anchor bolts.*

All embedded bolts shall be grouted in place with at least 1/2 in. (12.7 mm) of grout between the bolt and the masonry, except that 1/4 in. (6.4 mm) diameter bolts are permitted to be placed in bed joints that are at least 1/2 in. (12.7 mm) in thickness.

3.1.6.1 *Nominal axial tensile strength of headed anchor bolts* — The nominal axial tensile strength, B_{an}, of headed anchor bolts embedded in masonry shall be computed by Eq. (3-1) (strength governed by masonry breakout) and Eq. (3-2) (strength governed by steel). In computing the capacity, the smaller of the design strengths shall be used.

$$B_{an} = 4A_{pt}\sqrt{f'_m} \quad (3\text{-}1)$$

$$B_{an} = A_b f_y \quad (3\text{-}2)$$

3.1.6.1.1 *Projected area of masonry for headed anchor bolts* — The projected area, A_{pt}, in Eq. 3-1) shall be determined by Eq. (3-3).

$$A_{pt} = \pi l_b^2 \quad (3\text{-}3)$$

Where the projected areas, A_{pt}, of adjacent headed anchor bolts overlap, the projected area, A_{pt}, of each bolt shall be reduced by one-half of the overlapping area. The portion of the projected area overlapping an open cell, open head joint, or that is outside the wall shall be deducted from the value of A_{pt} calculated using Eq. (3-3).

3.1.6.1.2 *Effective embedment length for headed anchor bolts* — The effective embedment length for a headed anchor bolt, l_b, shall be the length of the embedment measured perpendicular from the masonry surface to the bearing surface of the anchor head. The minimum effective embedment length for headed anchor bolts resisting axial forces shall be 4 bolt diameters or in. (50.8 mm), whichever is greater.

3.1.6.2 *Nominal axial tensile strength of bent-bar anchor bolts* — The nominal axial tensile strength, B_{an}, for bent-bar anchor bolts (J- or L-bolts) embedded in masonry shall be computed by Eq. (3-4) (strength governed by masonry breakout), Eq. (3-5) (strength governed by steel), and Eq. (3-6) (strength governed by anchor pullout). In computing the capacity, the smaller of the design strengths shall be used.

$$B_{an} = 4A_{pt}\sqrt{f'_m} \quad (3\text{-}4)$$

$$B_{an} = A_b f_y \quad (3\text{-}5)$$

$$B_{an} = 1.5f'_m e_b d_b + [300\pi (l_b + e_b + d_b)d_b] \quad (3\text{-}6)$$

The second term in Eq. (3-6) shall be included only if the specified quality assurance program includes verification that shanks of J- and L-bolts are free of debris, oil, and grease when installed.

3.1.6.2.1 *Projected area of masonry for bent-bar anchor bolts* — The projected area, A_{pt}, in Eq. (3-4) shall be determined by Eq. (3-7).

$$A_{pt} = \pi l_b^2 \quad (3\text{-}7)$$

Where the projected areas, A_{pt}, of adjacent bent-bar anchor bolts overlap, the projected area, A_{pt}, of each bolt shall be reduced by one-half of the overlapping area. That portion of the projected area overlapping an open cell, open head joint, or that is outside the wall shall be deducted from the value of A_{pt} calculated using Eq. (3-7).

3.1.6.2.2 *Effective embedment length of bent-bar anchor bolts* — The effective embedment for a bent-bar anchor bolt, l_b, shall be the length of embedment measured perpendicular from the masonry surface to the bearing surface of the bent end, minus one anchor bolt diameter. The minimum effective embedment length for bent-bar anchor bolts resisting axial forces shall be 4 bolt diameters or 2 in. (50.8 mm), whichever is greater.

3.1.6.3 *Nominal shear strength of headed and bent-bar anchor bolts* — The nominal shear strength, B_{vn}, shall be computed by Eq. (3-8) (strength governed by masonry breakout) and Eq. (3-9) (strength governed by steel). In computing the capacity, the smaller of the design strengths shall be used.

$$B_{vn} = 4A_{pv}\sqrt{f'_m} \tag{3-8}$$

$$B_{vn} = 0.6A_b f_y \tag{3-9}$$

3.1.6.3.1 *Projected area of masonry* — The area A_{pv} in Eq. (3-8) shall be determined from Eq. (3-10).

$$A_{pv} = \frac{\pi l_{be}^{\,2}}{2} \tag{3-10}$$

3.1.6.3.2 *Minimum effective embedment length* — The minimum effective embedment length for headed or bent-bar anchor bolts resisting shear forces shall be 4 bolt diameters, or 2 in. (50.8 mm), whichever is greater.

3.1.6.4 *Combined axial and shear strength of anchor bolts* — Anchor bolts subjected to combined shear and tension shall be designed to satisfy Eq. (3-11).

$$\frac{b_{af}}{\phi B_{an}} + \frac{b_{vf}}{\phi B_{vn}} \le 1 \tag{3-11}$$

ϕB_{an} and ϕB_{vn} used in Eq. (3-11) shall be the governing design tensile and shear strengths, respectively.

Tables SD-91, SD-92, SD-93 and Table GN-91 give Strength Design values for shear and tension capacities of typical size anchor bolts based on MSJC Code Section 3.1.6.

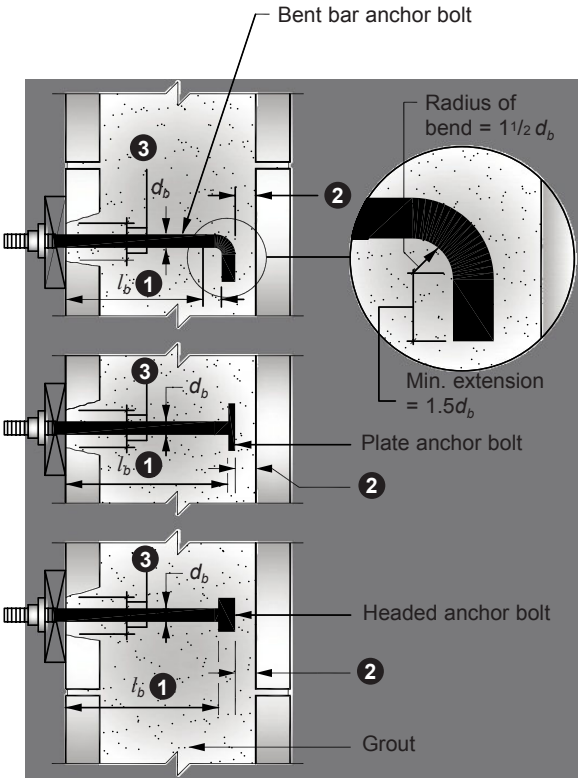
7.8.2 EFFECTIVE EMBEDMENT LENGTH

The minimum embedment depth l_b per MSJC Code Section 2.1.4.2.1 or 3.1.6.1.2 is 4 bolt diameters but not less than 2 in. (see Figure 7.17). Table 7.11 lists minimum embedment depths for common size anchor bolts.

TABLE 7.11 Minimum Anchor Bolt Embedment Depth[1] (in.)

Diameter (in.)	Minimum Embedment (in.)
3/8	2
1/2	2
5/8	2 1/2
3/4	3
7/8	3 1/2
1	4
1 1/8	4 1/2
1 1/4	5

1. Based on MSJC Code Section 2.1.4.2.1 or 3.1.6.1.2 with a minimum embedment of 4 bolt diameters but not less than 2 in.



1. Minimum embedment length $l_b = 4d_b$ but l_b may not be less than 2".
2. 1/4" for fine grout, 1/2" for coarse (pea gravel) grout
3. 1/2" min. Strength Design

FIGURE 7.17 Effective embedment.

7.8.3 MINIMUM EDGE DISTANCE AND SPACING REQUIREMENTS

The minimum edge distance, l_{be}, measured from the edge of the masonry parallel with the anchor bolt to the surface of the anchor bolt must be 12 bolt diameters or reduced by interpolation in accordance with MSJC Code Section 2.1.4.2.3. The designer may wish to consider this approach when using Strength Design which does not contain the same provision.

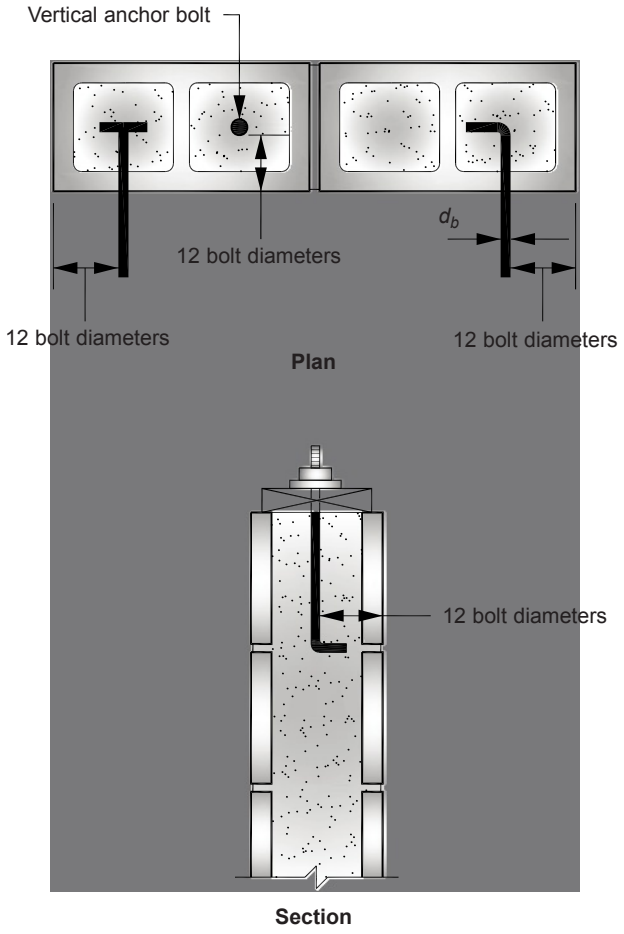


FIGURE 7.18 Minimum edge distance to achieve full ASD capacity of anchor bolts.

7.9 BEAMS

7.9.1 GENERAL

The MSJC Code does not specify a minimum amount of steel or steel ratio, ρ, for flexural beams. Engineering practice generally recommends that for masonry beams, the minimum reinforcement ratio, ρ, be not less than $80/f_y$. Therefore, for grade 60 steel, the minimum steel ratio should be ρ = 80/60,000 = 0.0013.

7.9.2 CONTINUITY OF REINFORCING STEEL IN FLEXURAL MEMBERS

Requirements for beams to span for continuity and bearing for flexural members is given in the MSJC Code Section 2.3.3.4:

MSJC Code Section 2.3.3.4

2.3.3.4 *Beams*

2.3.3.4.1 Span length of members not built integrally with supports shall be taken as the clear span plus depth of member, but need not exceed the distance between centers of supports.

2.3.3.4.2 In analysis of members that are continuous over supports for determination of moments, span length shall be taken as the distance between centers of supports.

2.3.3.4.3 Length of bearing of beams on their supports shall be a minimum of 4 in. (102 mm) in the direction of span.

2.3.3.4.4 The compression face of beams shall be laterally supported at a maximum spacing of 32 times the beam thickness.

2.3.3.4.5 Beams shall be designed to meet the deflection requirements of Section 1.10.1.

MSJC Code Section 2.1.10.4 provides continuity and general embedment requirements as applied to continuous beams and other flexural members as shown in Figure 7.19.

Some design guidelines are summarized below:

- Except at supports or at the free end of cantilevers, extend every reinforcing bar beyond the point at which it is no longer needed to resist tensile stress for a distance equal to 12 bar diameters or the depth of the beam, whichever is greater. No flexural bar shall be terminated in a tensile zone unless one of the following conditions is satisfied:
  - The shear is not over one half that permitted, including allowance for shear reinforcement, where provided.
  - Additional shear reinforcement in excess of that required is provided each way from the cutoff distance equal to the depth of the beam. Do not exceed $d/(8\beta_b)$ for shear reinforcement spacing.
  - The continuing bars provide double the area required for flexure at that point or double the perimeter required for reinforcing bond.
  - Extend at least one third of the total reinforcement provided for negative moment at the support beyond the

extreme position of the point of inflection a distance sufficient to develop one half the allowable stress in the bar, not less than one sixteenth of the clear span, or the depth *d* of the member, whichever is greater.

- Adequately anchor tensile reinforcement for negative moment in any span of a continuous restrained or cantilever beam, or in any member of a rigid frame by reinforcement bond, hooks or mechanical anchors in or through the supporting member.
- Extend at least one third of the required positive moment reinforcement in simple beams or at the freely supported end of continuous beams along the same face of the beam into the support at least 6 inches. At least one fourth of the required positive moment reinforcement at the continuous end of continuous beams shall extend along the same face of the beam into the support at least 6 inches.

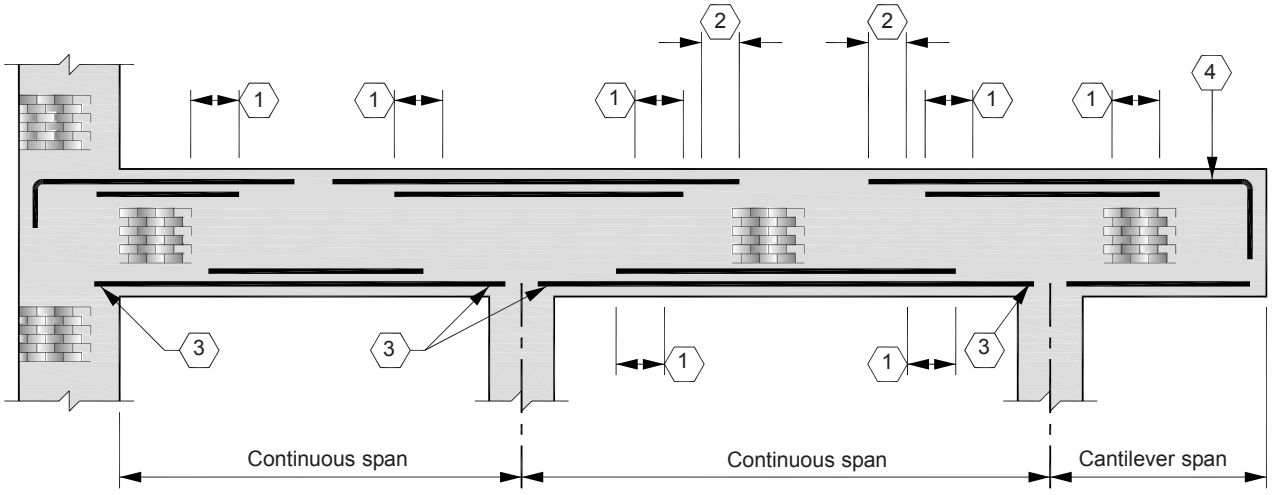
Anchor compression reinforcement in flexural members by ties or stirrups not less than 1/4 inch in diameter, spaced not farther apart than 16 bar diameters, 48 tie diameters or least column dimension, whichever is less. Such ties or stirrups shall be used throughout the distance where compression steel is required.

Figure 7.19 shows the design guidelines for continuity in flexural members. The provisions of MSJC Code Section 2.1.10.4 must be followed and may result in continuous reinforcement through the length of the beam. Continuous bars which are adequately anchored and lapped provide a certain amount of redundancy and added safety into the structure. Continuous reinforcement eliminates much of the concern over whether the bars are properly placed in the field.

Similarly, ending bars in tension zones may allow cracks to form at the ends of the bars. Although the MSJC Code requires additional precautions for shear near the ends of such terminated bars, extension of these bars and anchorage into the compression zone of the beam is recommended.

7.10 TIES FOR BEAM STEEL IN COMPRESSION

Compression reinforcement in flexural members should be tied to secure in position and to prevent buckling. Reinforcement used to resist compression loads must also be confined by ties to prevent buckling. MSJC Code Section 2.3.2.2.1 requires confinement reinforcement in accordance with the requirements of Section 2.1.6.5.



1 Extend steel at least effective depth of member, *d*, or 12 bar diameters, whichever is greater, beyond the point where it is no longer required for flexure (MSJC Code Section 2.1.10.4.1.3).

2 Extend at least one third of negative moment reinforcing beyond the inflection point for the distance of 12 bar diameters, 1/16 span, or the effective depth, *d* (MSJC Code Section 2.1.10.4.2).

3 Extend at least one fourth of the positive reinforcement from continuous beams into the support a distance of 6 in.

4 No flexural bars shall be terminated in a tension zone unless additional shear reinforcement is added (MSJC Code Section 2.1.10.4.1.5, similar to ACI 318 Section 12.10.5).

FIGURE 7.19 Steel detailing for continuity.

MSJC Code Section 2.3.2.2.1

2.3.2.2.1 The compressive resistance of steel reinforcement shall be neglected unless lateral reinforcement is provided in compliance with the requirements of Section 2.1.6.5.

MSJC Code Section 2.1.6.5 requires lateral ties or stirrups to be at least 1/4 inch in diameter and spaced not farther apart than 16 bar diameters, 48 tie diameters or least cross-section dimension, whichever is less. Such ties or stirrups shall be used throughout the distance where compression steel is required.

Note that these requirements are not for all compression members, such as pilasters, but are for members intended to have the compression reinforcement count as compressive force-carrying elements, such as in a column.

MSJC Code Section 2.1.6.5

2.1.6.5 *Lateral ties* — Lateral ties shall conform to the following:

(a) Longitudinal reinforcement shall be enclosed by lateral ties at least 1/4 in. (6.4 mm) in diameter.

(b) Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 lateral tie bar or wire diameters, or least cross-sectional dimension of the member.

(c) Lateral ties shall be arranged so that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135 degrees. No bar shall be farther than 6 in. (152 mm) clear on each side along the lateral tie from such a laterally supported bar. Lateral ties shall be placed in either a mortar joint or in grout. Where longitudinal bars are located around the perimeter of a circle, a complete circular lateral tie is permitted. Lap length for circular ties shall be 48 tie diameters.

(d) Lateral ties shall be located vertically not more than one-half lateral tie spacing above the top of footing or slab in any story, and shall be spaced not more than one-half a lateral tie spacing below the lowest horizontal reinforcement in beam, girder, slab, or drop panel above.

(e) Where beams or brackets frame into a column from four directions, lateral ties shall be permitted to be terminated not more than 3 in. (76.2 mm) below the lowest reinforcement in the shallowest of such beams or brackets.

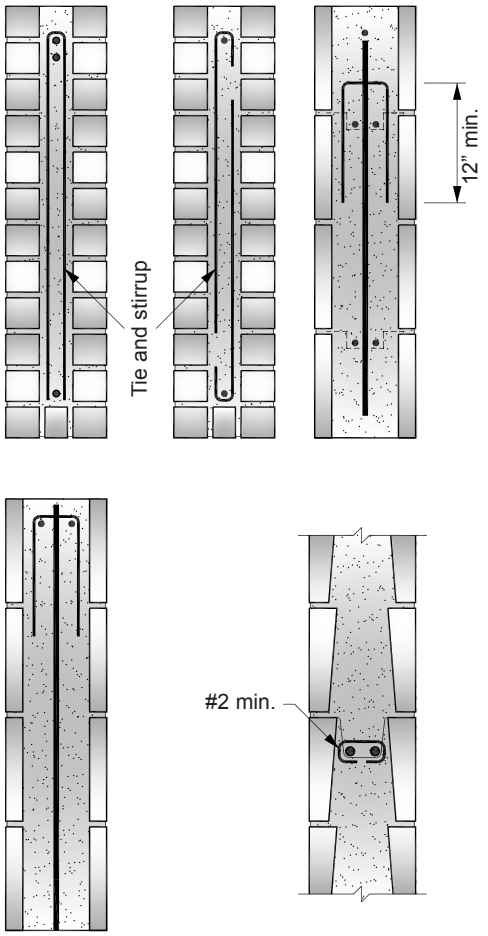


FIGURE 7.20 Ties for compression steel in beams.

7.11 SHEAR REINFORCEMENT REQUIREMENTS IN BEAMS

7.11.1 GENERAL

MSJC Code Section 2.3.5 requires that shear reinforcement be provided when the computed shear stress exceeds the allowable shear stress and that the shear reinforcement be designed to resist the entire shear force.

For beams, the maximum shear forces are generally at the end of the beams, with less shear force near the middle. Thus, the shear reinforcing bars will be required to be spaced more closely near the beam end. As a minimum, MSJC Code requires that web reinforcement be spaced so that every potential 45-degree crack extending from a point at *d*/2 of the beam to the longitudinal tension steel be crossed by at least one shear reinforcing bar.

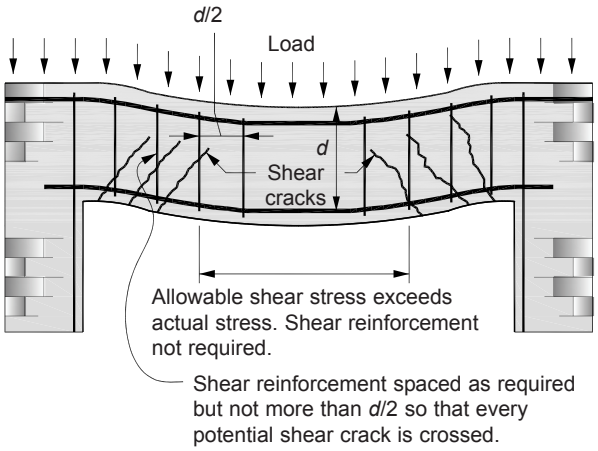


FIGURE 7.21 Beam showing potential shear cracks and shear reinforcing bars.

7.11.2 TYPES OF SHEAR REINFORCEMENT

Web reinforcement may consist of:

1. Stirrups or web reinforcement bars perpendicular to the longitudinal steel.
2. Longitudinal bars bent so that the axis of the inclined portion of the bar makes an angle of 15 degrees or more with the axis of the longitudinal portion of the bar.
3. Special arrangements of bars with adequate provisions to prevent slip of bars or splitting of masonry by the reinforcement.

7.11.3 ANCHORAGE OF SHEAR REINFORCEMENT

Bars used as shear reinforcement must be anchored at each end by one of the following methods.

1. Hooking tightly around the longitudinal reinforcement through 180 degrees.
2. Embedment above or below the mid-depth of the beam on the compression side a distance sufficient to develop the stress in the bar for deformed bars.
3. By a standard hook, as defined in MSJC Code Section 2.1.10.5, plus embedment sufficient to develop the remainder of the stress to which the bar is subjected (0.5 l_d). The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.

The ends of bars forming a single U or multiple U stirrup shall be anchored by one of the methods set forth in Items 1 through 3 above or shall comply with MSJC Code Section 2.1.10.6

7.11.4 SHEAR REINFORCEMENT DETAILS

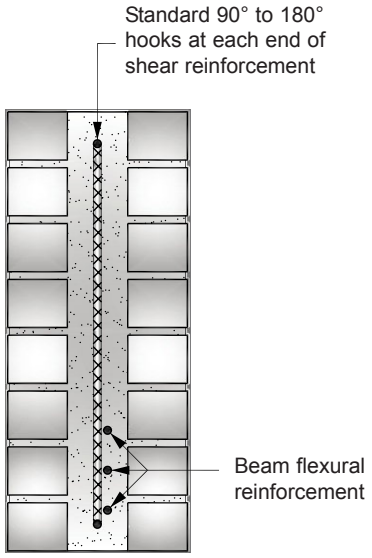


FIGURE 7.22 Cross section of beam showing vertical shear reinforcing steel.

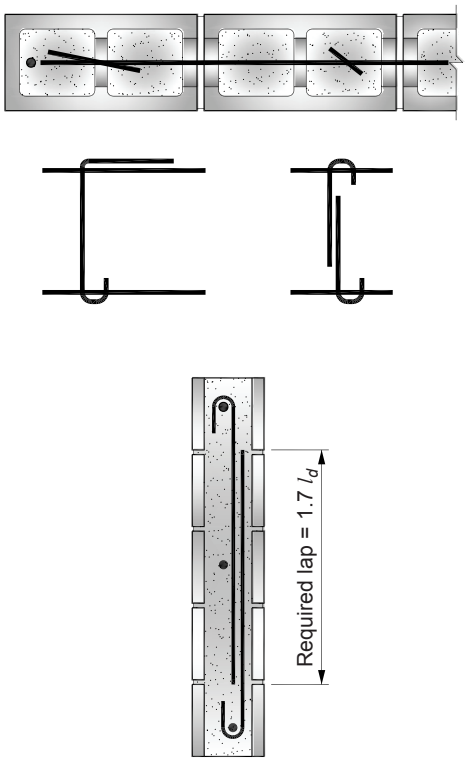


FIGURE 7.23 Anchorage details for shear reinforcement.

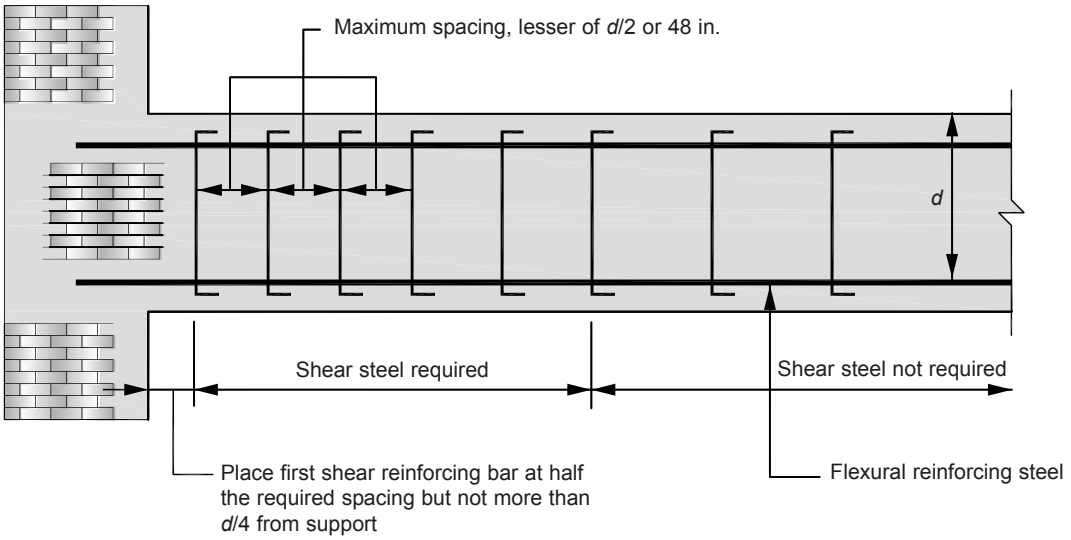


FIGURE 7.24 Vertical web or shear reinforcing steel arrangement for beams.

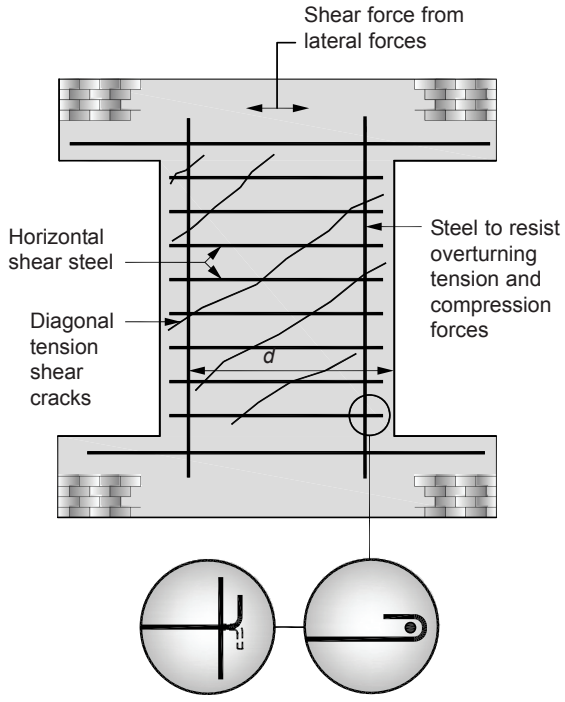


FIGURE 7.25 Shear wall reinforced with horizontal steel to resist lateral shear forces induced by wind or seismic forces.

7.12 COMPRESSION JAMB STEEL AT THE END OF PIERS AND SHEAR WALLS

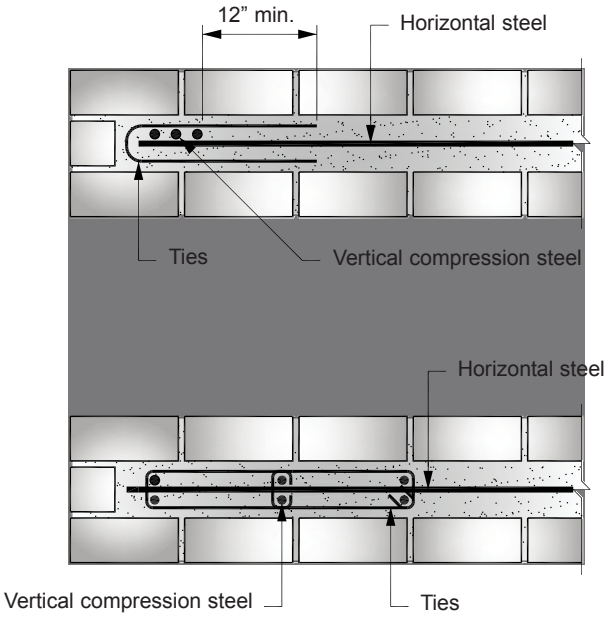


FIGURE 7.26 Door jamb reinforcement at the ends of brick walls or piers.

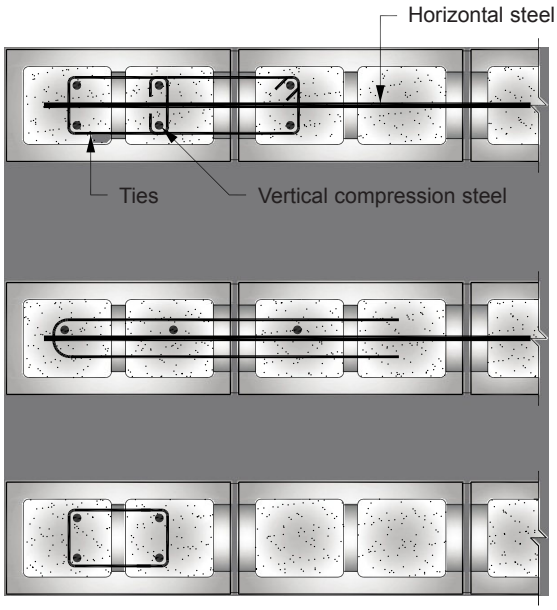


FIGURE 7.27 Door jamb reinforcement at the ends of concrete masonry walls.

7.13 COLUMNS

7.13.1 GENERAL

In the design of columns, vertical reinforcing steel significantly contributes to the load-carrying capacity of the member because the ties prevent reinforcing steel from buckling. MSJC Code Sections 2.1.6 and 3.3.4.4 provide criteria for column reinforcement.

MSJC Code Section 2.1.6

2.1.6 *Columns*

Design of columns shall meet the general requirements of this section.

2.1.6.1 Minimum side dimension shall be 8 in. (203 mm) nominal.

2.1.6.2 The ratio between the effective height and least nominal dimension shall not exceed 25.

2.1.6.3 Columns shall be designed to resist applied loads. As a minimum, columns shall be designed to resist loads with an eccentricity equal to 0.1 times each side dimension. Consider each axis independently.

2.1.6.4 Vertical column reinforcement shall not be less than $0.0025A_n$ nor exceed $0.04A_n$. The minimum number of bars shall be four.

MSJC Code Section 3.3.4.4

3.3.4.4 *Columns*

3.3.4.4.1 *Longitudinal reinforcement* — Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column, and shall comply with the following:

(a) Maximum reinforcement areas shall be determined in accordance with Section 3.3.3.5, but shall not exceed $0.04\ A_n$.
(b) Minimum reinforcement area shall be $0.0025\ A_n$.
(c) Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.

3.3.4.4.2 *Lateral ties* — Lateral ties shall be provided in accordance with Section 2.1.6.5.

3.3.4.4.3 *Construction* — Columns shall be solid grouted.

3.3.4.4.4 *Dimensional limits* — Dimensions shall be in accordance with the following:

(a) The nominal width of a column shall not be less than 8 in. (203 mm).
(b) The distance between lateral supports of a column shall not exceed 30 times its nominal width.
(c) The nominal depth of a column shall not be less than 8 in. (203 mm) and no greater than three times its nominal width.

Columns may be categorized by their location; they may be isolated (free standing), projecting from a wall, or flush in a wall. The least dimension of columns should not be less than 8 inches.

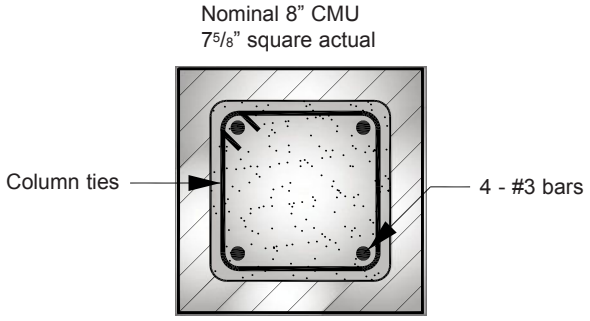


FIGURE 7.28 Minimum column size and reinforcement.

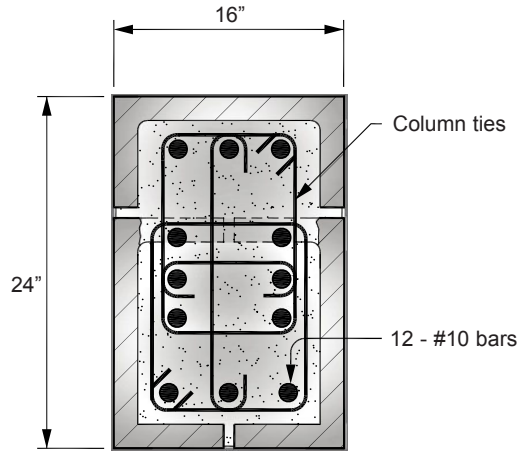


FIGURE 7.29 Maximum amount of steel in a 16 in. x 24 in. column.

7.13.2 PROJECTING WALL COLUMNS OR PILASTERS

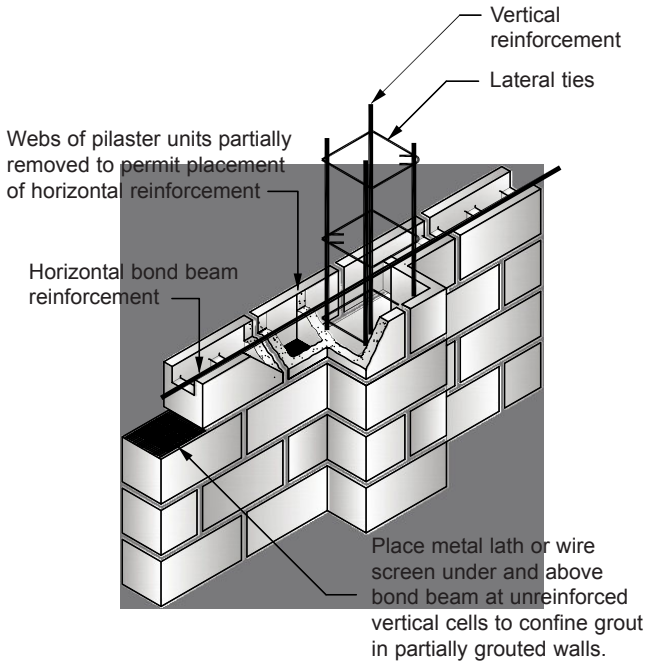


FIGURE 7.30 Construction of reinforced concrete masonry pilaster with continuous bond beams.

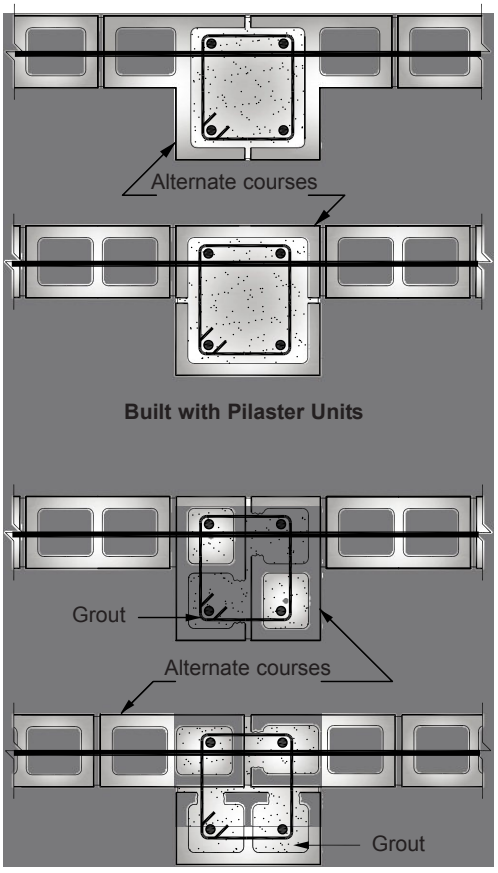


FIGURE 7.31 Projecting concrete masonry wall column details.

Heavily loaded girders which frame into a wall may require substantial base plates and columns. In order to provide a convenient girder seat and adequate column capacity, columns called pilasters are often built projecting out from the face of the wall.

Projecting pilasters also serve to stiffen the wall if they are adequately supported at the top and bottom. The wall between pilasters can then be designed to span horizontally allowing very high walls to be built using only nominal masonry thicknesses.

7.13.3 FLUSH WALL COLUMNS

If engineering design permits, an economical benefit may exist to the owner and the contractor to build columns that are contained in the wall and are flush with the wall. Wall-contained columns permit faster construction, since there are no projections from the wall and no special units are required. The reinforcing steel must be tied in accordance with the code requirements.

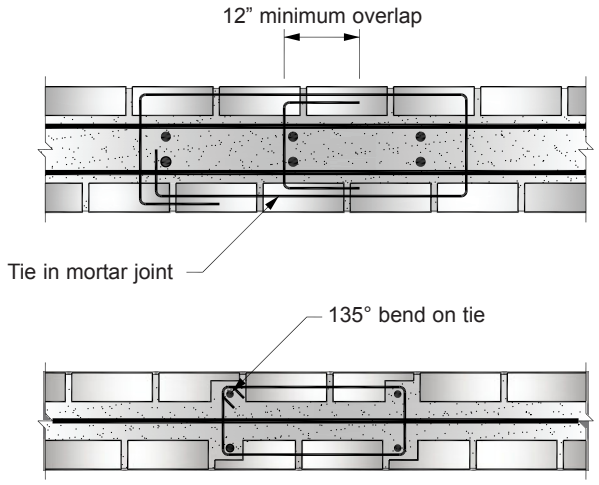


FIGURE 7.32 Flush wall brick columns with ties in mortar joint

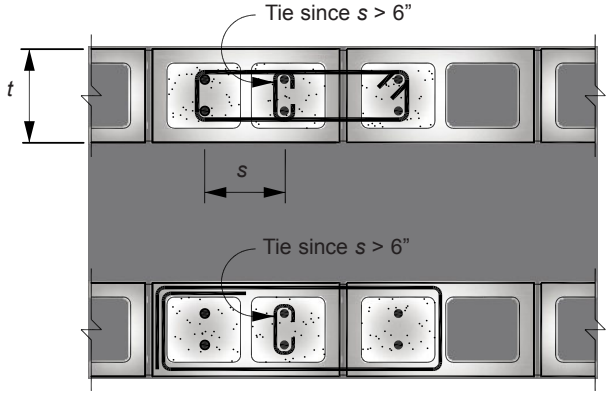


FIGURE 7.33 Flush wall columns in concrete masonry.

7.13.4 COLUMN TIE REQUIREMENTS

MSJC Code Section 2.1.6.5 covers the requirements for column ties based on longitudinal bar and tie diameters. Spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie diameters or the least dimension of the column. Ties shall be at least 1/4 in. in diameter.

All longitudinal bars for columns shall be enclosed by lateral ties. Lateral support shall be provided to the longitudinal bars by the corner of a complete tie having an included angle of not more than 135 degrees or by a standard hook at the end of a tie. The corner bars shall have such support provided by a complete tie enclosing the longitudinal bars. Alternate longitudinal bars shall have such lateral support provided by ties and no bar shall be farther than 6 in. from such laterally supported bar.

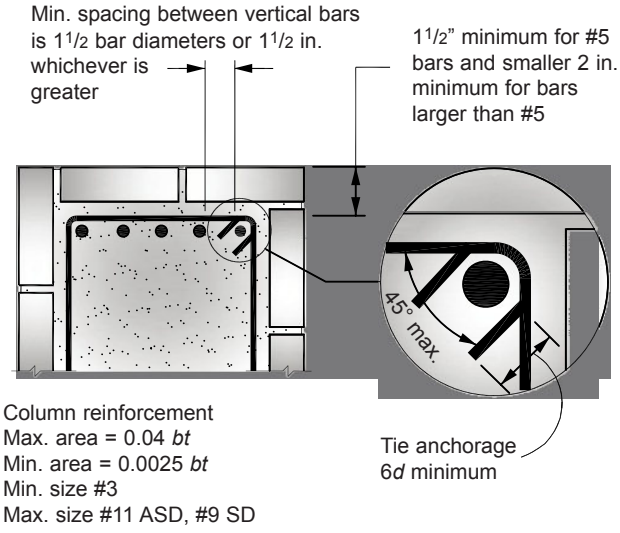


FIGURE 7.34 Reinforcing tie details.

TABLE 7.12 Maximum Tie Spacing Based on Longitudinal Bar Size[1]

Compression Steel Bar No.	Maximum Tie Spacing (in.)
#3	6
#4	8
#5	10
#6	12
#7	14
#8	16
#9	18
#10	20
#11	22

1. Based on MSJC Code Section 2.1.6.5. Maximum tie spacing may not exceed 16 longitudinal bar diameters, 48 tie diameters nor the least column dimension. Coordinate this Table with Table 7.13.

TABLE 7.13 Maximum Tie Spacing Based on Tie Size[1]

Tie Steel Size	Maximum Tie Spacing (in.)
1/4 in. (min)	12
#3	18
#4	24
#5	30

1. Based on MSJC Code Section 2.1.6.5. Maximum tie spacing may not exceed 16 longitudinal bar diameters, 48 tie diameters nor the least column dimension. Coordinate this Table with Table 7.12.

Note: #2 (1/4 in.) ties at 8 in. spacing is equivalent to #3 (3/8 in.) tie at 16 in. spacing.

7.13.5 LATERAL TIE SPACING FOR COLUMNS

7.13.5.1 LATERAL TIE SPACING IN SEISMIC DESIGN CATEGORIES A, B, AND C

There are no special tie spacing requirements for Seismic Design Categories A, B and C. Therefore, normal tie spacing of 16 bar diameters and 48 tie diameters, or least column dimension whichever is less applies. Additionally, MSJC Code Section 1.14.5.3.1 provides for two No. 4 lateral ties in the top 5 in. of the column in SDC C and above.

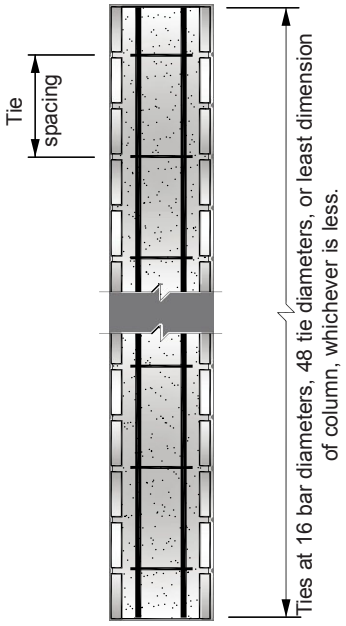


FIGURE 7.35 Maximum tie spacing in columns in Seismic Design Categories A, B, and C.

7.13.5.2 LATERAL TIE SPACING IN SEISMIC DESIGN CATEGORIES D, E, AND F

Specific lateral tie spacing requirements for columns located in Seismic Design Categories D and above are given in MSJC Code Section 1.14.6.5. Lateral tie spacing shall not exceed 8 in. on center and ties must be at least 3/8 inches in diameter. Figure 7.36 shows required lateral tie spacing.

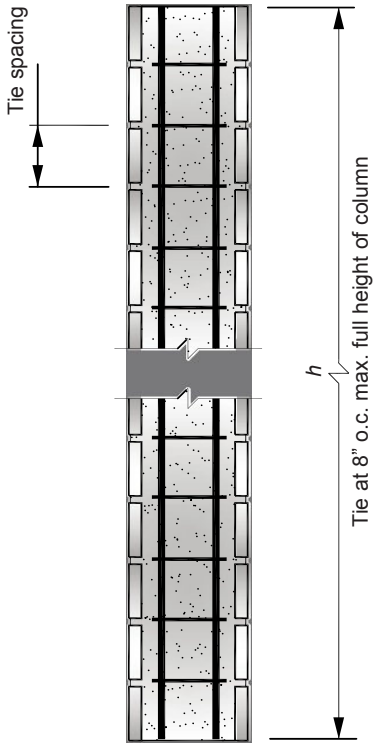


FIGURE 7.36 Maximum tie spacing in columns in Seismic Design Categories D, E, and F.

7.13.6 TIES AROUND ANCHOR BOLTS ON COLUMNS

Provide ties around anchor bolts which are set in the top of columns. Two ties should be placed within the top 5 in. of a column and confine vertical reinforcing bars and/or anchor bolts.

In SDC C and above, at least two #4 lateral ties are required within the top 5 in. of the column. Lateral ties must be designed and constructed to enclose both vertical bars and anchor bolts.

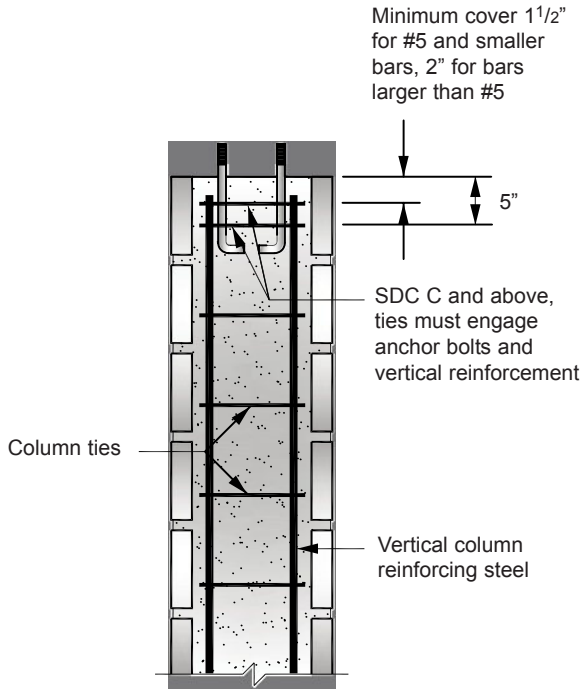


FIGURE 7.37 Ties at anchor bolts in the top of columns.

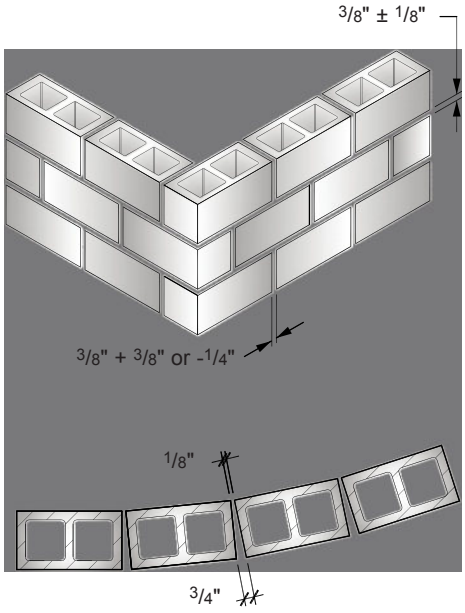
7.14 SITE TOLERANCES

Site tolerances for masonry construction are based on structural performance, not aesthetics. Masonry tolerances may be more restrictive than tolerances of other materials, therefore, verification of project conditions should be completed prior to masonry installation. MSJC Specification Article 3.3 G provides tolerances for masonry construction.

MSJC Specification Article 3.3 G

3.3 G. *Site tolerances* — Erect masonry within the following tolerances from the specified dimensions.

1. Dimension of elements
   a. In cross section or elevation
      .....-1/4 in. (6.4 mm), +1/2 in. (12.7 mm)
   b. Mortar joint thickness
      bed.....±1/8 in. (3.2 mm)
      head.....-1/4 in. (6.4 mm), + 3/8 in. (9.5 mm)
      collar.....-1/4 in. (6.4 mm), + 3/8 in. (9.5 mm)



(Plan View Radius Wall)

FIGURE 7.38 Permissible variations in mortar joint thickness.

MSJC Specification Article 3.3 G

3.3 G. *Site Tolerances* – Erect masonry within the following tolerances from the specified dimensions.

1. Dimension of elements
   c. Grout space or cavity width, except for masonry walls passing framed construction ……..…..-1/4 in. (6.4 mm), + 3/8 in. (9.5 mm)

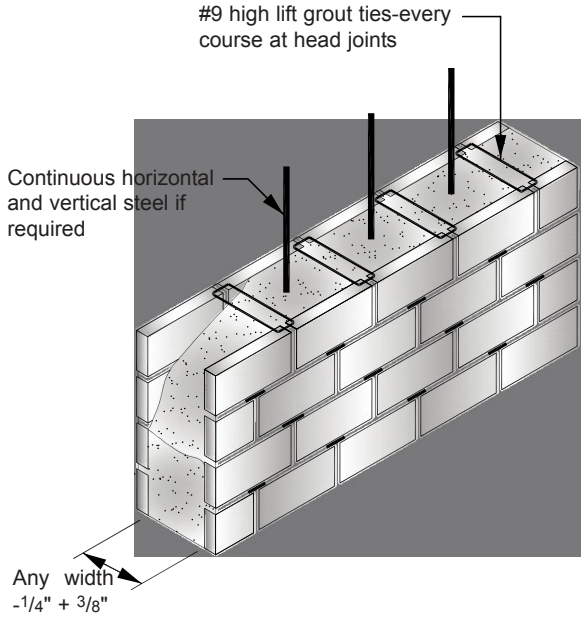


FIGURE 7.39 Permissible variation of grout space.

MSJC Specification Article 3.3 G

3.3 G. *Site Tolerances* – Erect masonry within the following tolerances from the specified dimensions.

2. Elements
   a. Variation from level:
      bed joints
      .....±1/4 in. (6.4 mm) in 10 ft. (3.05 m)
      .....±1/2 in. (12.7 mm) maximum
      top surface of bearing walls
      .....±1/4 in. (6.4 mm) in 10 ft. (3.05 m)
      .....±1/2 in. (12.7 mm) maximum

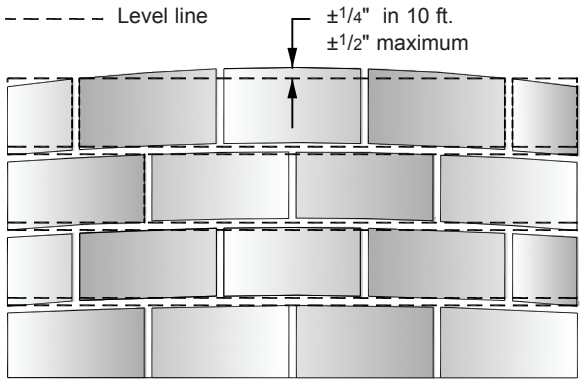


FIGURE 7.40 Permissible variation from level for bed joints.

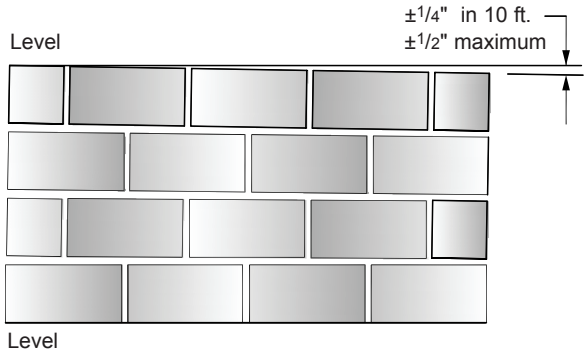


FIGURE 7.41 Permissible variation from level, top surface of bearing walls.

MSJC Specification Article 3.3 G

3.3 G. *Site Tolerances* – Erect masonry within the following tolerances from the specified dimensions.

2. Elements
   b. Variation from plumb
      .....±1/4 in. (6.4 mm) in 10 ft. (3.05 m)
      .....±3/8 in. (9.5 mm) in 20 ft. (6.10 m)
      .....±1/2 in. (13 mm) maximum

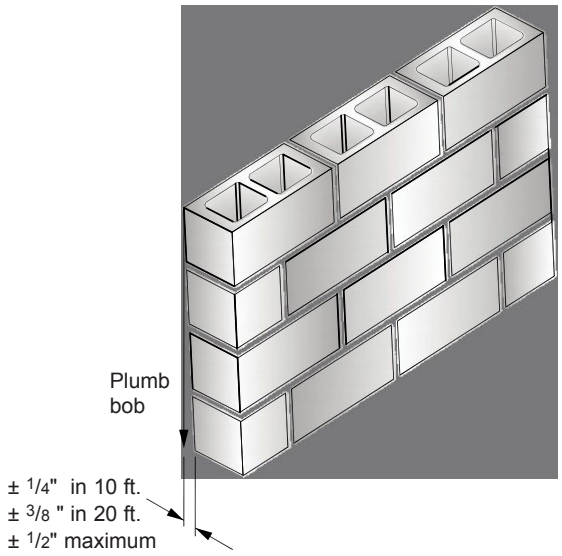


FIGURE 7.42 Permissible variation from plumb.

MSJC Specification Article 3.3 G

3.3 G. *Site Tolerances* – Erect masonry within the following tolerances from the specified dimensions.

2. Elements
   c. True to a line
      .....±1/4 in. (6.4 mm) in 10 ft. (3.05 m)
      .....±3/8 in. (9.5 mm) in 20 ft. (6.10 m)
      .....±1/2 in. (12.7 mm) maximum

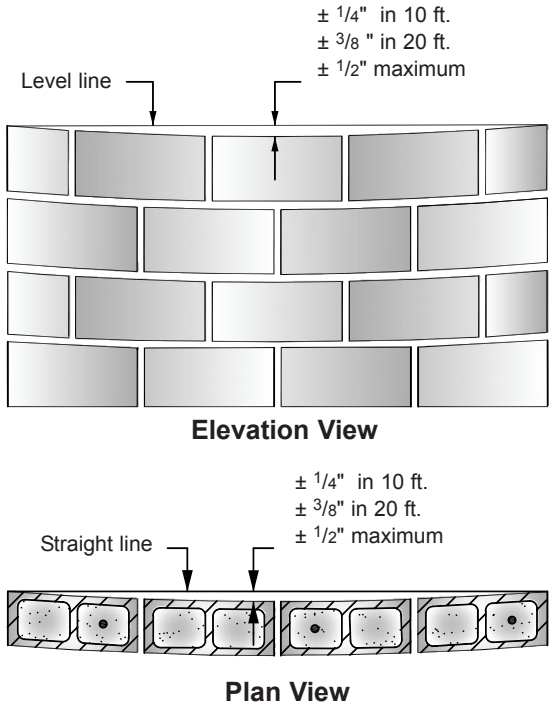


FIGURE 7.43 Permissible variation from true to line.

MSJC Specification Article 3.3 G

3.3 G. *Site Tolerances* – Erect masonry within the following tolerances from the specified dimensions.

2. Elements
   d. Alignment of columns and walls (bottom versus top)
      .....±1/2 in. (12.7 mm) for bearing walls
      .....±3/4 in. (19.1 mm) for nonbearing walls
3. Location of elements
   a. Indicated in plan
      .....±1/2 in. (12.7 mm) in 20 ft. (6.10 m)
      .....±3/4 in. (19.1 mm) maximum

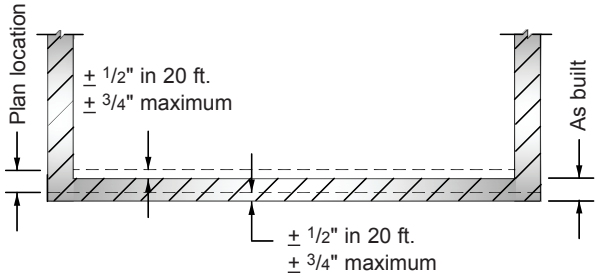


FIGURE 7.44 Permissible variation of element indicated in the plan.

MSJC Specification Article 3.3 G

3.3 G. *Site Tolerances* – Erect masonry within the following tolerances from the specified dimensions.

3. Location of elements
   b. Indicated in elevation
      .....±1/4 in. (6.4 mm) in story height
      .....±3/4 in. (19.1 mm) maximum

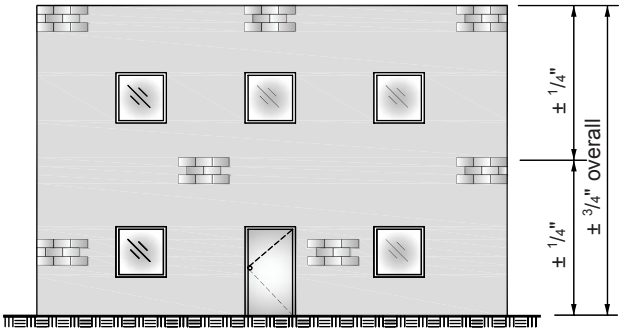


FIGURE 7.45 Permissible variation of element indicated in elevation.

MSJC Specification Article 3.3 G

3.3 G. *Site Tolerances* – Erect masonry within the following tolerances from the specified dimensions.

4. If the above conditions cannot be met due to previous construction, notify the Architect/ Engineer.

7.15 QUESTIONS AND PROBLEMS

7-1 What are the minimum reinforcing steel requirements for Seismic Design Categories A, B, and C?

7-2 What are two reasons to provide steel around openings in a wall?

7-3 Under what conditions or uses is joint reinforcement more desirable than deformed reinforcing bars?

7-4 Detail the reinforcing steel required for a two-story building located in Seismic Design Category C. Show the reinforcement at the corners, floor, roof, and around openings.

7-5 What is the minimum amount of reinforcing steel required for walls in Seismic Design Category D? If the vertical steel is in the center of a 9 in. brick wall and the steel ratio, ρ, = 0.004, how much steel must be used horizontally. Specify an appropriate size and spacing of reinforcing bars. If ρ = 0.002, what is the size, spacing and steel ratio of the horizontal steel?

7-6 A 10 in. solid grouted masonry wall has #6 vertical bars spaced at 18 in. o.c. How much horizontal steel must be placed to comply with the minimum code requirements for Seismic Design Category D?

7-7 Determine the minimum steel required for a 10 in. brick wall, 18 ft high located in Seismic Design Category C. The parapet extends 30 in. above the roof line. Use joint reinforcement between the footing and bond beam. Assume two #4 bars are used in the bond beam and at the top of the footing. Also assume the wall spans vertically. Use minimum steel requirements without structural calculations.

CHAPTER 8

BUILDING DETAILS

8.1 GENERAL CONNECTIONS

Connections are a critical part of any structure, particularly when the structure is subjected to seismic forces. When connections hold together and make the structure perform as a total system there is an excellent chance for the structure to survive even great earthquakes.

All connections must be satisfactory to transmit the forces due to lateral and vertical loads. The elements must be sufficiently tied together to cause them to act as a unit.

This section shows some of the more typical wall connections and building details.

8.2 WALL TO WALL CONNECTIONS

A significant issue for masonry constructed in higher Seismic Design Categories is positive connection of the elements. Adequate connections provide a continuous load path so that the forces can be reconciled. Details of structural reinforcing bar size and spacing are dependent on engineering requirements. Figures 8.1 through 8.4 give typical layout of providing continuous reinforcement at CMU wall intersections.

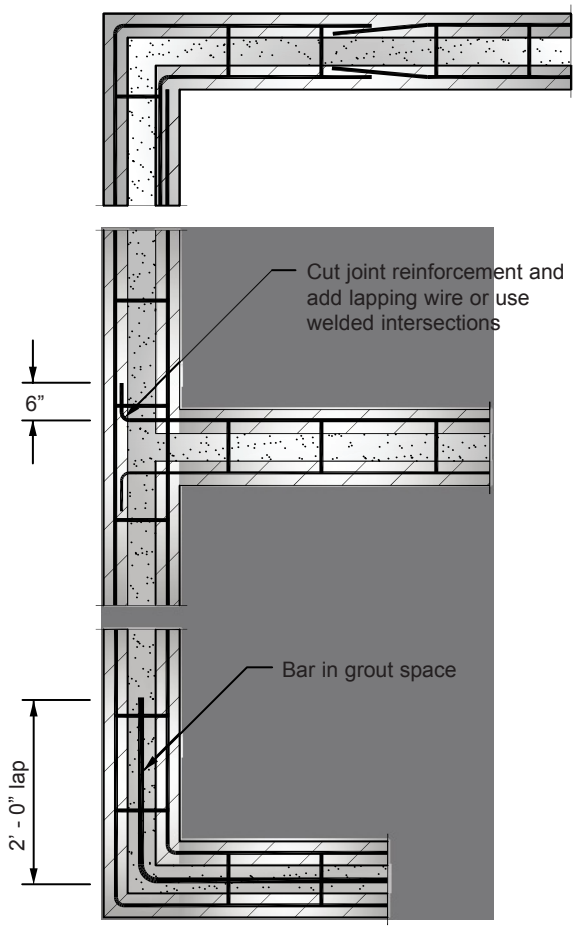


FIGURE 8.1 Plan of joint reinforcement for intersecting walls.

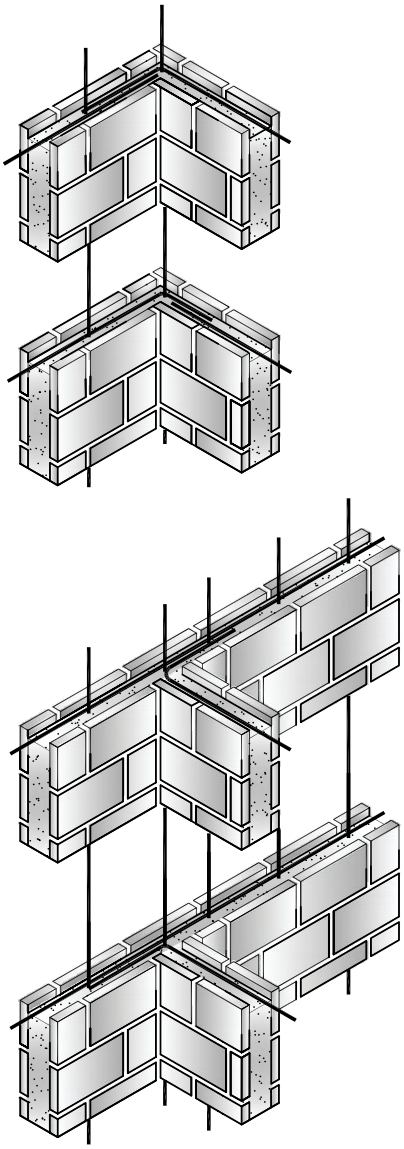


FIGURE 8.2 Exploded isometric view of reinforcing steel for intersecting walls.

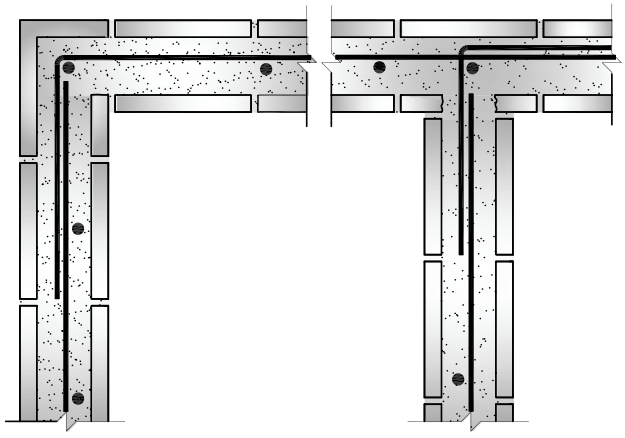


FIGURE 8.3 Typical wall connections. Plan view of horizontal reinforcement for intersecting walls.

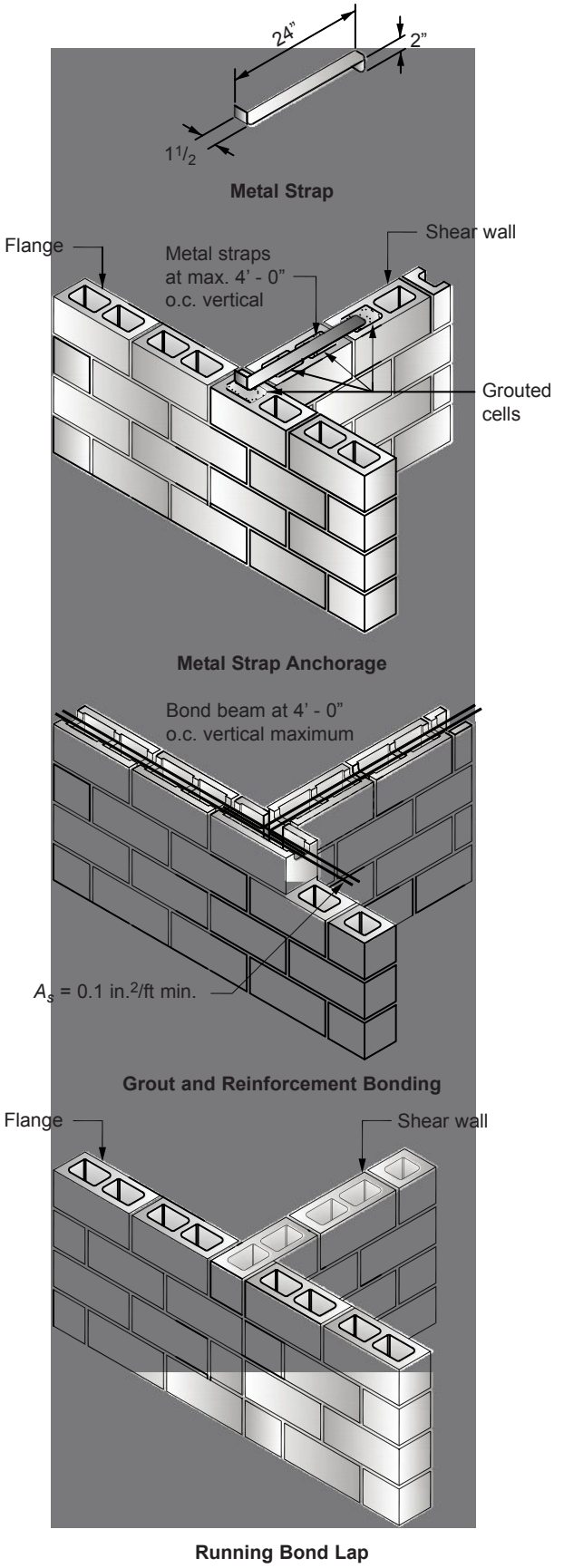


FIGURE 8.4 Anchorage, reinforcement bonding and bond lap at wall intersection.

8.3 LINTEL AND BOND BEAM CONNECTION

A lintel is a beam that spans over an opening; typically a window or doorway. Reinforced CMU is an easy and cost effective way to create lintels. One of the key components in detailing a lintel is to extend the lintel reinforcement past the edge of the opening and into the wall. The design professional will determine the exact distance of the reinforcement extension past the opening edge.

FIGURE 8.5 Masonry beam spanning an opening.

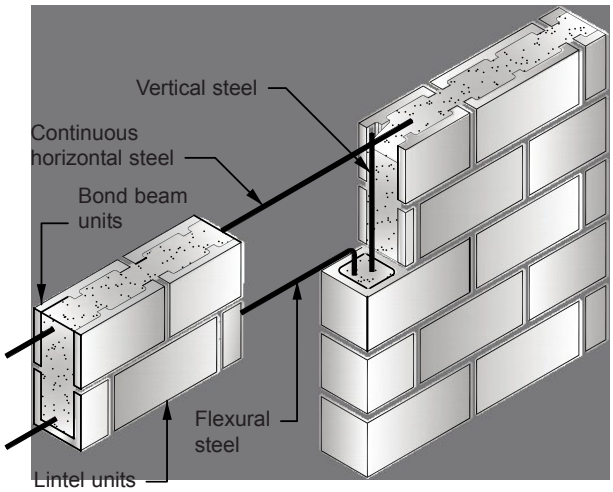
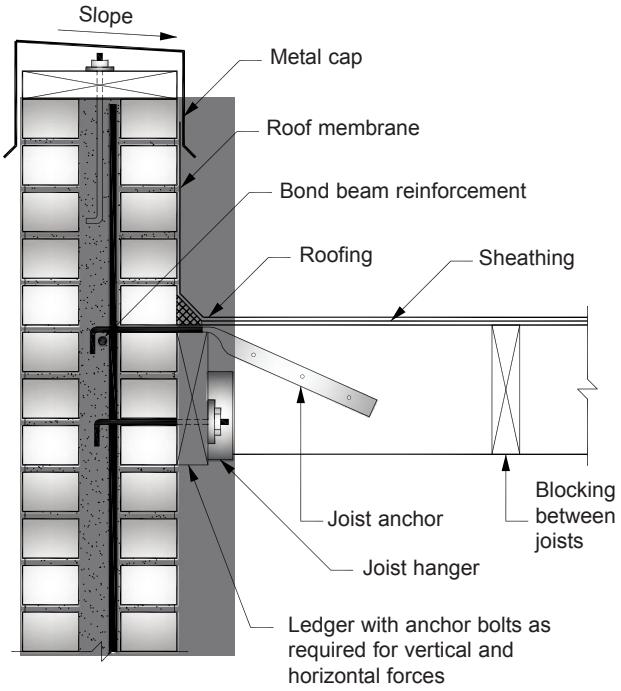
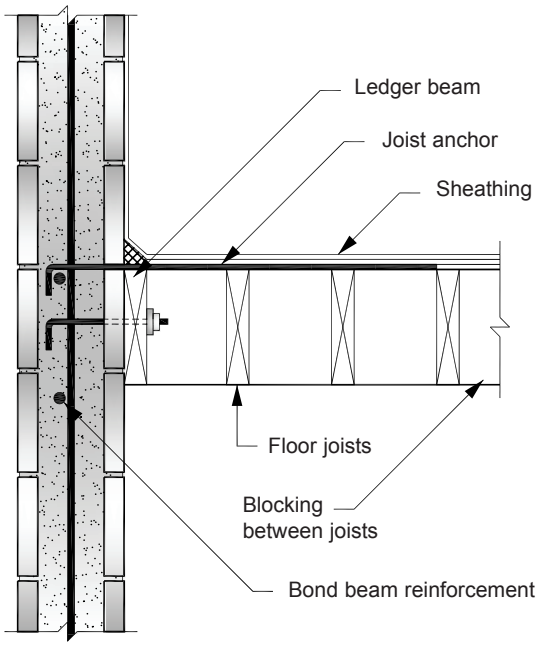


FIGURE 8.6 Lintel and bond beam detail.

8.4 WALL TO WOOD DIAPHRAGM CONNECTIONS



(a) Joist perpendicular to wall



(b) Joist parallel to wall

FIGURE 8.7 Connection details of wood joists to masonry walls.

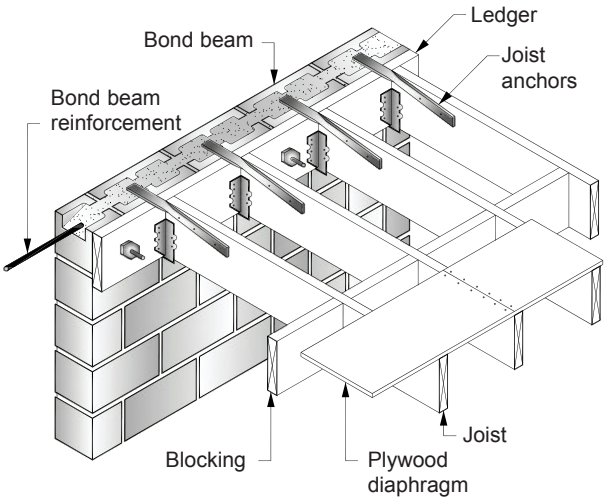


FIGURE 8.8 Isometric view of connection of wood diaphragm to masonry wall.

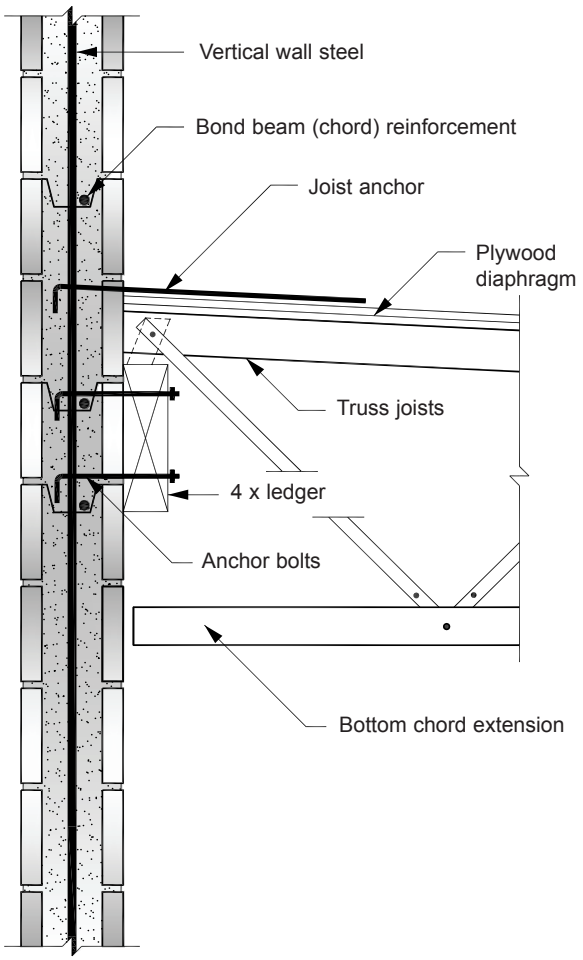


FIGURE 8.9 Connection of wood truss joist to masonry wall.

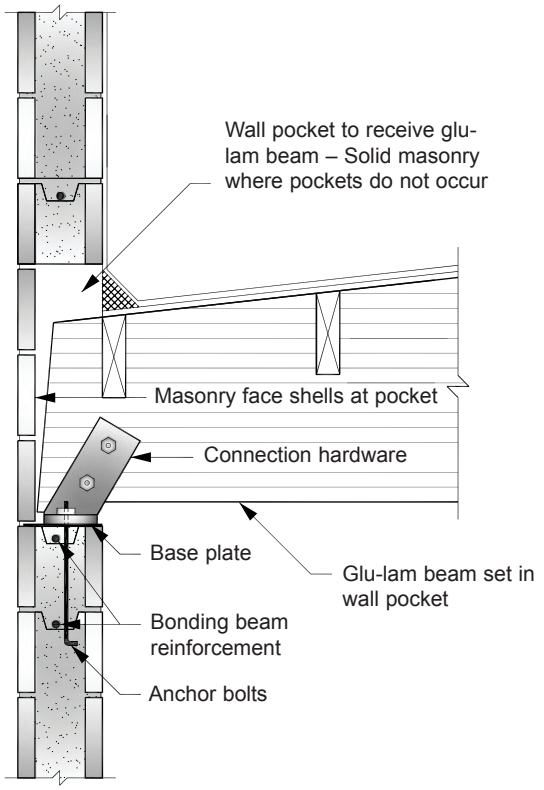


FIGURE 8.10 Connection of glu-lam beam to masonry wall.

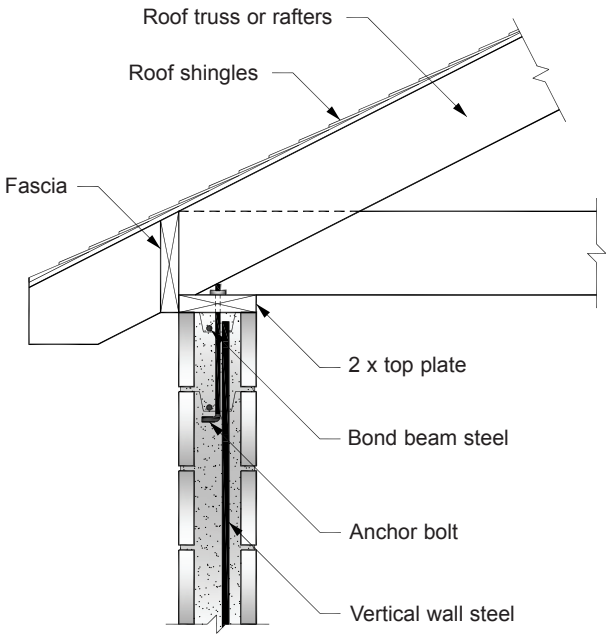


FIGURE 8.11 Connection of roof rafters or truss to masonry wall.

8.5 WALL TO CONCRETE DIAPHRAGM CONNECTIONS

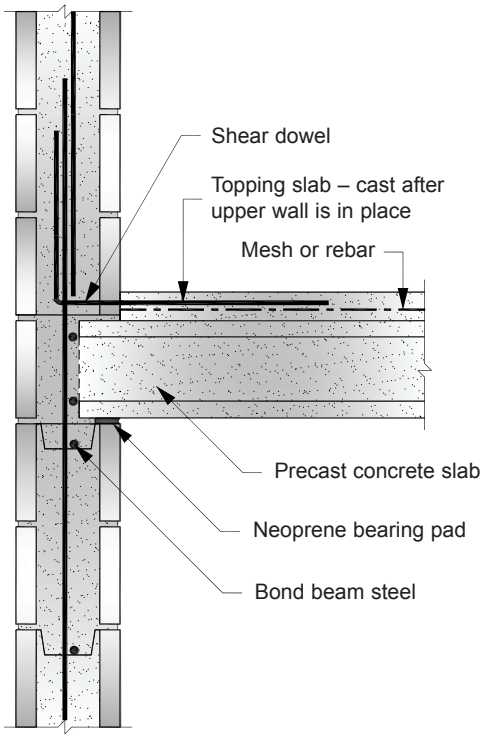


FIGURE 8.12 **Precast slab bearing on masonry wall.**

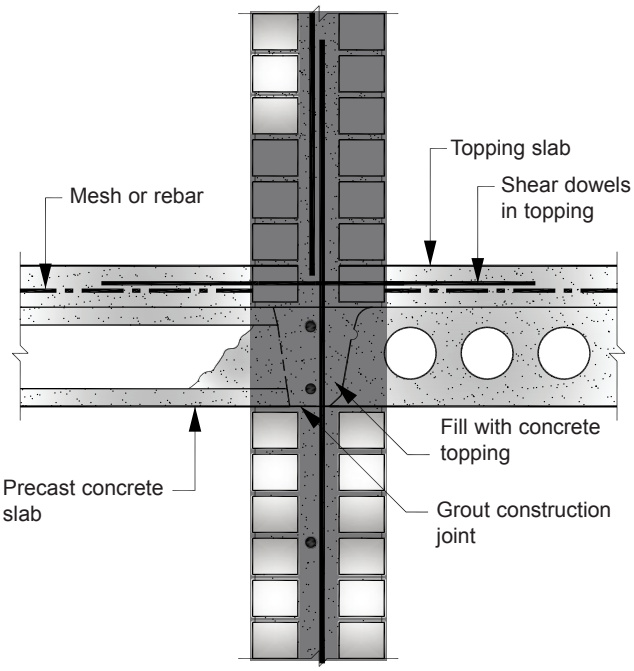


FIGURE 8.13 **Precast, prestressed hollow core slabs with concrete topping on masonry wall.**

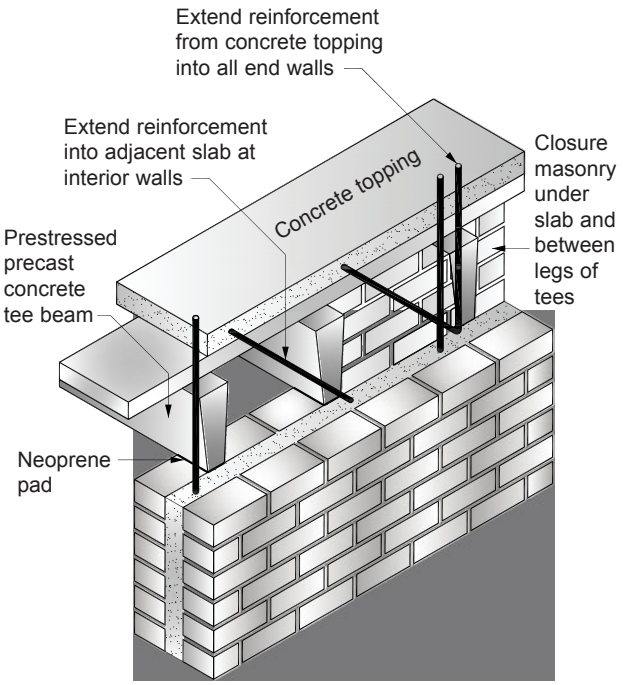


FIGURE 8.14 **Precast tee beam bearing on masonry wall.**

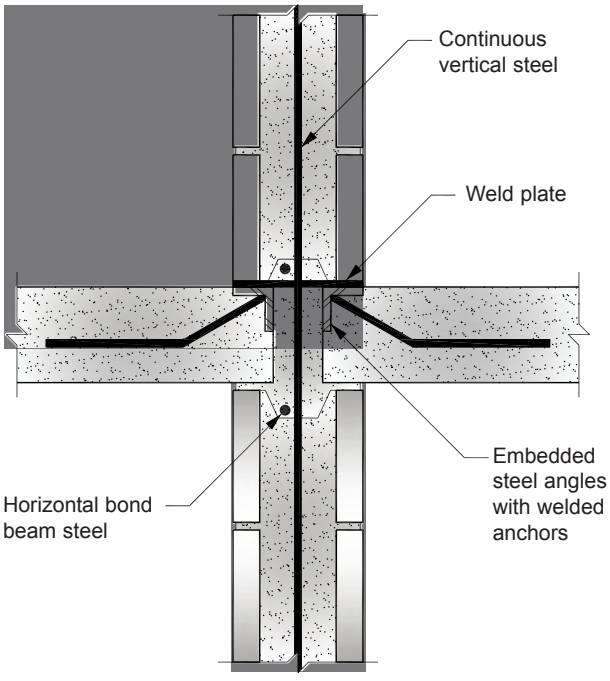


FIGURE 8.15 **Precast concrete slabs connected to masonry wall with welded connections.**

8.6 WALL TO STEEL DIAPHRAGM CONNECTIONS

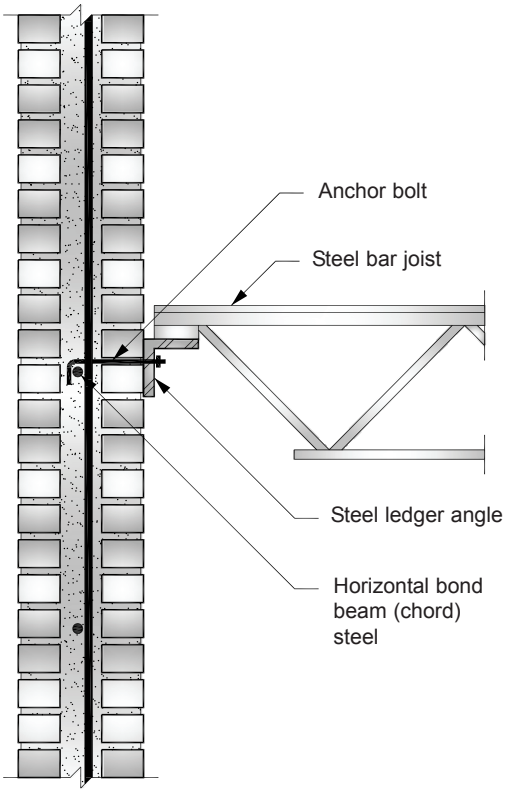


FIGURE 8.16 Steel bar joists floor or roof system connected to masonry wall with a ledger angle.

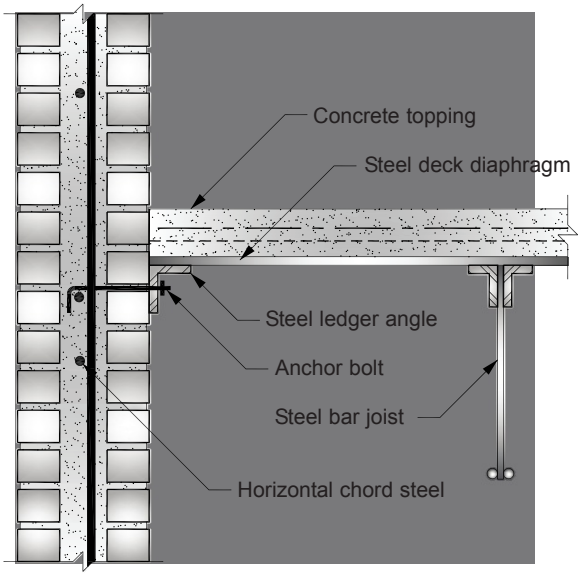


FIGURE 8.17 Steel bar joist and roof deck connection with bar parallel to wall.

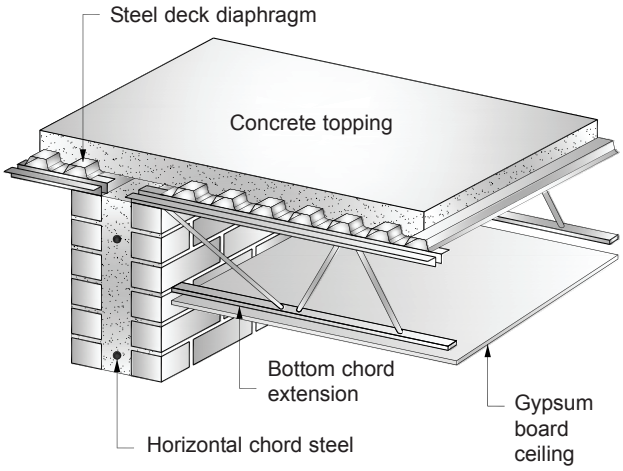


FIGURE 8.18 Isometric view of connection of steel bar joist floor system to masonry wall.

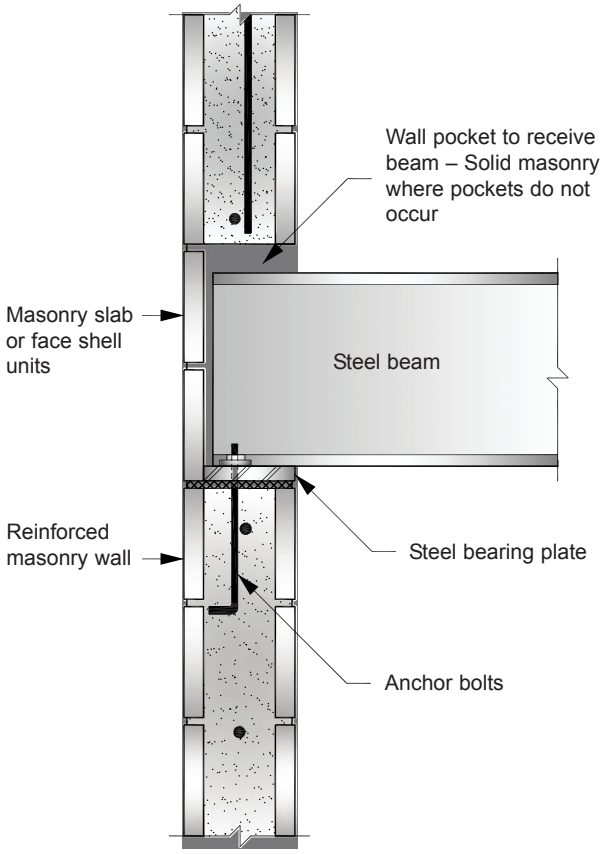


FIGURE 8.19 Steel beam bearing on masonry wall.

8.7 WALL FOUNDATION DETAILS

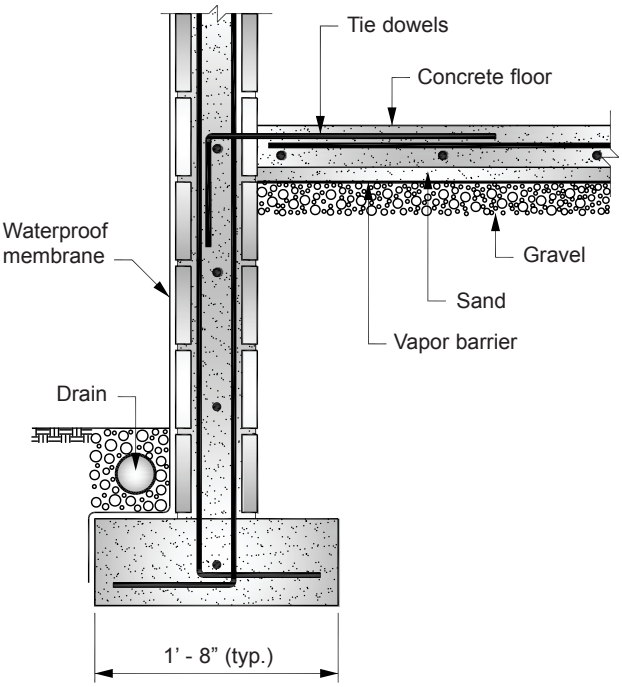


FIGURE 8.20 **Exterior bearing wall with earth backfill.**

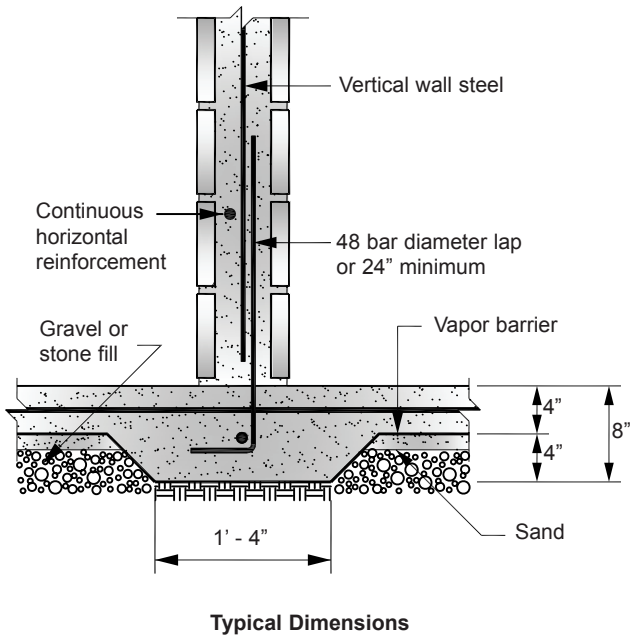


FIGURE 8.22 **Typical interior, non-loading bearing wall and footing detail.**

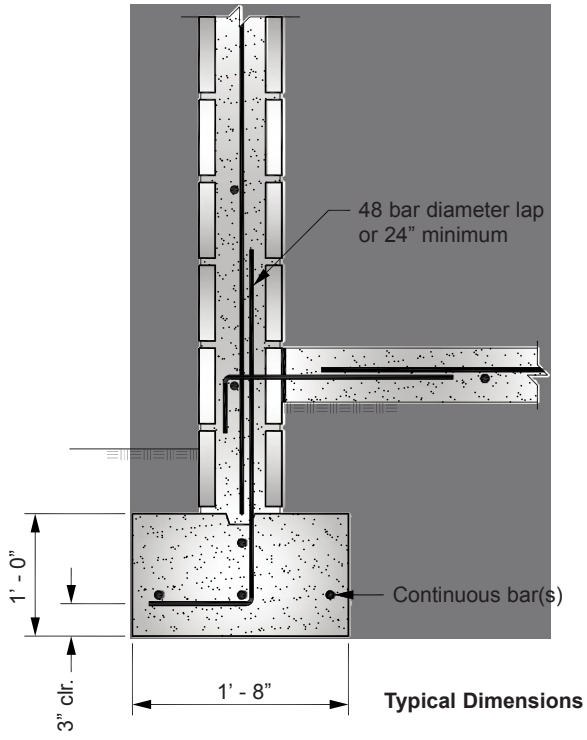


FIGURE 8.21 **Typical footing detail for exterior masonry wall.**

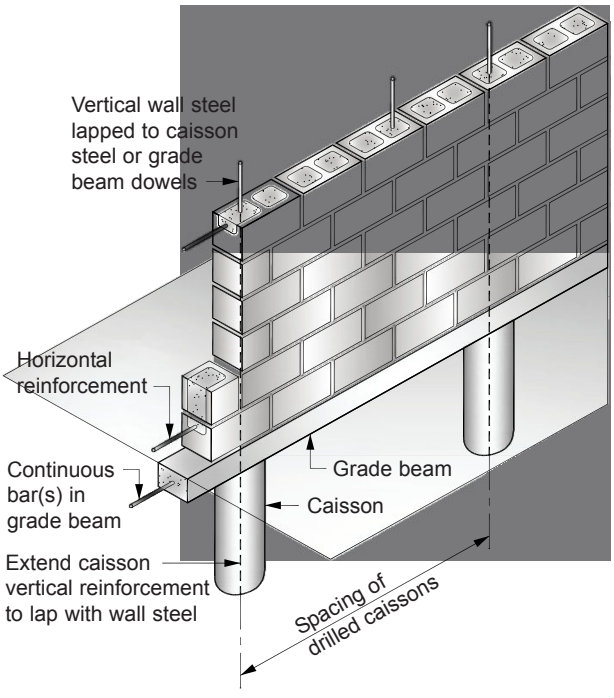


FIGURE 8.23 **Grade beam and caisson system for supporting masonry wall.**

CHAPTER 9

SPECIAL TOPICS

9.1 MOVEMENT JOINTS

9.1.1 GENERAL

Movement joints are provided in reinforced and unreinforced masonry walls to properly accommodate moisture and temperature fluctuation. Shrinkage and temperature hairline cracks can develop allowing water passage into masonry. As a general rule, control joints should be placed in concrete masonry every 25 ft but not more than 1 1/2 times the wall height, and expansion joints every 15 to 20 ft in clay masonry.

All structural elements move when subjected to changes in moisture, temperature and loads. The movements can cause damage or cracks especially when no provisions are made to allow the structure to move.

The type, location and spacing of movement joints depends on masonry materials, climatic conditions, size and type of structure, local factors and experience.

Movement joints should be located where they will least impair the strength of the finished structure, where they will not adversely affect the architectural design, and where they can facilitate the construction of the walls. They should never be located by chance or convenience without regard for effect on the strength or appearance of the completed structure.

Movement joints in a wall, whether control joints or expansion joints, should match any joints built into the roof system, the floor system, the spandrel beams or other elements intended to accommodate the overall movement of the building.

9.1.2 MOVEMENT JOINTS FOR CLAY MASONRY STRUCTURES

9.1.2.1 GENERAL

Clay masonry units are normally smallest in overall size just after firing. As they gain moisture they may expand 0.02% for each percent of moisture increase. Thus, a 100 foot long brick wall may increase in length approximately 1/4 in. for each percent of moisture increase. If the wall is restrained from movement, compressive stresses will develop, often high enough to crush the brick or push an adjoining wall out of plumb when expansion joints are absent from the system.

Clay masonry units also expand approximately 0.036% per 100°F temperature increase.

9.1.2.2 VERTICAL EXPANSION JOINTS

Expansion joints are used to accommodate increases in length and height of a masonry wall due to thermal expansion or swelling of the clay masonry from moisture increase. The need for expansion joints depends upon:

1. The climatic area in which the structure is located.
2. Dimensions and configuration of the building.
3. Temperature change and provisions for temperature control.
4. Type of structural frame, connection to the foundation, and symmetry of stiffness against lateral displacement.
5. Materials of construction.

The Brick Industry Association's (BIA) Technical Notes 18 estimates unrestrained expansion of clay masonry as:

$$m_u = (k_e + k_f + k_t \Delta T)L$$

Where:

m_u = total unrestrained movement of the brickwork, in. (mm)

k_e = coefficient of moisture expansion, in./in. (mm/mm)

k_f = coefficient of freezing expansion, in./in./°F (mm/mm/°C)

k_t = coefficient of thermal expansion, in./in./°F (mm/mm/°C)

ΔT = temperature range experienced by brickwork, °F (°C)

L = length of wall, in. (mm)

Note, however, that BIA recognizes the total amount of expansion as somewhat less due to indeterminate factors such as restraint, shrinkage and plastic flow of mortar, age of masonry and variations in workmanship.

9.1.2.3 LOCATION AND SPACING OF EXPANSION JOINTS

There are no suggestions on the positioning and spacing of expansion joints that can be applicable to all structures. Every building should be analyzed to determine the potential movements and provisions should be made to relieve excessive stress which might be expected from such movement. Typical recommended spacing of expansion joints is 15 ft to 20 ft apart. Spacing of expansion joints in a solid wall without openings should not exceed 25 ft.

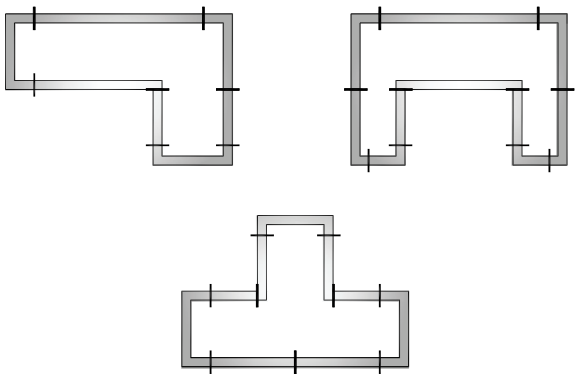


FIGURE 9.1 Typical location of expansion joints in irregular shaped buildings.

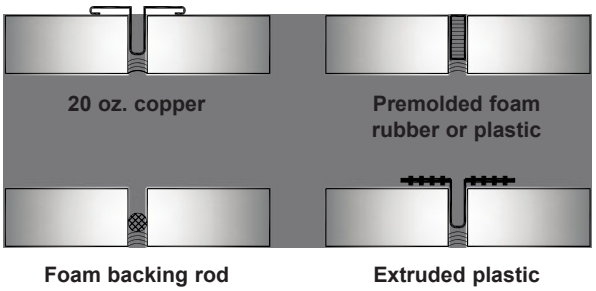


FIGURE 9.2 Details of vertical expansion joints.

The majority of expansion joint materials are typically 25% to 50% compressible. Size of the expansion joints will depend on joint spacing and the performance of the sealant. The actual joint dimension must be twice the anticipated movement if the expansion material can only be compressed 50%.

The geometry of a structure affects the placement of vertical expansion joints. Several typical locations of vertical expansion joints include spacing at 25 ft in long runs of walls, at or near offsets and at intersecting walls and corners.

Typically, expansion joints can be placed on a symmetrical basis with respect to openings and elevations, at the jambs of openings or at building grids such as column lines. Toothed expansion joints, joints that follow natural vertical and horizontal mortar lines, are difficult to install and may not permit proper functioning of the sealant.

9.1.2.4 HORIZONTAL EXPANSION JOINTS

Horizontal expansion joints or soft joints, (Figure 9.3), are limited to brick veneer and are especially important on high-rise buildings. These joints allow vertical shortening of the building frame, vertical deflection of the supporting members and expansion of the brick veneer. Their absence can create severe problems in both reinforced concrete frame buildings and in masonry buildings with exterior clay-brick wythes. The combined effect of drying shrinkage, creep and plastic flow in a structural frame reduces the building's floor-to-floor height. Any expansion of the clay brick veneer adds to the problem. Without horizontal soft joints between the bottom of a shelf angle and the top course of the masonry panel below, cracking or crushing is likely to occur.

The location of horizontal expansion joints must be directly under intermediate supports, such as shelf

angles, which are attached to the main structure. The shelf angles are a natural interruption of masonry and thus a logical place for an expansion joint. Movement between the veneer and the structural frame, including seismic and wind, occurs at the shelf angles. Vertical growth of the masonry beneath the shelf angle is permitted by a horizontal expansion joint.

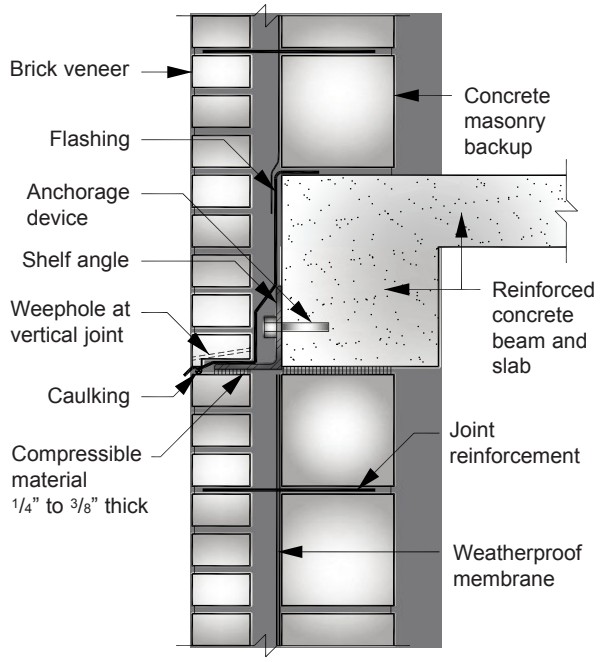


FIGURE 9.3 Horizontal expansion joint to allow movement of the wall.

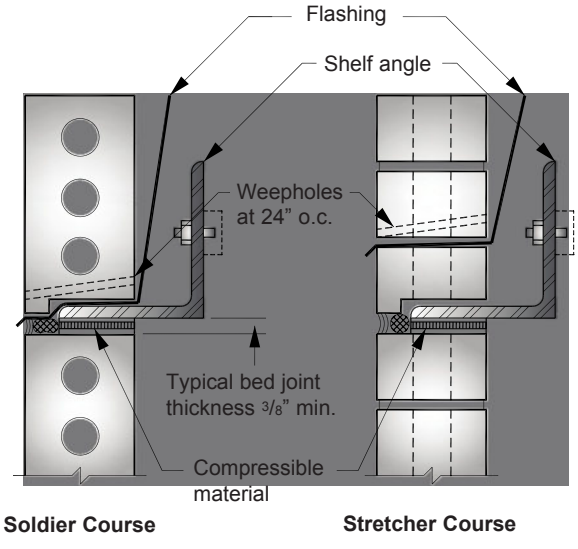


FIGURE 9.4 Manufacture or cut units to reduce height of exposed movement joint at support angle.

To properly account for vertical movements, a compressible material should be provided at the top of the panel just below the supporting shelf angle (Figures 9.3 and 9.4).

9.1.3 MOVEMENT JOINTS IN CONCRETE MASONRY STRUCTURES

Concrete masonry units are subjected to significant shrinkage due to moisture variations and/or temperature decrease. Of particular concern is the drying shrinkage of concrete masonry during the early curing and drying of masonry walls which introduces tension stresses into the masonry units. During the construction of concrete masonry walls, wet fluid grout adds moisture to the masonry wall, causing it to expand. As the grout hydrates and hardens and as the masonry units dry out, walls try to shrink. Since the face shells lose moisture rapidly in a dry climate, they shrink quickly, thus putting them into tension while the interior of the wall may be subjected to compression (Figure 9.5). Cracking of concrete masonry units can occur if these tensile stresses exceed the tensile strength of the materials.

Several major factors affect the volume change of concrete masonry generated by moisture fluctuations including the type of aggregate and the curing method. Generally, concrete masonry units made with normal weight sand and gravel aggregate exhibit less volume change than those made with lightweight aggregate or cinders. Similarly, units cured by steam pressure or autoclaving show a significant decrease in volume change characteristics compared to masonry cured by air. Coefficients for the moisture related volume change of concrete masonry units vary from about 0.01% to 0.1%.

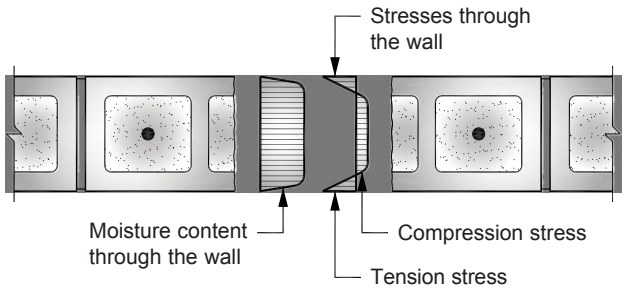


FIGURE 9.5 Moisture content and shrinkage stresses in a concrete masonry wall.

9.1.3.1 CRACK CONTROL FOR CONCRETE MASONRY

Methods used to control cracking in concrete masonry structures include:

1. Specifying materials which limit the drying-shrinkage potential.
2. Reinforcing masonry to resist tension stresses and thus increase crack resistance.
3. Providing control joints to accommodate the expected movement.

9.1.3.2 CONTROL JOINTS IN CONCRETE MASONRY WALLS

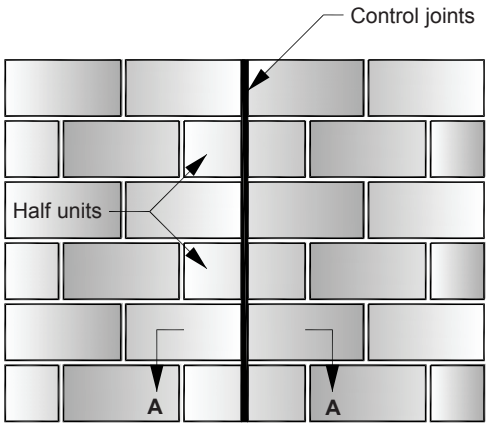
Shrinkage control joints panelize a wall, allowing shrinkage to take place within a small, relatively unrestrained panel. Since the control joints allow the panel to shorten in length, shrinkage cracks are less likely to occur (see Figure 9.6).

These control joints are basically weakened head joints which extend vertically straight up and down the wall through the use of full and half masonry units. Mortar at the control joints is either omitted entirely or deeply raked back. Joint reinforcement is generally terminated at the control joint, although chord reinforcing steel at floors and roofs must continue through the control joints. Likewise, any horizontal reinforcement required for structural considerations, such as lintel reinforcement, should continue through the joints.

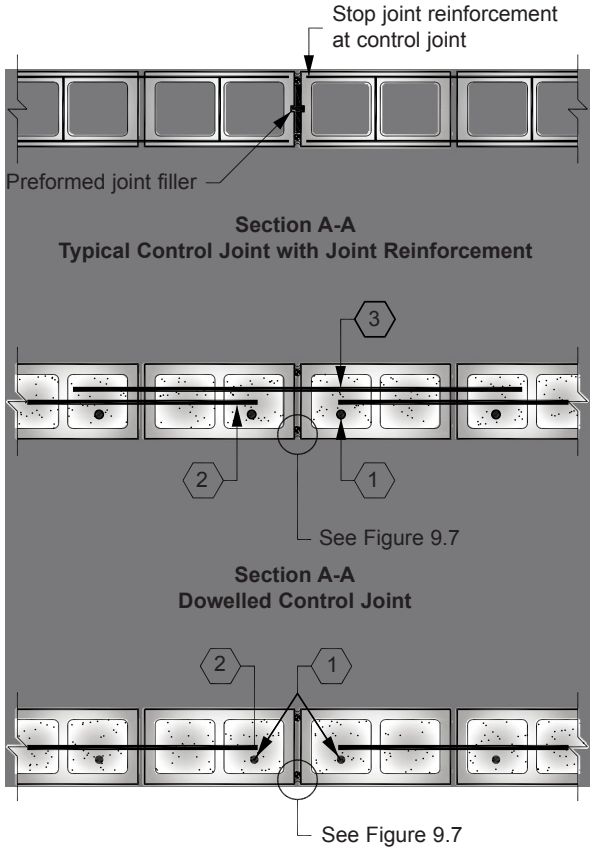
9.1.3.3 SPACING OF VERTICAL CONTROL JOINTS

Vertical control joints are usually spaced at close intervals so that when shortening takes place, the resulting crack is more likely to occur at the joint. It is important to provide sufficient control joints so that movement occurs at the joint rather than midway between control joints.

The recommended maximum horizontal spacing of control joints in concrete masonry walls should be approximately 25 ft, but a length (spacing) to height of wall ratio is a better measure. See Table 9.1 for the maximum recommended control joint spacing of horizontally reinforced walls.



Wall Elevation



Section A-A
Typical Control Joint

1. Additional vertical bars on each side of all control joints.
2. Terminate all non-structural reinforcement 2 in. from control joints. Where structural reinforcement must continue through control joint, mortar may be raked back to provide joint to accommodate shrinkage cracking.
3. Provide 4 ft - 0 in. long smooth dowels across the joint as required to keep walls relatively in-plane. Prevent bond between bar and grout with grease or a plastic sleeve. Cap all dowels to allow 1 in. of movement.

FIGURE 9.6 Typical control joint detail in concrete masonry walls.

TABLE 9.1 Recommended Control Joint Spacing for Above Grade Exposed Concrete Masonry Walls[1]

Distance between joints should not exceed the lesser of:	
Length to height ratio	Or ft (m)
1 1/2	25 (7.62)

1. Based on NCMA TEK Note 10-2B, Table 1.

9.1.3.4 VERTICAL EXPANSION JOINTS IN CONCRETE MASONRY WALLS

Generally, concrete masonry walls less than 25 ft long do not require expansion joints if adequate control joints have been provided. However, if a concrete masonry structure is of unusual size or length or if it is subjected to severe conditions, expansion joints may be advisable.

Additionally, the need for thermal expansion joints in long buildings should be determined based on local practice.

9.1.4 CAULKING DETAILS

Control joints should be constructed as continuous vertical head joints, by using full and half masonry units, and by raking back the mortar at least one inch deep. The raked vertical head joint should then be caulked to keep it weatherproof. A backer rod should be provided in the joint to limit the depth of the caulking and to limit the adhesion of the caulk to the ends of the block.

A vertical expansion joint may be caulked in the same manner, however, the area behind the caulking and backer rod must contain compressible material or be completely void of material.

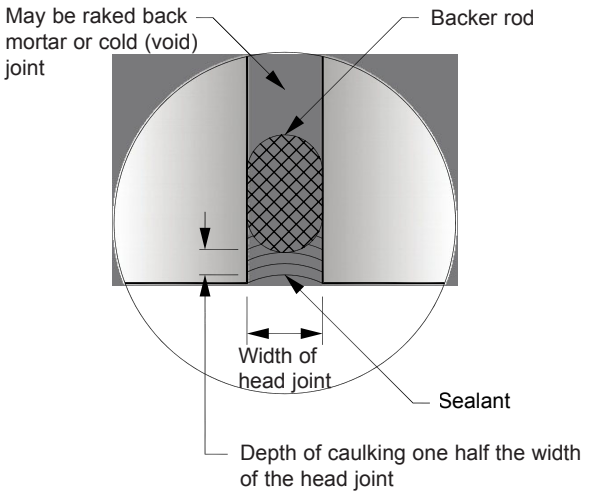


FIGURE 9.7 Caulking detail.

9.2 WATERPROOFING MASONRY STRUCTURES

9.2.1 GENERAL

Masonry materials are relatively porous and may absorb water under certain conditions. Since water infiltration can deteriorate the masonry as well as damage a building's appearance and interior finishes, every effort should be taken to repel water infiltration.

Normally masonry structures are protected from water damage by one of two methods. The first method consists of constructing the walls with an exterior masonry veneer separated from the structural back-up by an air space. Any water which penetrates the veneer runs down the back side of the veneer since it cannot cross the air space. Flashing and weepholes at the base of the cavity direct the water back out the wall, thus keeping the interior of the building dry. This veneer system is quite effective and has been used extensively in the past. BIA *Technical Notes*, Series 28 as well as other publications provide excellent design and detailing procedures for this type of wall.

The second method to limit water damage is to repel water infiltration through proper design techniques, material selection, construction methods, surface treatments and maintenance. Reinforced and unreinforced single wythe masonry walls must be waterproofed in this fashion. The remainder of this section will provide general guidelines to effectively waterproof these types of walls.

9.2.2 DESIGN CONSIDERATIONS

Thoughtful design and careful detailing of a masonry building can significantly reduce potential leaks. Special attention should be given to vulnerable areas as described in this section.

9.2.2.1 MORTAR JOINTS

Certain types of mortar joints, such as concave and V type joints are much more weather resistant than others, as indicated in Figure 1.14. Well-tooled joints compact the mortar, filling voids and cracks which could lead to water migration.

9.2.2.2 PARAPETS AND FIRE WALLS

Exposed on both faces, parapets and firewalls are subjected to high wind forces, extreme rain and snow, and severe temperature fluctuations. Providing

a well-constructed wall cap and a positive membrane waterproofing on the roof side of these walls can effectively eliminate water penetration.

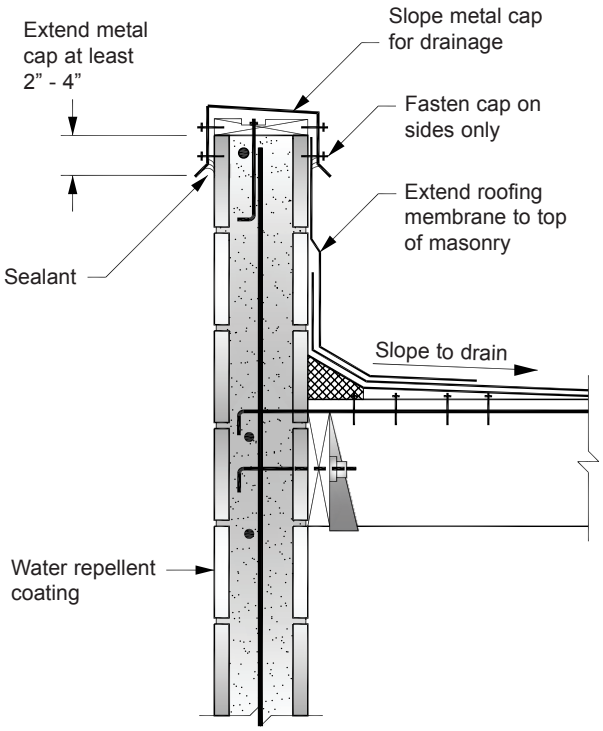


FIGURE 9.8 Parapet wall detail.

9.2.2.3 MOVEMENT JOINTS

Generally, too few movement joints are provided in masonry structures to properly accommodate moisture and temperature fluctuations. Shrinkage and temperature hairline cracks which can develop without these joints may allow water passage through the masonry. Additionally, leakage can occur at movement joints, and through cracked, unbonded or misapplied caulks and sealants.

9.2.2.4 HORIZONTAL SURFACES – PROJECTIONS, LEDGES AND SILLS

Horizontal surfaces contribute to the possibility of water penetration. Ledges and sills are particularly vulnerable as water may penetrate the top of the mortar joints, causing cracking and spalling. If possible, slope all projections, ledges and sills or provide a sloped flashing above them.

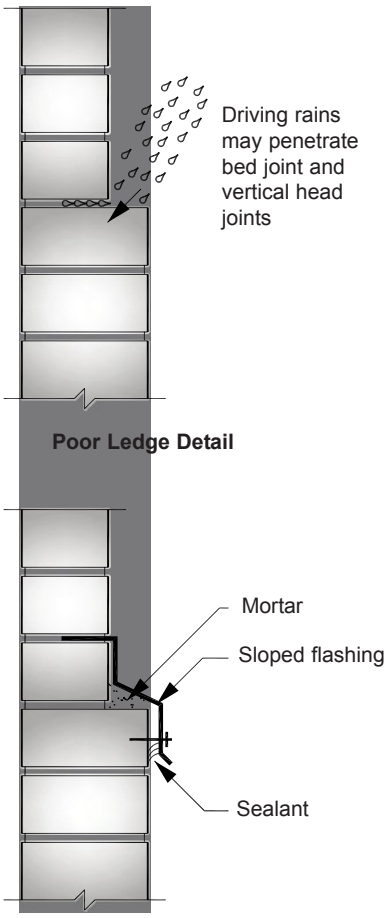


FIGURE 9.9 Ledge detail.

9.2.2.5 COPINGS AND WALL CAPS

Adequate slope should be provided on the top of all copings and wall caps so that water is shed quickly. Masonry and precast copings should extend past the face of the wall to reduce water penetration through the joint between the coping and the wall. Additionally, all overhangs should have drip edges to prevent water migration along the bottom of the wall cap and then down the wall (Figure 9.10).

Mortar caps should be avoided since they crack easily and are quite porous. If a mortar cap must be used, add a latex admixture to the mortar to reduce cracking and to increase the tensile strength and bond of the mortar.

Metal wall caps can prevent water penetration effectively, provided joints between cap pieces are lapped and sealed and provided the cap extends sufficiently down the face of the masonry. Since walls

are often topped with 2 x 6 or 2 x 8 wood nailers and metal caps slope, extension of the skirt should be 4 to 6 inches in order to sufficiently cover the masonry.

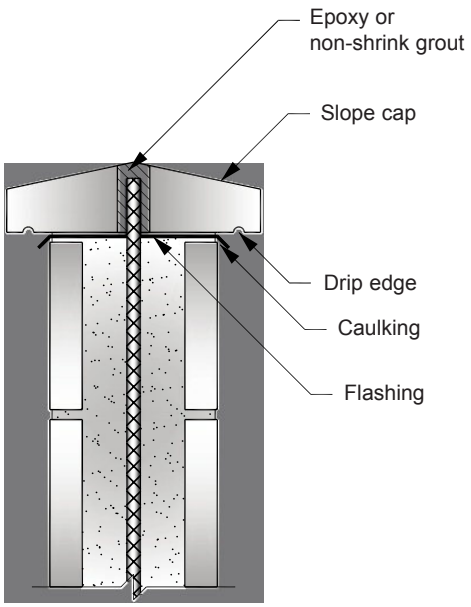


FIGURE 9.10 Typical drip edge on precast concrete wall cap.

9.2.2.6 WALL PENETRATIONS

Possibly the most overlooked areas subjected to leaking are wall openings. Door and window frames must be installed and caulked properly to eliminate leaking. Likewise, penetrations for plumbing and electrical conduits will leak if not properly flashed and caulked.

9.2.3 MATERIAL SELECTION

Because of the numerous types and diversity of masonry materials and products it is often difficult to generically recommend the best materials for a particular application. Each material has characteristics making it useful in certain types of construction. This section covers basic concerns in selecting materials for water resistant structures. For additional information refer to manufacturer literature to select masonry materials which will provide the best resistance to water infiltration.

Common water repellent coatings products are discussed in Section 9.2.5 which can be used for general information on these products. Under all circumstances, the material manufacturer should be consulted to obtain specific product information.

Specified materials should be of high quality, meeting all the appropriate standards of the industry. Choose products from reputable manufacturers that have a history of successful use of the desired product. Where appropriate, require guarantees of at least 5 years, especially for applied waterproofing products and sealants. Always completely follow the manufacturer's installation instructions.

Concrete masonry materials should be in climatic balance at the time of installation to limit the possibility of drying shrinkage cracks. "Green" block, which has not cured thoroughly or which is wet and has not achieved climatic balance, shrinks substantially and can develop numerous cracks despite proper control joints or reinforcing steel.

9.2.4 CONSTRUCTION PROCEDURES AND APPLICATION METHODS

Quality work with proper materials helps assure weathertight walls. Because of this, choose qualified, well established contractors for all aspects of construction.

Masonry industry standards and procedures should be followed throughout the construction process to help eliminate the potential for water penetration. Special care should be taken to provide adequate bond between the masonry units and mortar since leaks can occur at the bed joints. Masonry materials should be properly stored, generally off the ground and away from detrimental materials. If exposed to rain or snow, units should be covered since excessively wet units may not adequately bond to mortar and grout. Additionally, drying shrinkage cracks and efflorescence can develop if masonry materials become saturated.

Mortar and grout must be mixed thoroughly. As previously mentioned, tooled mortar joints compact the mortar, reduce cracks and improve bond.

Grout should contain sufficient water for a slump of 8 to 11 in. to flow readily into small voids and cavities. Thoroughly consolidating grout eliminates voids and also provides better bond between masonry units and reinforcing steel.

Prior to applying waterproofing products, the masonry surface must be clean and properly prepared. Oil, dust, efflorescence and other detrimental substances must be removed from the surface of masonry so the applied coatings will adhere properly. Since few waterproofing products effectively span over cracks, all cracks should be repaired.

Some coatings require the surface to be dry prior to application, while others require damp substrates. Therefore, the product manufacturer should confirm that the surface is properly prepared prior to product use. Always follow the manufacturer's recommendations fully and ensure products are applied at the appropriate coverage rates.

9.2.5 WATERPROOFING

Clay brick or concrete block masonry may require the application of a water repellent coating. Often, the masonry system sheds moisture effectively and is a true maintenance-free system. If moisture penetration through the masonry is a problem or concern, the moisture-repellent coating may be applied.

Efflorescence is the accumulation of a white, powdery film on the face of masonry. Efflorescence is caused by the existence of salts or salines in the masonry which travel to the surface and crystalize, thereby causing the white film. Salts may naturally be in the materials or may be introduced by external sources, such as rain water. The manner in which salts travel to the surface and crystalize is water, so if water is prevented from entering the masonry, efflorescence does not occur.

9.2.5.1 WATERPROOFING PRODUCTS

There are numerous waterproofing products available, each with special characteristics and attributes. The following information briefly describes the major types of waterproofing products which are readily available. Note that no product works equally well on all substrates and the manufacturer should always be consulted to determine the most effective product for the job.

9.2.5.2 BITUMINOUS WATERPROOFING PRODUCTS

Used primarily below grade to resist moisture penetration through basement and retaining walls, bituminous waterproofing products have a long history of success. To perform effectively, a system to remove seepage and/or groundwater must be provided (Figure 9.11). These products can be combined with felts or fabrics to form a built-up membrane.

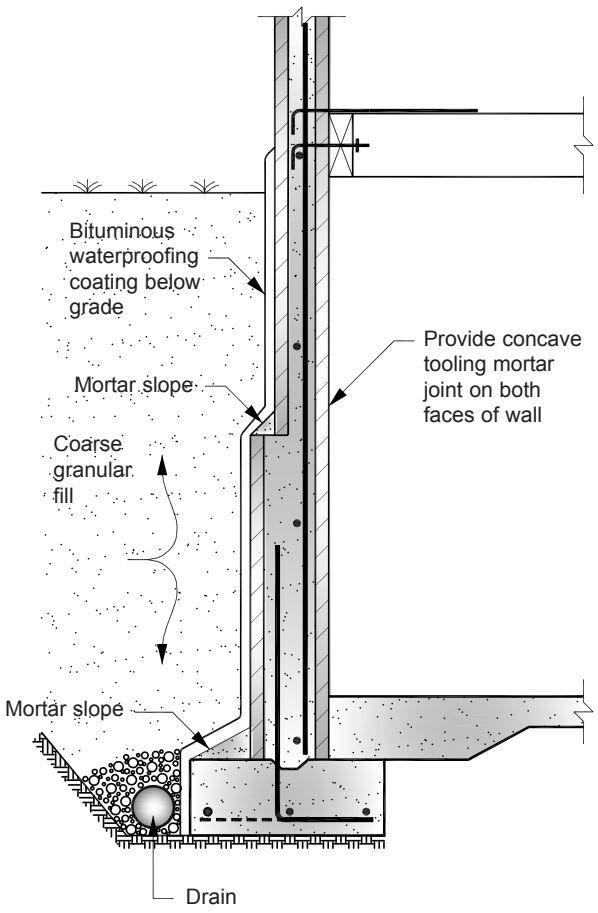


FIGURE 9.11 Bituminous waterproofing system.

9.2.5.3 CLEAR WATER REPELLENTS

Clear water repellent products may be used on brick and block walls to shield the masonry from rain water. The main advantage over other waterproofing products is that color and texture of the masonry can be seen. Some repellents can also include colored stains to enhance the masonry.

Most coatings repel water by producing high capillary pore angles so the masonry will not readily absorb water. Coatings span over only the smallest cracks and every effort should be made to fill cracks and beeholes.

Breathable water repellents are recommended so internal moisture can escape. If the moisture becomes trapped in the wall it can freeze causing severe cracking and spalling. Unfortunately, salts cannot as readily escape through some waterproofing materials. Salts may build up within the wall and cause cracking and spalling of the masonry.

Water repellents should not yellow with age nor abnormally darken the masonry surface. Repellents which do not give sheens are generally considered more acceptable. Select a repellent effective in resisting wind driven rain. Note that no known clear water repellents withstand water under pressure and therefore should not be used below grade.

9.2.5.3.1 Types of Clear Water Repellents

Four generic types of clear water repellents are prevalent: acrylics, silicones, silanes, and siloxanes.

Acrylics and Silicones

The first two, acrylics and silicones, are deposited on the surface of the masonry, forming a thin film as the solvent evaporates. Generally they are applied with a low-pressure, airless sprayer on an air dry surface. Some acrylics may slightly darken the color of the masonry.

Silanes and Siloxanes

Silanes and siloxanes are characterized as penetrating repellents which, by undergoing a chemical reaction, form a water repellent barrier in the pores of the masonry. Some of these products, especially many silanes, react more completely in the presence of moisture and alkalies. Since concrete is by nature an alkaline material, these products often form an effective barrier on moist concrete block.

9.2.5.4 Paints

Paints can also provide a relatively low cost method to achieve water resistance. With a long history of success when applied properly, paints can be very durable. Paints can add a variety of color to a masonry structure although their opaque nature can also be a disadvantage since it hides the beauty and texture of the masonry.

Like water repellents, paints should normally be breathable so internal moisture will not be trapped within the wall. Since moisture vapor enters through the interior surface of walls in cold climates and tries to exit through the exterior face of the wall, exterior paints should generally be more permeable than internal paints. If an impermeable paint is applied on the outside face of such a wall, the trapped water may cause blistering and peeling of the paint or even worse, cracking and spalling of the masonry. Because of this, impermeable paints are generally recommended only for surfaces which are constantly subjected to moisture, such as swimming pools.

9.2.5.4.1 Types of Paints

The two most common types of paint are cement-based and latex-based paints. Oil-based paints may be used however, environmental issues make oil-based paints less favorable.

Cement-Based Paints

Cement-based paints are very durable, form a hard, flat, breathable coating and are not normally harmed by alkalies, and may be applied to new concrete masonry as soon as the mortar dries. Unfortunately, these paints often chalk and fade with time and will crack and chip if applied too thickly.

Latex Paints

Latex paints are also breathable and quite durable under normal conditions, have excellent color retention, and are easy to use. Although latex paints are permeable to water, some trap salts within the wall as the water vapor escapes. Since salt build-up within the wall generates extreme pressures, which can cause spalling and cracking of the masonry, materials relatively free of salts are recommended.

9.2.5.5 Elastomeric Coatings

Elastomeric coatings are extremely water resistant but can have a high application cost. Excellent flexibility allows bridging over hairline cracks when properly applied. Elastomeric coatings can be applied in a variety of colors but, unfortunately, like paint, are not clear and transparent.

9.2.5.6 Integral Water Repellents

Used primarily in concrete masonry construction, integral water repellents provide an effective alternative to clear water repellents. These products are added directly into the concrete mix used to make the block units and must also be added into the mortar. They fill pores of the concrete masonry units and mortar, making both more water resistant. Because they are added directly into the concrete block and mortar, they should not wear off like applied repellents.

The largest drawback of integral water repellents is their inability to span over cracks or gaps in the masonry. If the mortar does not bond well to the units, water will pass through the cracks just as in any other concrete masonry wall. Therefore, whenever using

these products, special care should be taken to assure mortar joints are properly tooled and adequate movement joints provided.

Integral water repellents cannot withstand water under pressure and should not be relied on in below grade situations. Note also that integral water repellents must be added to the concrete block mix and mortar in precise dosages, as given by the manufacturer. Adding excessive amounts of these products may increase the water repellency of the wall, but it can also decrease the bond between the units and mortar. Similarly, excessive amounts of integral water repellents have been reported to retard the mortar set. Grouted masonry relies on the units to absorb excess water from the grout and integral water repellents systems do not absorb water in the same manner. Excess water may exit through cracks and pores. Plasticizers, or water reducing admixtures, may be required in grout used in integral water repellent systems.

9.2.5.7 MEMBRANE WATERPROOFING

Continuous waterproofing membranes can effectively resist water penetration under most circumstances. When designed and installed correctly, membranes can withstand water under pressure, and therefore are often applied against basement walls. By using asphalt for water resistance, and plastic polymers for added ultra violet radiation durability, these membranes can effectively resist moisture penetration through the roof side of parapets (Figure 9.8).

9.2.6 MAINTENANCE OF WATERPROOFING SYSTEMS

Throughout the life of a structure, maintenance must be performed to keep the waterproofing system working as intended. Periodic inspections of the structure should be performed to define areas requiring attention. Any work required should be performed promptly since delay often allows significant damage to occur.

Roof drains, gutters and weepholes must be kept clean and free from obstruction. Cracks in the masonry should be filled as they form. Paint and other applied waterproofing products require periodic applications in order to remain effective. Likewise, caulking and sealants should be removed and replaced as they crack or separate from the substrates. In severely deteriorated structures, broken or cracked masonry units should be replaced and deteriorated mortar joints should be re-pointed.

9.3 FIRE RESISTANCE

9.3.1 GENERAL

Masonry walls must be not only structurally adequate but also be fire resistant. Masonry walls excel in resisting the passage of heat or flames and can also be used to effectively contain most fires. A joint publication Standard by TMS and ACI "*Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies* (ACI 216.1-07/TMS-0216-07)" gives the latest information for the determination of fire resistance.

The standard describes acceptable methods for determining the fire resistance of concrete building and masonry building assemblies and structural elements, including walls, floor and roof slabs, beams, columns, lintels, and masonry fire protection for structural steel columns. These methods are based on the fire exposure and applicable end-point criteria of ASTM E119. This document is intended for determining the design requirements for concrete and masonry elements to resist fire and provide fire protection.

Fire resistance is determined by a series of fire tests conducted in accordance with the ASTM E119, *Standard Test Methods for Fire Tests of Building Construction and Materials*. These fire tests require that a wall specimen be subjected to fire having the time/temperature curve shown in Figure 9.12.

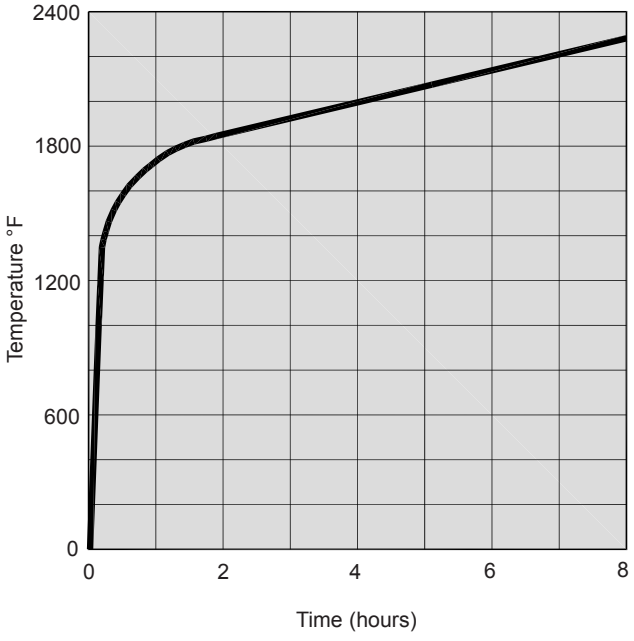


FIGURE 9.12 The ASTM E119 standard time-temperature curve.

9.3.1.1 TEMPERATURE RISE TEST

The termination of the fire test, or end point is reached, 1) when the passage of flame or gases are hot enough to ignite cotton waste on the opposite side of the wall, or 2) when the average temperature rises more than 250°F, based on temperatures recorded at no less than nine points on the unexposed side of the wall. Additionally, the temperature at any single recording point on the unexposed side of the wall may not rise to more than 325°F. The fire test rating is then given initially as an hourly rating.

9.3.1.2 HOSE STREAM TEST

A similar wall specimen to be used for the hose stream test is then subjected to a fire exposure of one half the time determined by the time/temperature curve but not to exceed one hour.

Immediately after the second fire test, the fire-exposed side of the wall specimen is subjected to the hose stream tests. The impact, erosion and cooling effects of the hose stream is first directed at the center of the wall and then at all parts of the exposed face. The water pressure and the duration of application of the hose stream are based on the rating classification time period. For example, if a wall achieves a one-hour rating, the water pressure must be 30 pounds per square inch applied to the wall for one minute per 100 square inch of exposed area.

FIGURE 9.13 Hose Stream Test.

9.3.1.3 END OF TEST

If a wall reaches the end point either by increased average temperature or single point increase, for one hour and 59 minutes, it is rated as a one hour wall. If it reaches the end point at 2 hours and 1 minute, the wall is rated as a 2 hour wall. Note: both walls must also pass the hose stream test for these ratings.

Because masonry walls resist fire penetration extremely well, the masonry wall specimen that was subjected to the time temperature fire test may also be subjected to the hose steam test. This is far more severe than the ASTM E119 requirements, but it eliminates the need to test two walls.

TABLE 9.2 ASTM E119 Acceptance Criteria for Walls

Acceptance Criteria Applicable to:					
Structural Fire Resistance			Barrier Fire Resistance		
	Sustain Load	Hose Stream	Hose Stream	Unexposed Surface Temp.	Cotton Waste
Bearing	X	X	X	X	X
Non-Bearing	NA	X	X	X	X

9.3.1.4 FIRE RATINGS (IBC)

Table 9.3 shows certain fire ratings from IBC Table 720.1(2), *Rated Fire-Resistance Periods for Various Walls and Partitions*. This table provides a classification of masonry walls based on the required wall thickness for a specified fire rating time.

The specified wall thickness for masonry shown in Table 9.3 is the equivalent solid thickness. For solid clay units, the equivalent solid thickness is the actual thickness of the unit or wall. However, for hollow clay or concrete units, the wall is considered either ungrouted or solid grouted (partial grouted walls are considered as ungrouted). Tables GN-3a and GN-3b, *Average Weight of Completed Walls and Equivalent Solid Thickness*, can be used to find the equivalent solid thicknesses of ungrouted hollow unit walls.

As an alternative to IBC Table 720.1(2), fire-resistive construction may be approved by the building official, based on evidence submitted showing that the construction meets the required fire-resistive classification.

TABLE 9.3 Rated Fire-Resistive Periods for Various Walls and Partitions[a, 1, 7] (IBC Table 720.1(2))

Material	Item Number	Construction	Minimum Finished Thickness Face-to-Face[2] (in .)			
			4 Hr	3 Hr	2 Hr	1 Hr
1. Brick of Clay or Shale	1-1.1	Solid brick of clay or shale[3]	6.0	4.9	3.8	2.7
	1-1.2	Hollow brick, not filled	5.0	4.3	3.4	2.3
	1-1.3	Hollow brick unit wall, grout or filled with perlite vermiculite or expanded shale aggregate	6.6	5.5	4.4	3.0
	1-2.1	4" nominal thick units at least 75 percent solid backed with a hat-shaped metal furring channel 3/4" thick formed from 0.021" sheet metal attached to the brick wall on 24" centers with approved fasteners, and 1/2" Type X gypsum wallboard attached to the metal furring strips with 1" long Type S screws spaced 8" on center.	–	–	5[4]	–
2. Combination of clay brick and load-bearing hollow clay tile	2-1.1	4" solid brick and 4" tile (at least 40 percent solid)	–	8	–	–
	2-1.2	4" solid brick and 8" tile (at least 40 percent solid)	12	–	–	–
3. Concrete masonry units	3-1.1[5,6]	Expanded slag or pumice	4.7	4.0	3.2	2.1
	3-1.2[5,6]	Expanded clay, shale or slate	5.1	4.4	3.6	2.6
	3-1.3[5]	Limestone, cinders or air-cooled slag	5.9	5.0	4.0	2.7
	3-1.4[5,6]	Calcareous or siliceous gravel	6.2	5.3	4.2	2.8

For SI: 1 inch = 25.4 mm, 1 square inch = 645.2 mm^2, 1 cubic foot = 0.0283 m^3.

a. Generic fire-resistance ratings (those not designated as PROPRIETARY* in the listing) in the GA 600 shall be accepted as if herein listed.

1. Staples with equivalent holding power and penetration may be used as alternate fasteners to nails for attachment to wood framing.

2. Thickness shown for brick and clay tile are nominal thicknesses unless plastered, in which case thicknesses are net. Thickness shown for concrete masonry and clay masonry is equivalent thickness defined in Section 721.3.1 for concrete masonry and Section 721.4.1.1 for clay masonry. Where all cells are solid grouted or filled with silicone-treated perlite loose-fill insulation; vermiculite loose-fill insulation; or expanded clay, shale or slate lightweight aggregate, the equivalent thickness shall be the thickness of the block or brick using specified dimensions as defined in Chapter 21. Equivalent thickness may also include the thickness of applied plaster and lath or gypsum wallboard, where specified.

3. For units in which the net cross-sectional area of cored brick in any plane parallel to the surface containing the cores is at least 75 percent of the gross cross-sectional area measured in the same plane.

4. Shall be used for nonbearing purposes only.

5. The fire-resistance time period for concrete masonry units meeting the equivalent thicknesses required for a 2-hour fire-resistance rating in Item 3, and having a thickness of not less than $7^5/_8$ in. is 4 hours when cores which are not grouted are filled with silicone-treated perlite loose-fill insulation; vermiculite loose-fill insulation; or expanded clay, shale or slate lightweight aggregate, and sand or slag having a maximum particle size of 3/8 inch.

6. The fire-resistance rating of concrete masonry units composed of a combination of aggregate types or where plaster is applied directly to the concrete masonry shall be determined in accordance with ACI 216.1/TMS 0216. Lightweight aggregates shall have a maximum combined density of 65 pounds per cubic foot.

7. NCMA TEK 5-8A, shall be permitted for the design of fire walls.

EXAMPLE 9-A Fire Resistive Period Calculation.

Calculate the fire-resistive period of a wall constructed of standard, 8 in. lightweight concrete masonry units based on the ratings in IBC Table 720.1(2) (see Table 9.3).

Approximate the net volume based on the typical dimensions of an 8 in. standard block.

$$V_n = A_n \text{ x height} = \begin{bmatrix} 2(15.625 \text{ x } 1.25) + \\ 2(5.125 \text{ x } 1.25) + \\ (5.125 \text{ x } 1) \end{bmatrix} \text{x } 7\,\tfrac{5}{8} \text{ in.}$$

$$= 57 \text{ in. x } 7\tfrac{5}{8}$$

$$T_E = \frac{V_n}{L \text{ x } H} = \frac{57 \text{ x } 7\,\tfrac{5}{8}}{15.625 \text{ x } 7\,\tfrac{5}{8}} = 3.65 \text{ in.}$$

Lightweight units made with expanded slag or pumice, provide a fire-resistive rating of 2 hours (Table 9.3).

Alternately T_E could be found assuming the block is approximately 50% solid. Thus:

$$T_E = \frac{0.50(7.625 \text{ x } 15.625) \text{x } 7.625}{15.625 \text{ x } 7.625}$$

$$= 3.8 \text{ in.}$$

From Table 9.3, the wall is still rated as 2 hours.

9.4 INTERNATIONAL SYSTEM OF UNITS (SI, SYSTEM)

9.4.1 GENERAL

The American system of measurement is gradually changing from the English system of pounds, kips, inches and feet to the International System of Measurement (SI system) as adopted in 1960 by the General Conference of Weights and Measures. Based on earlier metric systems, the SI system standardized several units of measurements which are scaled in multiples of 1000. To avoid confusion with other measurement systems, numbers are arranged in groups of three with respect to the decimal point by spaces. Thus a long block of numbers such as 12345.6789 is grouped as 12 345.678 9, not as 12,345.6789.

Masonry construction uses a 'soft' metric conversion, that is, the material sizes are not changing, but the nomenclature may be metric. A #4 (English) size reinforcing bar is referred to as a metric #13 (mm) bar.

9.4.2 MEASUREMENT CONVERSION FACTORS

Tables GN-25 through GN-31 are provided in Chapter 14 and give a comprehensive list of common conversion factors between the English and SI systems.

For convenience, common units of measurement are discussed briefly with appropriate conversion factors.

Length or Distance Measurements

Under the SI system, the basic distance measurement unit is the meter which is approximately equal to 3 ft - 3 in. or $39\frac{3}{8}$ in. A millimeter, or $\frac{1}{1000}$ of a meter, is equivalent to 0.0394 inch (about $\frac{1}{32}$ in.). Thus, one inch equals 25.4 mm.

In the SI system, meters and millimeters are predominantly used for length measurements. However, in areas where the metric system has been used for a long period, the designation of centimeters (ten millimeters) is commonly used. Mortar joint thicknesses may be given as one centimeter not ten millimeters.

A standard U.S. concrete block with nominal dimensions of 4 x 8 x 16 in. is shown in Figure 9.14 with the actual dimensions and SI conversions. Metric blocks, however, are manufactured to actual dimensions of 90 x 190 x 390 mm making them slightly smaller than U.S. standard concrete masonry units. The inclusion of 10 mm mortar joints standardizes SI nominal dimensions as 100 x 200 x 400 mm.

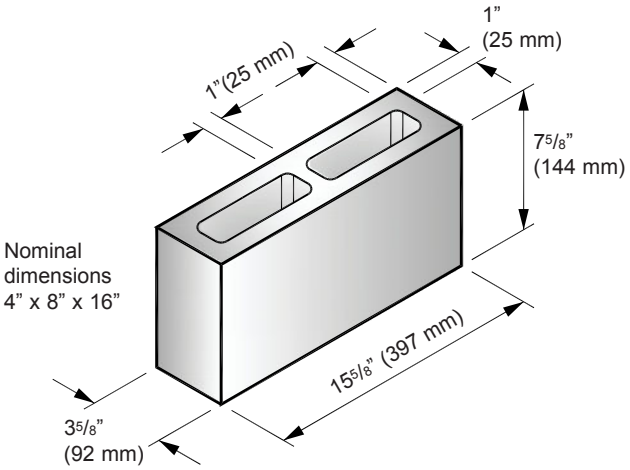


FIGURE 9.14 Standard U.S. hollow concrete unit with SI conversions shown.

Under the SI system an area is measured in square meters (m^2) where one square meter is approximately equal to 10.75 square feet (3.28 m^2). Thus, a rough approximation is 10 square feet in a square meter.

Mass

Under the SI system, mass is measured in kilograms (kg). Since one kilogram is equal to 2.205 pounds, 1000 pounds or 1 kip is equal to 454.5 kilograms, and 1 ton is equal to 907.0 kilograms.

Force

Force is mass times the acceleration of gravity, g, and although it can be stated as kilograms force, kgf, it should actually be given as newtons, N, or kilogram meter/second^2, kg•m/s^2.

Since one pound mass equals 0.454 kg, one pound force equals 0.454 kg times the acceleration of gravity (9.807 m/s^2). Thus one pound force equals 4.45 N and 1000 N equals about 225 pounds force.

Pressure

In the English system, pressure is commonly measured in pounds per square inch or pounds per square foot. In the SI system, it is measured in newtons per square meter. N/m^2 or pascals, Pa, where one pascal equals 1000 N/m^2. Pressure may also be measured in kilograms per meter second^2, kg/m•s^2. The conversion factors from psi and psf to pascals are respectively 6,894.8 psi/Pa and 47.88 psf/Pa as determined as follows:

1 psi = (1 kg/2.2 lbs) x 9.807 m/s^2 x (1 in./0.0254 m)^2
6900 kg/m•s^2 = 6900 Pa

1 psf = (1 kg/2.2 lbs) x 9.807 m/s^2 x (1 ft/0.3048 m)^2
= 47.88 kg/m•s^2 = 47.88 Pa

EXAMPLE 9-B Change f'_m = 1500 psi into pascals or mega pascals.

From Table GN-25a
1 psi = 6 894.8 Pa
1 ksi = 6.895 MPa

Thus,

f'_m = 1500 x 6 895
= 10 342 500 Pa or
= 1.5 x 6.895
= 10.34 MPa

Alternately, use Table GN-28a to find:

f'_m = 10.34 MPa

EXAMPLE 9-C Convert the modulus of elasticity of steel into SI units.

E_s = 29,000,000 psi

Since 1 psi = 6895 Pa,

E_s = 29 000 000 x 6 895
= 199 955 000 000 Pa
= 199 955 MPa
= 199.955 GPa

EXAMPLE 9-D Change the E_m equation for the modulus of elasticity of masonry into SI units.

E_m = 900 f'_m (psi) for concrete masonry and 700 f'_m for clay masonry

E_m = 900 x 6.895 f'_m for concrete masonry and 700 x 6.895 f'_m
= 6205.5 f'_m Pa (for f'_m in psi) for concrete
and 4826.5 Pa for clay masonry
= 6.21 f'_m MPa (for f'_m in psi) for concrete
and 4.83 MPa for clay masonry

EXAMPLE 9-E Check the modular ratio for f'_m = 1500 psi (concrete masonry only).

f'_m = 1500 psi = 10.34 MPa (from Example 9-B)
E_m = 900 x 10.34 = 9306 MPa
(Alternately E_m = 6.21 (1500) = 9306 MPa)

$$n = \frac{E_s}{E_m} = \frac{199\,955\ \text{MPa}}{9\,306\ \text{MPa}} = 21.5$$

EXAMPLE 9-F For w = 200 lbs/ft and l = 25 ft, find the simple beam moment in SI units.

Since

1 lb/ft = 14.585 N/m;

w = 200 lbs/ft x 14.6 N/m

= 2917 N/m

$$M = \frac{wl^2}{8} = \frac{2917 \times 7.625^2}{8} = 21200\ \text{N} \bullet \text{m}$$

l = 25 ft. x 0.305 m/ft = 7.625 m

= 21.2 kN•m

Alternate solution:

$$M = \frac{200 \times 25^2}{8}$$

M = 15,625 ft lbs

Use Table GN-29a to estimate:

M = 21 200 N•m

EXAMPLE 9-G Determine the section modulus of a section 8 in. wide by 18 in. deep.

$$S = \frac{bd^2}{6}$$

b = 8 in. = 8 x 25.4 = 203.2 mm

d = 18 in. = 18 x 25.4 = 457.2 mm

$$S = \frac{bd^2}{6} = \frac{203.2 \times 457.2^2}{6}$$

= 7 079 212 mm^3

= 7.08 x 10^3 m^3

9.5 QUESTIONS AND PROBLEMS

9-1 Name four factors which affect the size and spacing of movement joints.

9-2 Why are movement joints in clay masonry structures generally called expansion joints while most joints in concrete masonry structures are called control or shrinkage joints?

9-3 What is the estimated expansion of a 150 ft long brick wall which undergoes a temperature change of 60°F?

9-4 Name four areas where leakage can easily occur if not properly designed, detailed or constructed.

9-5 What type of waterproofing system would be prudent for (a) a brick wall, (b) a concrete block wall and (c) a concrete block basement wall.

9-6 Based on the IBC, calculate the fire-resistive period rating of a 6 in. hollow concrete masonry wall if

a) ungrouted
b) grouted at 48 in. o.c.
c) grouted at 24 in. o.c.
d) solid grouted.

Assume pumice aggregate was used to make the block.

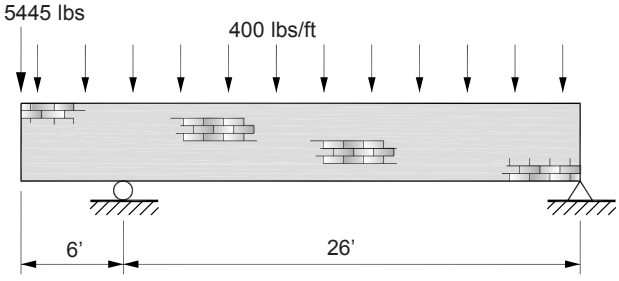
9-7 Using the IBC, find the fire-resistive period rating of a 6 in. hollow clay masonry wall if it is:

a) ungrouted
b) grouted at 48 in. o.c.
c) grouted at 24 in. o.c.
d) solid grouted.

9-8 Convert the following English units to SI units

a) 13.5 in.
b) 12 ft 7 1/4 in.
c) 367 sq ft
d) 163 ft 11 3/8 in.
e) 237 pounds
f) 43.23 kips
g) 1742 foot pounds
h) 42.7 foot kips per foot
i) 150 pounds per cubic foot
j) 3740 pounds per cubic yard
k) 1200 pounds per square inch
l) 2000 pounds per square foot
m) 26,667 pounds per square inch.

9-9 Calculate the maximum negative and positive moments in SI units for the beam shown.



9-10 Calculate the unit compressive stress in SI units for an 8 1/2 in. wall, 16 ft high with a load of 10,000 pounds per foot. If f'_m = 2000 psi what is the allowable compressive stress, F_a.

CHAPTER 10

FORMULAS FOR REINFORCED MASONRY DESIGN

10.1 GENERAL

This section is presented in two main subsections to show the formulas for reinforced masonry designed by (1) the Allowable Stress Design (ASD) Method, and (2) the Strength Design (SD) Method.

The Allowable Stress subsection is organized in two tables.

1. Table 10.1 Allowable Stress Equations
2. Table 10.2 Design Formulas

The Strength Design subsection is also organized in two tables.

1. Table 10.3 Strength Design Equations
2. Table 10.4 Design Formulas

Where applicable, IBC and MSJC Code references are provided along with reference to any application in this book.

Given the nature of the code and the relevance of ASCE-7 provisions other equations not provided may be necessary for the design and analysis of masonry provisions. Definitions of symbols and notations are provided at the beginning of this book.

10.2 ALLOWABLE STRESS DESIGN (ASD) FORMULAS

TABLE 10.1 Allowable Stress Design (ASD) Equations

Item	Allowable Stress Equation	Code Reference		Applicable Tables
		IBC	MSJC	
Allowable Tensile Stress in Steel Reinforcement, F_s	Deformed bars, F_s = 24,000 psi maximum		Sec. 2.3.2.1(b) limits F_s to 24,000 psi	Table ASD-4
	Wire reinforcement F_s = 30,000 psi maximum		Sec. 2.3.2.1(c) limits F_s to 30,000 psi	Table ASD-4
	Ties, anchors and smooth bars, F_s = 20,000 psi maximum		Sec. 2.3.2.1(a) limits F_s to 20,000 psi for Grade 40	Table ASD-4

Continued on following page

TABLE 10.1 Allowable Stress Design (ASD) Equations (continued)

Item	Allowable Stress Equation	Code Reference		Applicable Tables
		IBC	MSJC	
Allowable Compressive Stress in Steel Reinforcement, F_s	Deformed bars in columns, $F_{s(\text{compression})} = 0.4\ f_y \le 24{,}000$ psi		Sec. 2.3.2.2.2 if ties provided according to Sec. 2.1.6.5	Table ASD-4
Modulus of Elasticity, E_s, E_m, E_g	Reinforcing steel, $E_s = 29{,}000{,}000$ psi Concrete Masonry $E_m = 900\ f'_m$ Clay Masonry $E_m = 700\ f'_m$ Grout $E_g = 500\ f_g$		Sec. 1.8.2.1 Sec. 1.8.2.2.1 Sec. 1.8.2.2.1 Sec. 1.8.2.4	Table ASD-4 Table ASD-3 Table ASD-3
Shear Modulus, E_v	$E_v = 0.4E_m$		Sec. 1.8.2.2.2	Table ASD-3
Allowable Axial Compressive Stress, F_a	When $\frac{h'}{r} \le 99$, $F_a = 0.25f'_m\left[1-\left(\frac{h'}{140r}\right)^2\right]$ When $\frac{h'}{r} > 99$, $F_a = 0.25f'_m\left(\frac{70r}{h'}\right)^2$ When $\frac{h'}{r} \le 99$, $P_a = (0.25f'_m A_e + 0.65A_sF_{sc})\left[1-\left(\frac{h'}{140r}\right)^2\right]$ When $\frac{h'}{r} \le 99$, $P_a = (0.25f'_m A_e + 0.65A_sF_{sc})\left(\frac{70r}{h'}\right)^2$		Sec. 2.2.3.1(a) uses h not h' Eq. 2-12 Sec. 2.2.3.1(b) Uses h not h' Eq. 2-13 Sec. 2.3.3.2.1(a) uses h, A_n, A_{st} & F_s and not h', A_e, A_s & F_{sc} Eq. 2-17 Sec. 2.3.3.2.1(b) uses h, A_n, A_{st} & F_s and not h', A_e, A_s & F_{sc} Eq. 2-18	Tables ASD-3, ASD-9a and 9b Tables ASD-3, and ASD-9c Tables ASD-3, ASD-9a, 9b, ASD-84a thru ASD-87 Tables ASD-3, ASD-9c, ASD-84a thru ASD-87
Allowable Flexural Compressive Stress, F_b	$F_b = 0.33\ f'_m$ for unreinforced masonry and if $f_a \le F_a$ from Sec. 2.2.3.1(a) & (b) then also for reinforced masonry		Eq. 2-14 Sec. 2.2.3.1(c) Sec.2.3.3.2.2	Table ASD-3
Combined Compressive Stress (Unity Equation)	$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$ or 1.33		Eq. 2-10	
Allowable Shear Stress, F_v	Flexural members without shear reinforcement, $F_v = 1.0\sqrt{f'_m}$ $F_{v(\max)} = 50$ psi Flexural members where shear reinforcement is provided to resist all of the calculated shear, $F_v = 3.0\sqrt{f'_m}$ $F_{v(\max)} = 150$ psi		Sec. 2.3.5.2.2(a) Eq. 2-20 Sec. 2.3.5.2.3(a) Eq. 2-23	Table ASD-3 Table ASD-3

Continued on following page

TABLE 10.1 Allowable Stress Design (ASD) Equations (continued)

Item	Allowable Stress Equation	Code Reference		Applicable Tables
		IBC	MSJC	
Allowable Shear Stress in Shear Walls, F_v	Shear walls with masonry designed to carry all shear force, When $\frac{M}{Vd} < 1$ $F_v = \frac{1}{3}\left[4 - \left(\frac{M}{Vd}\right)\right]\sqrt{f'_m}$ and $F_v \le 80 - 45\left(\frac{M}{Vd}\right)$ psi When $\frac{M}{Vd} \ge 1$ $F_v = 1.0\sqrt{f'_m}$ and $F_v \le 35$ psi Shear walls with reinforcing steel designed to carry all the shear force, When $\frac{M}{Vd} < 1$ $F_v = \frac{1}{2}\left[4 - \left(\frac{M}{Vd}\right)\right]\sqrt{f'_m}$ and $F_v \le 120 - 45\left(\frac{M}{Vd}\right)$ psi When $\frac{M}{Vd} \ge 1$ $F_v = 1.5\sqrt{f'_m}$ and $F_v \le 75$ psi		Sec. 2.3.5.2.2(b) Eq. 2-21 Eq. 2-22 Sec. 2.3.5.2.3(b) Eq. 2-24 Eq. 2-25	Table ASD-5 Diagram ASD-5 Table ASD-6 Diagram ASD-6
Shear Stress, f_v, for Masonry Subject to Flexural Tension	Members shall be reinforced to resist the tension and designed for shear determined by: $f_v = \frac{V}{bd}$		Sec. 2.3.5.2.1 Eq. 2-19	Tables ASD-56, 58, 60 62 Diagrams ASD-56, 58, 60, 62
Area of Shear Steel, A_v	Minimum area of shear reinforcement required $A_v = \frac{Vs}{F_s d}$		Sec. 2.3.5.3 Eq. 2-26	Tables ASD-56, 58, 60 62 Diagrams ASD-56, 58, 60, 62
Shear Wall Maximum Flexural Reinforcement Percentage, ρ_{max}	Maximum reinforcement percentage for special reinforced masonry shear walls with $M/Vd \ge 1$ and axial load $P > 0.05f'_m A_n$ $\rho_{max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'_m}\right)}$	Sec. 2107.8 Eq. 21-3		
Development of Reinforcement, l_d	$l_d = 0.002d_b f_s$ for lap splices where f_s is computed stress in reinforcement due to design loads	Sec. 2107.5 Eq. 21-2		
	$l_d = \frac{0.13d_b^2 f_y \gamma}{K\sqrt{f'_m}} \ge 12$ in. γ = 1.0 for No. 3 thru No. 5 bars γ = 1.3 for No. 6 thru No. 7 bars γ = 1.5 for No. 8 thru No. 11 bars Equivalent Embedment Length of Standard Hooks in Tension $l_e = 11.25d_b$ Wires in Tension $l_d = 0.0015d_b F_s \ge 6$ in.		Sec. 2.1.10.3 Eq. 2-9 Sec. 2.1.10.3 Sec. 2.1.10.5.1 Sec. 2.1.10.2 Eq. 2-8	Table GN-22a Table GN-22b & ASD-22

Continued on following page

TABLE 10.1 Allowable Stress Design (ASD) Equations (continued)

Item	Allowable Stress Equation	Code Reference		Applicable Tables
		IBC	MSJC	
Allowable Bearing Stress, F_{br}	On full area $F_{br} = 0.25\, f'_m$ Where supporting area A_2 is wider on all sides than the load area A_1 $F_{br} = 0.25\sqrt{A_2 / A_1} f'_m \le 0.5 f'_m$		Sec. 2.1.9.3 Sec. 2.1.9.2	Table ASD-3 Table ASD-3
Allowable Tension on Embedded Anchor Bolts, B_t	The lesser of, $B_t = 0.5 A_p \sqrt{f'_m}$ where $A_p = \pi l_b^2$ or $A_p = \pi l_{be}^2$ $B_t = 0.2 A_b f_y$		Sec. 2.1.4.2.2 Eq. 2-1 uses B_a and not B_t Sec. 2.1.4.2.2.1 Eq. 2-3 Eq.2-4 Sec. 2.1.4.2.2 Eq. 2-2 uses B_a and not B_t	Table ASD-91 Table ASD-91
Allowable Shear on Embedded Anchor Bolts, B_v	The lesser of, $B_v = 350 \sqrt[4]{f'_m A_b}$ $B_v = 0.12 A_b f_y$		Sec. 2.1.4.2.3 Eq. 2-5 for $l_{be} \ge 12 d_b$ Eq. 2-6 for $l_{be} \ge 12 d_b$	 Table ASD-93 Table ASD-93
Combined Shear and Tension on Anchor Bolts	$\frac{b_t}{B_t} + \frac{b_v}{B_v} \le 1.0$		Eq. 2-7 uses B_a and not B_t	

TABLE 10.2 Design Formulas - Allowable Stress Design

Item	Design Formula	Standard Units	Useful Tables and Diagrams
Modular ratio, n	$n = \frac{E_s}{E_n}$		Table ASD-3
Tension Steel Reinforcement Ratio, ρ	$\rho = \frac{A_s}{bd} = \frac{K}{f_s j}$		Tables GN-23a thru GN-23m & GN-24a
Area of Tension Steel, A_s	$A_s = \rho bd = \frac{M}{f_s jd} = \frac{T}{f_s}$	sq in.	Tables GN-20a thru GN-20d & GN-24a Tables ASD-24a thru ASD-29b
Compression Steel Reinforcement Ratio, ρ'	$\rho' = \frac{K - K_b}{(2n-1)\left(\frac{k - d'/d}{k}\right)\left(1 - \frac{d'}{d}\right)2f_b} = \frac{A'_s}{bd}$		Tables ASD-74a thru ASD-79b & Diagrams ASD-74a thru ASD-79b
Area of Compression Steel A'_s	$A'_s = \frac{M - KF}{cd}$	sq in.	
Perimeter of Circular Reinforcing Bar Σ_o	$\Sigma_o = \pi d$	in.	Tables GN-19a thru GN-19c
Moment Capacity of Masonry, M_m	$M_m = \frac{1}{2} F_b kjbd^2 = Kbd^2$	in. lb ft k	Tables ASD-36 thru ASD-48b
Moment Capacity of Tension Steel	$M_s = F_s A_s jd = Kbd^2$	in. lb ft k	Tables ASD-24a thru ASD-29b
Flexural Coefficient, K_f	$K_f = \frac{1}{2} f_b kj = \frac{M}{bd^2} = f_s \rho j$		Tables ASD-24a thru ASD-29b
Coefficient, k	For members with tension steel only, $k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho$ $k = \frac{1}{1 + \frac{f_s}{nf_b}}$ Members with tension and compression reinforcement $k = \sqrt{[n\rho + (2n-1)]^2 + 2(2n-1)\rho' \frac{d'}{d}} - [n\rho + (2n-1)\rho']$		Tables ASD-24a thru ASD -29b
Coefficient, j	Members with tension steel only $j = 1 - \frac{k}{3}$ Members with tension and compression steel $j = 1 - \frac{z}{3}$		Tables ASD-24a thru ASD-29b
Coefficient, z	$z = \frac{\frac{1}{6} + \frac{(2n-1)A'_s}{kbd}\frac{d'}{kd}\left(1 - \frac{d'}{kd}\right)}{\frac{1}{2} + \frac{(2n-1)A'_s}{kbd}\left(1 - \frac{d'}{kd}\right)}$		
Dimensional Coefficient, F	$F = \frac{bd^2}{12{,}000}$		

Continued on following page

TABLE 10.2 Design Formulas - Allowable Stress Design - continued

Item	Design Formula	Standard Units	Useful Tables and Diagrams
Resultant Compression Force, C	$C = \frac{1}{2} f_b kdb$	lbs kips	
Resultant Tension Force, T	$T = A_s f_s$	lbs kips	
Tension Steel Stress, f_s	$f_s = \frac{M}{A_s jd}$	psi	Tables ASD-24a thru ASD-29b
Compression Steel Stress, f_{sc}, f'_s	$f_{sc} = 2nf_b\left(\frac{kd - d'}{kd}\right)$	psi	
Masonry Stress, f_b	$f_b = \frac{2M}{bd^2 jk} = \frac{2}{jk} K$	psi	Tables ASD-24a thru ASD-29b
Shear stress, f_v or v for beams and shear walls	$f_v = \frac{V}{bjd}$ or $\frac{V}{bd}$ or $\frac{V}{bl}$	psi	Tables ASD-56, 58,60, 62 Diagrams ASD-56, 58, 60 62
Spacing of Shear Steel, s	$s = \frac{A_v F_s d}{V}$	in.	Tables ASD-56, 58,60, 62 Diagrams ASD-56, 58, 60 62
Shear Strength provided by the Reinforcing Steel, F_v	$F_v = \frac{A_v F_s}{bjs}$ or conservatively, $F_v = \frac{A_v F_s}{bs}$	psi	Tables ASD-56, 58, 60, 62
Bond Stress, μ	$\mu = \frac{V}{\Sigma_o jd}$	psi	Tables ASD-3, GN-19a, GN-20a thru GN-20d
Effective Height to thickness reduction factor, R	$R = \left[1 - \left(\frac{h'}{140r}\right)^2\right]$ for $\frac{h'}{r} \le 99$; $R = \left(\frac{70r}{h'}\right)^2$ for $\frac{h'}{r} > 99$		Tables ASD-9a thru ASD-9c
Interaction of Axial Load and Moment	$f_b = \left(1 - \frac{f_a}{F_a}\right) F_b$ $f_a = \frac{P}{Ae} = \frac{P}{bd}$ $f_m = f_a + f_b$ $kd = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$ $a = \frac{1}{6} t f_m$ $b = -\frac{1}{2} t f_m d$ $c = P\left(\frac{l}{2} - d_1\right) + M$		
Reduced Shear Strength for Embedded Anchor Bolts Due to Edge Distance	Reduced $B_v = \left(\frac{l_{be} - 1}{12d_b - 1}\right) B_v$ for $1 \le l_{be} < 12d_b$ and $B_v = 0$ for $l_{be} < 1$		Sec. 2.1.4.2.3 Table ASD-94

10.3 STRENGTH DESIGN (SD) FORMULAS

TABLE 10.3 Strength Design (SD) Equations

Item	Strength Design Equation	Code Reference		Applicable Tables
		IBC	MSJC	
Strength Reduction Factors	ϕ = 0.90 Reinforced Masonry Flexure or Axial ϕ = 0.60 Unreinforced Masonry Flexure or Axial ϕ = 0.80 Masonry Shear ϕ = 0.50 Masonry Anchor Bolt Breakout ϕ = 0.90 Masonry Anchor Bolt Steel ϕ = 0.65 Masonry Anchor Bolt Pullout ϕ = 0.60 Masonry Bearing		Sec. 3.1.4.1 Sec. 3.1.4.2 Sec. 3.1.4.3 Sec. 3.1.4.4 Sec. 3.1.4.4 Sec. 3.1.4.4 Sec. 3.1.4.5	
Total Factored Load Equations	1.4 (D + F) 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R) 1.2D + 1.6(L_r or S or R) + (f_1L or 0.8W) 1.2D + 1.6W + f_1L + 0.5(L_r or S or R) 1.2D + 1.0E + f_1L + f_2S 0.9D + 1.6W + 1.6H 0.9D + 1.0E + 1.6H		Sec. 1605.2.1 Eq. 16-1 Eq. 16-2 Eq. 16-3 Eq. 16-4 Eq. 16-5 Eq. 16-6 Eq. 16-7	
Limits on Masonry Strain, ε_{mu}	$\varepsilon_{mu} \le 0.0025$ for concrete masonry $\varepsilon_{mu} \le 0.0035$ for clay masonry		Sec. 3.3.2c	
Limits on Masonry Strength, f'_m	1,500 psi $\le f'_m \le$ 4,000 psi for concrete masonry 1,500 psi $\le f'_m \le$ 6,000 psi for clay masonry		Sec. 3.1.8.1.1	
Limits on Grout Strength, f'_g	$f'_m \le f'_g \le$ 5,000 psi for concrete masonry $f'_m \le f'_g \le$ 6,000 psi for clay masonry		Sec. 3.1.8.1.2	
Modulus of Rupture, f_r, f_g	Modulus of Rupture for Masonry in Bending (see MSJC Code Table 3.1.8.2.1) Modulus of Rupture for Grout f_g = 250 psi		Sec. 3.1.8.2 & Table 3.1.8.2.1 Sec. 3.1.8.2.1	Table SD-24
Nominal Axial Strength of (Plain) Unreinforced Masonry, P_n	For $\frac{h}{r} \le 99$ $P_n = 0.80\left\{0.80A_n f'_m\left[1-\left(\frac{h}{140r}\right)^2\right]\right\}$ For $\frac{h}{r} > 99$ $P_n = 0.80\left\{0.80A_n f'_m\left(\frac{70r}{h}\right)^2\right\}$		Sec. 3.2.3a Eq. 3-12 Sec. 3.2.3b Eq. 3-13	

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TABLE 10.3 Strength Design (SD) Equations - continued

Item	Strength Design Equation	Code Reference		Applicable Tables
		IBC	MSJC	
Nominal Masonry Shear Strength, V_n	Nominal Shear Strength shall be the smallest of: $3.8A_n\sqrt{f'_m}$ and $300A_n$ and the applicable conditions $56A_n + 0.45N_u$ (running bond not solid grouted $56A_n + 0.45N_u$ (stack bond with open end units solid grouted) $90A_n + 0.45N_u$ (running bond solid grouted) $23A_n$ (stack bond other than open end units solid grouted)		Sec. 3.2.4 Sec. 3.2.4a Sec. 3.2.4.b Sec. 3.2.4c Sec. 3.2.4d Sec. 3.2.4e Sec. 3.2.4f	
Limits on Reinforcement Strength, f_y	$f_y \le 60{,}000$ psi Actual $f_y \le 1.3\ f_y$		Sec. 3.1.8.3	
Limits on Reinforcement	Bar size ≤ No. 9 $0.0007bd \le A_s$ for pier longitudinal reinforcement $0.0025A_n \le A_s \le 0.04A_n$ for column longitudinal reinforcement		Sec. 3.3.3.1	
Extreme Tensile Reinforcement Strain Limits	For $\frac{M_u}{V_u d_v} \ge 1$ or $R \ge 1.5$ = 1.5 (Yield strain) for flexural elements = 3 (Yield strain) for Intermediate Reinforced Masonry Shear Walls = 4 (Yield strain) for Special Reinforced Masonry Shear Walls		Sec. 3.3.3.5 Sec. 3.3.3.5.1 Sec. 3.3.3.5.2 Sec. 3.3.3.5.3	
Development of Reinforcement, l_d	$l_d = \frac{0.13d_b^2 f_y \gamma}{K\sqrt{f'_m}} \ge 12$ in. γ = 1.0 for No. 3 thru No. 5 bars γ = 1.3 for No. 6 thru No. 7 bars γ = 1.5 for No. 8 thru No. 9 bars Equivalent Embedment Length of Standard Hooks in Tension $l_e = 13d_b$		Sec. 3.3.3.3 Eq. 3-15 Sec. 3.3.3.3 Sec. 3.3.3.2 Eq. 3-14	Table GN-22a Tables GN-22b & SD-22
Nominal Axial and Flexural Strength of Reinforced Masonry, P_n	For $\frac{h}{r} \le 99$ $P_n = 0.80\left[0.80f'_m(A_n - A_s) + f_y A_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$ For $\frac{h}{r} > 99$ $P_n = 0.80\left[0.80f'_m(A_n - A_s) + f_y A_s\right]\left(\frac{70r}{h}\right)^2$		Sec. 3.3.4.1.1a Eq. 3-16 Sec. 3.3.4.1.1b Eq. 3-17	

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TABLE 10.3 Strength Design (SD) Equations - continued

Item	Strength Design Equation	Code Reference		Applicable Tables
		IBC	MSJC	
Limits on Factored Axial Load, P_u	$P_u \le 0.05A_n f'_m$ for beams $P_u \le 0.3A_n f'_m$ for piers		Sec. 3.3.4.3.1	
Limiting Vertical Stress Equation for Slender Wall Design	For Slenderness Ratio > 30 $\left(\frac{P_u}{A_g}\right) \le 0.05f'_m$		Sec. 3.3.5.4 Eq. 3-23	
Factored Axial Load at Mid-height of Walls, P_u	$P_u = P_{uw} + P_{uf}$		Sec. 3.3.5.4 Eq. 3-25	
Factored Moment at Mid-height of Walls, M_u	$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u\delta_u$ where $\left(\frac{P_u}{A_g}\right) \le 0.20f'_m$ and for Slenderness Ratio > 30 $\left(\frac{P_u}{A_g}\right) \le 0.05f'_m$ Where δ_s is determined from δ_s replacing M_{ser} with M_u where for $M_{ser} < M_{cr}$ $\delta_s = \frac{5M_{ser}h^2}{48E_m I_g}$ and for $M_{cr} < M_{ser} < M_n$ $\delta_s = \frac{5M_{cr}h^2}{48E_m I_g} + \frac{5(M_{ser} - M_{cr})h^2}{48E_m I_{cr}}$		Sec. 3.3.5.4 Eq. 3-24 Eq. 3-23 Sec. 3.3.5.5 Eq. 3-30 Eq. 3-31	
Limiting Moment Strength Equation	$M_u \le \phi M_n$		Eq. 3-26	
Nominal Moment Strength, M_n	$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right)$ where $a = \frac{\left(A_s f_y + P_u\right)}{0.80f'_m b}$ for walls where $\left(\frac{P_u}{A_g}\right) \le 0.20f'_m$ and for Slenderness Ratio > 30 $\left(\frac{P_u}{A_g}\right) \le 0.05f'_m$ and for reinforcement placed in the center of the wall		Sec. 3.3.5.4 Eq. 3-27 Eq. 3-28 Sec. 3.3.5.4 Eq. 3-23	

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TABLE 10.3 Strength Design (SD) Equations - continued

Item	Strength Design Equation	Code Reference		Applicable Tables
		IBC	MSJC	
Depth of Stress Block, a	$a = \frac{P_u + A_s f_y}{0.80 f'_m b}$		Eq. 3-28	
Mid-height Deflection Limitation for Slender Walls	$\delta_s \leq 0.007h$		Eq. 3-29	
Calculated Mid-height Deflection, δ_s	$M_{ser} \leq M_{cr}$, $\delta_s = \frac{5M_{ser}h^2}{48E_m I_g}$ $M_{cr} < M_{ser} < M_n$, $\delta_s = \frac{5M_{cr}h^2}{48E_m I_g} + \frac{5(M_{ser} - M_{cr})h^2}{48E_m I_{cr}}$		Sec. 3.3.5.5a Eq. 3-30 Sec. 3.3.5.5b Eq. 3-31	
Special Boundary Elements Requirements of Shear Walls	Not required in shear walls meeting the following conditions: $P_u \leq 0.10 A_g f'_m$ (geometrically symmetrical) $P_u \leq 0.05 A_g f'_m$ (geometrically unsymmetrical) and $\frac{M_u}{V_u l_w} \leq 1.0$ or $V_u \leq 3A_n\sqrt{f'_m}$ and $\frac{M_u}{V_u l_w} \leq 3.0$		Sec. 3.3.6.6 Sec. 3.3.6.6 1 Sec. 3.3.6.6 2 Sec. 3.3.6.6 3	
Special Boundary Elements at the Edges of Shear Walls	Required for Compression Zone with M_n at base of wall where $c \geq \frac{l_w}{600\left(\frac{C_d \delta_{ne}}{h_w}\right)}$ with vertical extension from base the larger of l_w or $\frac{M_u}{4V_u}$ Required where stresses due to factored forces including earthquake exceeds $0.2f'_m$		Sec. 3.3.6.8 Sec. 3.3.6.8(a) with P_u for ASCE 7-02 Load Case 5 Sec. 3.3.6.8(b) Sec. 3.3.6.9	
Nominal Shear Strength of a Shear Wall, V_n	$V_n = A_n \rho_n f_y$ for seismic loading $V_n = V_m + V_s$ Where for $\frac{M_u}{V_u d_v} \leq 0.25$ $V_n \leq 6A_n\sqrt{f'_m}$ and for $\frac{M_u}{V_u d_v} \geq 1.00$ $V_n \leq 4A_n\sqrt{f'_m}$	Sec. 2106.5.2 Eq. 21-1	Sec. 3.3.4.1.2 Eq. 3-18 Sec. 3.3.4.1.2(a) Eq. 3-19 Sec. 3.3.4.1.2(b) Eq. 3-20	Table SD-27 Diagram SD-27

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TABLE 10.3 Strength Design (SD) Equations - continued

Item	Strength Design Equation	Code Reference		Applicable Tables
		IBC	MSJC	
Nominal Strength Provided by the Masonry, V_m	$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u$ and $\frac{M_u}{V_u d_v} > 0$		Sec. 3.3.4.1.2.1 Eq. 3-21 Sec. 3.3.4.1.2.2	Table SD-26 Diagram SD-26
Nominal Shear Strength Provided by the Shear Reinforcement, V_s	$V_s = 0.5\left(\frac{A_v}{s}\right) f_y d_v$		Sec. 3.3.4.1.2.3 Eq. 3-22	
Bearing Strength, C_n	On full area $C_n = 0.60 f'_m A_1$ Where supporting area A_2 is wider on all sides than the load area A_1 $C_n = 0.60 A_1 \sqrt{A_2 / A_1} f'_m \le 1.2 A_1 f'_m$		Sec. 3.1.7a Sec. 3.1.7b Sec. 2.1.9.2	
Nominal Axial Tensile Strength of Headed Anchor Bolts, B_{an}	$B_{an} = 4 A_{pt} \sqrt{f'_m}$ where $A_{pt} = \pi l_b^2$ $B_{an} = A_b f_y$		Sec. 3.1.6.1 Eq. 3-1 Eq. 3-3 Eq. 3-2	Table SD-91 Table SD-92
Nominal Axial Tensile Strength of Bent-Bar Anchor Bolts, B_{an}	$B_{an} = 4 A_{pt} \sqrt{f'_m}$ where $A_{pt} = \pi l_b^2$ $B_{an} = A_b f_y$ $B_{an} = 1.5 f'_m e_b d_b + [300 \pi d_b (l_b + e_b + d_b)]$		Sec. 3.1.6.2 Eq. 3-4 Eq. 3-7 Eq. 3-5 Eq. 3-6	Table SD-91 Table SD-92
Nominal Shear Strength of Headed and Bent-Bar Anchor Bolts, B_{vn}	$B_{vn} = 4 A_{pv} \sqrt{f'_m}$ Where $A_{pv} = \frac{\pi l_{be}^2}{2}$ $B_{vn} = 0.6 A_b f_y$		Sec. 3.1.6.3 Eq. 3-8 Eq. 3-10 Eq. 3-9	Table SD-93 Table SD-93
Combined Axial and Shear Strength of Anchor Bolts	$\frac{b_{af}}{\phi B_{an}} + \frac{b_{vf}}{\phi B_{vn}} \le 1$		Sec. 3.1.6.4 Eq. 3-11	

TABLE 10.4 Design Formulas - Strength Design

Item	Design Formula	Standard Units	Useful Tables and Diagrams
Reinforcing Steel Ratio, ρ	$\rho = \frac{A_s}{bd} = q\frac{f'_m}{f_y}$		
Reinforced Design Reinforcing Steel Ratio, ρ_b	$\rho_b = \frac{0.80(0.80)f'_m}{f_y}\left(\frac{72{,}500}{72{,}500 + f_y}\right)$ for Concrete Masonry $= 5.836 \times 10^{-6} f'_m$ for f_y = 60,000 psi $\rho_b = \frac{0.80(0.80)f'_m}{f_y}\left(\frac{101{,}500}{101{,}500 + f_y}\right)$ for Clay Masonry $= 6.704 \times 10^{-6} f'_m$ for f_y = 60,000 psi		Table 6.1 Table 6.1
Maximum Flexural Reinforcing Steel Ratio, ρ_{max}	$\rho_{max} = \frac{0.64f'_m\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + 1.5\varepsilon_y}\right) - \frac{P}{bd}}{f_y}$		
Area of Tension Steel, A_s	$A_s = \rho bd = \frac{M_u}{a_u d}$	sq in.	
Maximum Area of Tension Steel, A_{smax}	$A_{s\,max} = \rho_{max} bd = \frac{0.64f'_m\, bd\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + 1.5\varepsilon_y} - P\right)}{f_y}$	sq in.	
Steel Quotient, q	$q = \frac{\rho f_y}{f'_m}$ $q = (1 - 0.625q) = \frac{M_u}{\phi f'_m bd^2}$		Tables SD-12 thru SD-19
Nominal Moment Capacity, M_n	$M_n = 0.80f'_m ab\left(d - \frac{a}{2}\right)$ for Masonry Capacity $M_n = A_s f_y\left(d - \frac{a}{2}\right)$ for Steel Capacity $M_n = \rho f_y bd^2\left(1 - \frac{0.625\rho f_y}{f'_m}\right)$ combined $M_n = K_n\, bd^2$ in terms of Flexural Coefficient	in. lb ft k	
Ultimate Moment Capacity, M_u	$M_u \le \phi M_n$ $M_u \le \phi\, bd^2 f'_m\, q\,(1 - 0.625q)$ $M_u \le \phi K_n\, bd^2 = K_u\, bd^2$	in. lb ft k	
Flexural Coefficient, K_u	$K_u = \phi K_n$ where $K_n = f'_m\, q(1 - 0.625q)$ $K_u = \phi f'_m\, q\,(1 - 0.625q)$	psi	Tables SD-2 thru SD-12

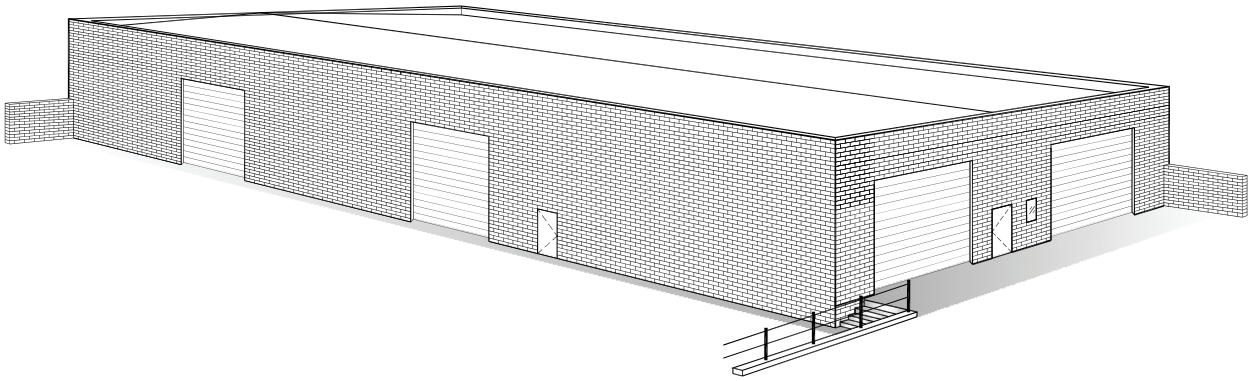
Continued on following page

TABLE 10.4 Design Formulas - Strength Design - continued

Item	Design Formula	Standard Units	Useful Tables and Diagrams
Coefficient, c_b, a_b	$c_b = \frac{0.0025d}{0.0025 + f_y / E_s} = \frac{a_b}{0.80} = \frac{72{,}500d}{72{,}500 + f_y}$ Concrete Masonry $c_b = 0.547d$ and $a_b = 0.438d$ for f_y = 60,000 psi $c_b = \frac{0.0035d}{0.0035 + f_y / E_s} = \frac{a_b}{0.80} = \frac{101{,}500d}{101{,}500 + f_y}$ Clay Masonry $c_b = 0.629d$ and $a_b = 0.503d$ for f_y = 60,000 psi	in.	
Coefficient, *a, c*	$a = \frac{\rho b d f_y}{0.80 b f'_m} = 0.80c = \frac{qd}{0.80}$	in.	
Coefficient, a_u	$a_u = \frac{K_u}{12{,}000\rho} = \frac{M_u}{A_s d} = \frac{\phi f_y (1 - 0.625q)}{12{,}000}$	psi	Tables SD-2 thru SD-12
Interaction Coefficient, *a*	$a = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$ $a = 0.4 f'_m t$ $b = 0.80 f'_m t$ (l - d_1) note l - d_1 = d b = $0.80 f'_m td$ $c = P\left(\frac{l}{2} - d_1\right) + M$		
Tension Force, *T*	$T = A_s f_y$ $= C - P$	lbs, kips	
Compression Force, *C*	C - $0.80 f'_m at$	lbs, kips	
Interpolated Shear Strength, *Vn*	For $0.25 < \frac{M_u}{V_u d_v} < 1.00$ $V_n \le \left[4 + \frac{8}{3}\left(1 - \frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m}$	lbs, kips	MSJC Code Sec. 3.3.4.1.2(c) Tables SD-26 & 27 Diagrams SD-26 & 27

CHAPTER

DESIGN OF ONE–STORY INDUSTRIAL BUILDING



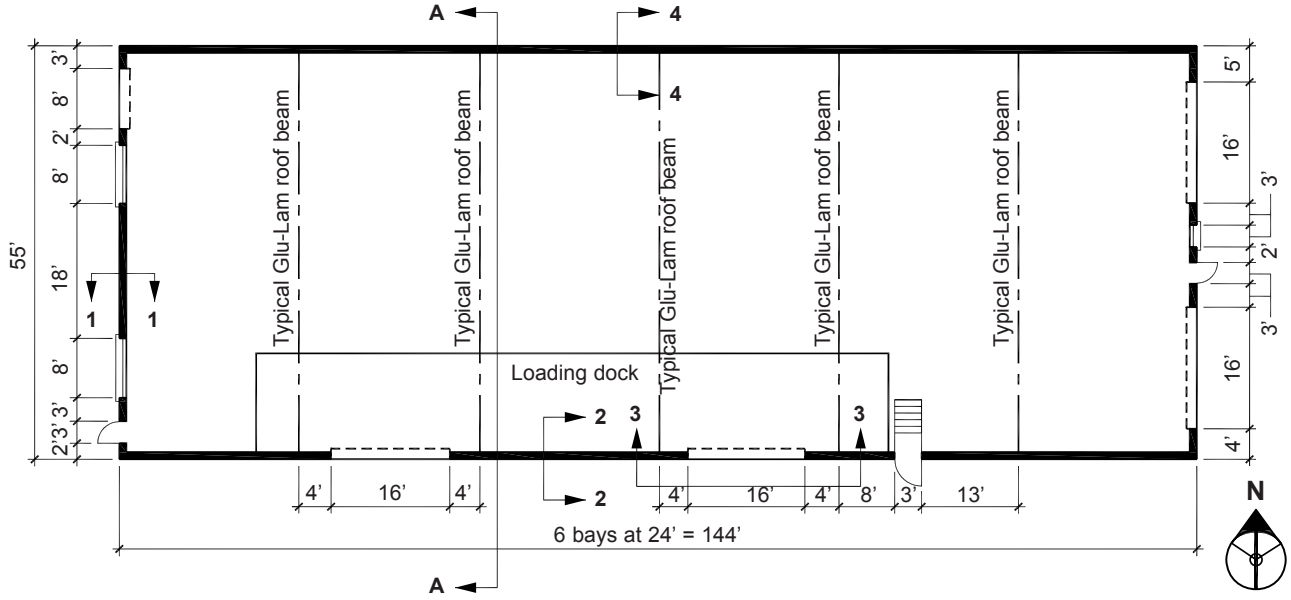
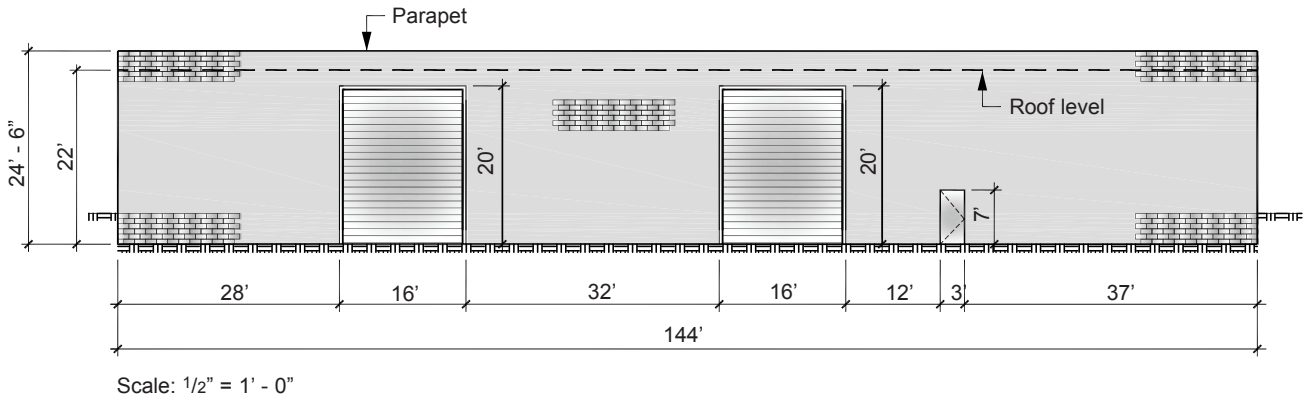
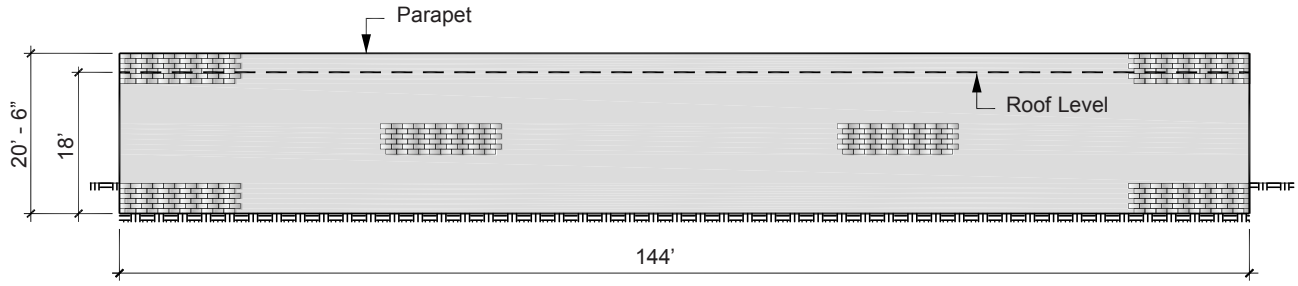


FIGURE 11.1 **Floor plan.**



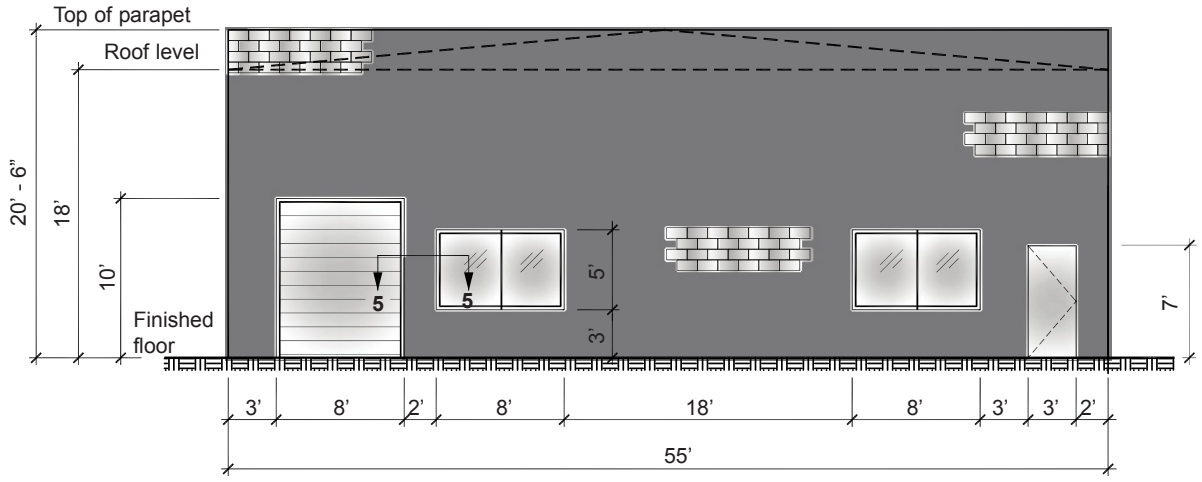
Scale: 1/2" = 1' - 0"

FIGURE 11.2 South elevation.



Scale: 1/2" = 1' - 0"

FIGURE 11.3 North elevation.



Scale: 1" = 1' - 0"

FIGURE 11.4 West elevation.

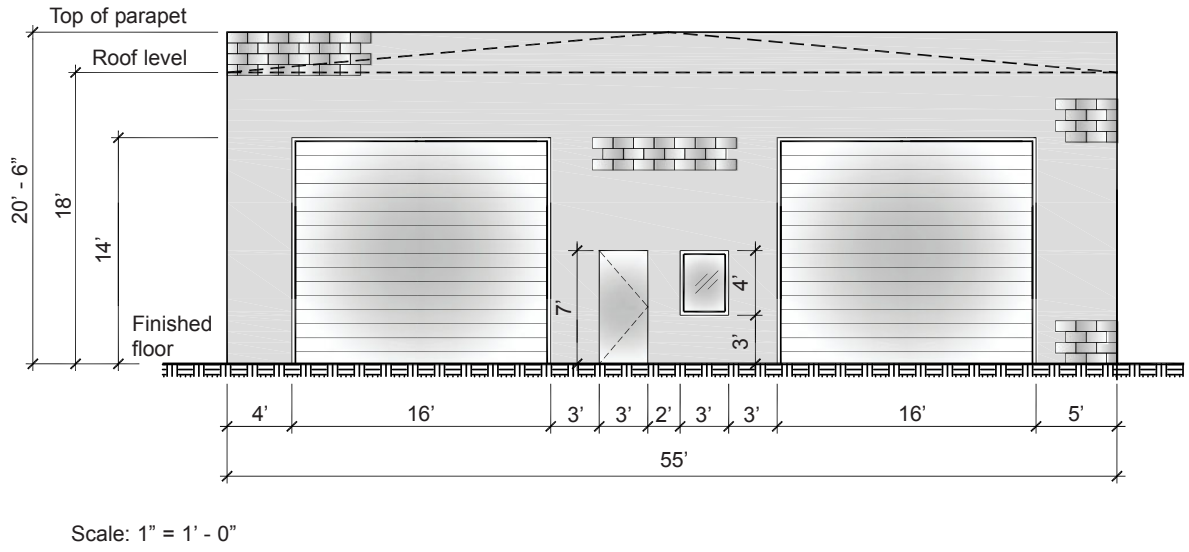


FIGURE 11.5 East elevation.

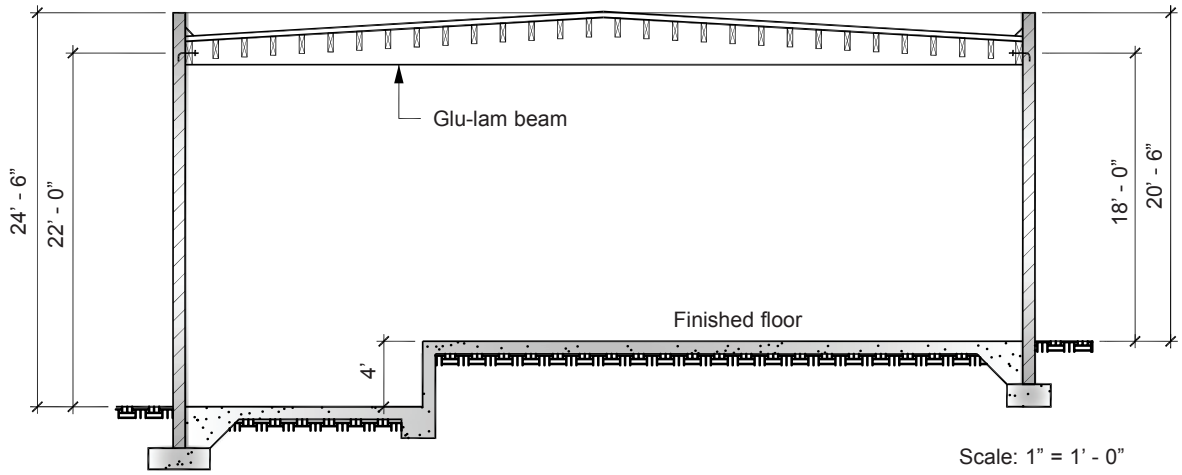


FIGURE 11.6 Typical section AA.

11.1 DESIGN CRITERIA: ALLOWABLE STRESS DESIGN

11.1.1 MATERIALS AND ALLOWABLE STRESSES

Design to be based on the 2006 IBC and 2005 MSJC Code.

Walls are to be single wythe (width) 8 in. medium weight concrete masonry walls, solid grouted.

CMU: Minimum Strength = 2400 psi

Mortar: Type S with 1 part portland cement, 1/2 part lime, and 4 1/2 parts sand.

Grout: Coarse pea gravel grout with a minimum strength of 2000 psi.

Masonry: Strength of masonry, f'_m = 1800 psi.

Reinforcing Steel: Grade 60

f_y = 60,000 psi

F_s = 24,000 psi

E_s = 29,000,000 psi

Concrete: f'_c = 3000 psi at 28 days

Modular ratio, $n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{900f'_m} = 17.9$

Maximum allowable bending compressive stresses:

$$F_b = \frac{f'_m}{3} = 600 \text{ psi}$$

All allowable stresses may be increased by 1/3 when considering wind or seismic forces (MSJC Code Section 2.1.2.3)

11.1.2 LOADS

11.1.2.1 LATERAL LOADS (WIND AND SEISMIC)

11.1.2.1.1 SEISMIC LOADS (IBC CHAPTER 16)

In recent years, the seismic design of even simple structures has become complex. From the seismic maps it is determined that S_S = 0.5 and S_1 = 0.2. There are many steps:

Seismic Design Category:

1. Adjust seismic Maximum Considered Earthquake (MCE) coefficients for soil, ASCE 7 Tables 11.4-1 and 11.4-2.

   The building is on Site Classification B.

   $S_{MS} = F_a S_S = (1.0)\ (0.5) = 0.50$

   $S_{M1} = F_v S_1 = (1.0)\ (0.2) = 0.20$

2. Spectral Design Parameters

   The design parameters are 2/3 of the values above.

   $S_{DS} = (2/3)\ (0.50) = 0.33$

   $S_{D1} = (2/3)\ (0.20) = 0.13$

3. Importance Factor

   Since the occupancy is I, the importance factor is 1.0 (ASCE 7 Table 11.5-1)

4. Seismic Design Category

   From ASCE 7 Tables 11.6-1 and 11.6-2 the building is a Seismic Design Category C building.

The period maybe taken as:

$T = C_t h_n^x$ ASCE 7 Table 12.8-7

C_t = 0.02 ASCE 7 Table 12.8-2

H_n = 20.5 ft

x = 0.75 ASCE 7 Table 12.8-2

T = 0.19 seconds

From ASCE 7 Table 12.2-1 we will be conservative and select walls conforming to "Special Reinforced Masonry" shear walls. The out-of-plane requirements will likely result in the required reinforcement. Thus R = 5

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.33}{\left(\frac{5}{1}\right)} = 0.066$$

Base shear is therefore:

$V = (0.066)\ (W)$

Seismic load on portions of the building:

$F_p = 0.4 S_{DS} W_p = (0.4)\ (0.33)\ (W_p) = 0.132 W_p$ (ASCE 7 Section 12.11)

Since the walls weigh 78 psf (Medium weight units solidly grouted), F_p = 0.132(78) = 10.2 psf

11.1.2.1.2 WIND LOADS (PER ASCE 7 METHOD 2)

The wind speed is 85 mph and the exposure is "B". ASCE 7 method 2 will be used. The building conforms to the limitations of method 2.

The mean roof height is 26.5 ft

V = 85 mph (from wind maps)

The wind importance factor is 1.0.

Assume this is an "enclosed" building.

The internal pressure coefficient GC_{pi} = 0.18

K_d = 0.85 (ASCE 7 Table 6-4)

K_{zt} = 1.0 (Assume no hills in area)

K_h = 0.70 (ASCE 7 Table 6-3)

q_h = 10.6 psf (ASCE 7 Eq 6-15)

p = 16.7 psf (ASCE 7 Eq 6-22)

11.1.2.2 VERTICAL LOADS

No snow load considered

Roof load - Slope roof 1/4 in./ft for drainage

Roofing (Built up)	=	4.0 psf
Plywood, 1/2" thick	=	1.5
Framing	=	2.5
Sprinklers	=	1.5
Miscellaneous	=	1.5
Roof framing	=	11.0 psf
Roof beams	=	3.0
Total roof-dead load	=	14.0 psf
Roof live load (reduceable)	=	20.0
Total dead and live loads for load bearing wall design	=	34.0 psf

Roof live load for pilaster

Design (Live load reduced as allowed in ASCE 7 4.9.1).

See bearing angle design (Section 11-5 for calculation) = 12.0 psf

Total dead and live loads for pilaster design (14 + 12) = 26.0 psf

Weight of wall (From Table GN-3a) (verify the value) = 78.0 psf

11.2 DESIGN OF WEST MASONRY BEARING WALL – SECTION 1-1

11.2.1 VERTICAL LOADS ON WALL

Tributary width of roof = 1/2 x 24 ft roof span

= 12 ft per ft length of wall

Roof live load = 20 psf (not reduced) x 12 ft = 240 plf

Roof dead load = 14 psf x 12 ft = 168 plf

Dead load on wall at mid-height (between footing and ledger beam)

DL parapet = 78 psf x 2.5 = 195 plf

DL wall = 78 psf x 1/2(18 ft) = 702 plf

Total wall *DL* at mid-height = 897 plf

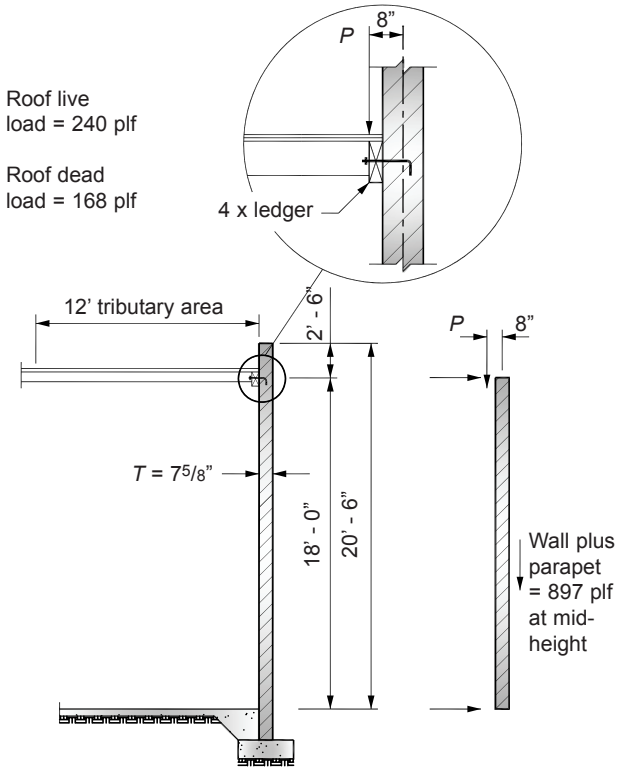


FIGURE 11.7 West wall Section 1-1.

11.2.2 LATERAL FORCES ON WALL

From Section 11.1.2, Lateral loads:

Wind force, p = 16.7 psf

Seismic force, F_p = 10.2 psf < 16.7 psf

Seismic calculation is an ultimate load and wind load is allowable. In this case, the wind force governs. Use this value for the design of the wall between the ledger and the footing.

Lateral wind moment on wall assuming pin connection at top and bottom with no wind load on parapet.

No wind load on the parapet is a conservative assumption.

$$M = (1/8)wh^2 = 1/8 \times 16.7 \times 18^2$$

= 676 ft lbs/ft

Moment due to eccentric roof dead load

$$M = 168 \times \frac{8}{12} = 112 \text{ ft lbs/ft}$$

Moment at mid-height

M = 1/2 x 112 = 56 ft lbs/ft

Moment due to eccentric roof live load

$$M = 240 \text{ x } \frac{8}{12} = 160 \text{ ft lbs/ft}$$

Moment at mid-height

M = 1/2 x 160 = 80 ft lbs/ft

Total moment, lateral plus roof *DL* at mid-height

M = 676 + 56 = 732 ft lbs/ft

Maximum moment at mid-height

M = 676 + 56 + 80 = 812 ft lbs/ft

Total moment at top of wall due to live and dead loads

M = 112 + 160 = 272 ft lbs/ft

11.2.3 VERTICAL LOAD ON WALL AT MID-HEIGHT

Vertical dead load on wall at mid-height

= roof + parapet + 1/2 wall

= 168 + 78 (2.5 + (1/2 x 18)) = 1065 plf

Vertical dead and live load on wall at mid-height

= 1065 + 240 = 1305 plf

NOTE: When lateral wind or seismic forces are considered, the roof live load is generally ignored except for heavy snow load.

11.2.4 DESIGN WALL FOR CONDITION AT MID-HEIGHT — SECTION 1-1

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{12(7.625)^3/12}{(7.625)(12)}} = 2.2$$

$$\frac{H}{r} = \frac{18 \text{ x } 12}{2.2} = 98$$

From MSJC Code – Equation 2.17

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

Neglecting the contribution of the reinforcement,

$$F_a = \frac{P_a}{A_n} = (0.25)(1800)\left[1 - \left(\frac{(18)(12)}{(140)(2.2)}\right)^2\right] = 228 \text{ psi}$$

The applied axial stress, $f_a = \frac{P}{A}$

$$= \frac{1065}{(7.625)(12)} = 11.6 \text{ psi}$$

Bending moment at mid-height = 812 ft lbs. Design wall to span vertically between footing and diaphragm.

Steel is located in the center of the wall:

d = 7.625/2 = 3.8125 in.

Check flexural stress as if section was uncracked

$$f_b = \frac{6M}{bt^2} = \frac{(6)(812)(12)}{(12)(7.625)^2} = 83.8 \text{ psi}$$

= 83.8 psi > 11.6 psi

The bending stress is greater than the dead load axial stress–therefore design for tension as a cracked section.

Maximum moment at mid-height,

M = 812 ft lbs/ft

Estimate the required area of reinforcement:

$$A_s \cong \frac{M}{0.9F_s d} = \frac{(812)(12)}{(0.9)(24000)(4/3)(3.8125)} = 0.088 \text{ in.}^2$$

Where 0.9 is an estimate for j and the allowable stress of 24,000 has been increased by 1/3 as allowed for load combinations including wind.

For bars spaced at 48 in., A_s = 0.35 in.^{2}. Try (1) No. 6 at 48 in. on center.

$$\rho = \frac{A_s}{bd} = \frac{0.44}{(48)(3.8125)} = 0.0024$$

$$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{(900)(1800)} = 17.9$$

$n\rho$ = 0.043

$$k = \sqrt{n\rho + 2n\rho} - n\rho = 0.253 \qquad j = \left(1 - \frac{k}{3}\right) = 0.916$$

$$M_t = A_s jdF_s = (0.44)\,(0.916)\,(3.8125)\,(24{,}000)\left(\frac{4}{3}\right)$$

$$= 49{,}143 \text{ lb-in.}$$

$$M_c = \frac{bd^2}{2} kjF_b$$

$$= \frac{(48)(3.8125)^2}{2}(0.253)(0.916)\left(\frac{1800}{3}\right)\left(\frac{4}{3}\right)$$

$$= 64{,}765 \text{ lb-in.}$$

Tension controls. The applied moment on the 48 in. of wall is:

$$(812)\,(4)\,(12) = 38{,}900 \text{ lb-in.}$$

Use the linear interaction diagram to check the capacity of the wall. Use the alternative load combinations IBC Section 1605.3.2

$$D + L + (\omega W)$$

The ω is equal to 1.3. The moment is:

$$M_{applied} = [(1.3)(676) + 56 + 80](4)(12) = 48{,}710 \text{ lb-in.}$$

$$\frac{M_{applied}}{M_{allowable}} + \frac{P_{applied}}{P_{allowable}} \le 1$$

$$\frac{48{,}710}{49{,}143} + \frac{(1305)(4)}{(228)(7.625)(48)} = 0.99 + 0.06 = 1.05$$

The design is 5% over. This is generally considered close enough. The linear interaction diagram is conservative, or, the slender wall provisions of MSJC Code Section 3.3.5.4 could be used as demonstrated in the next section.

11.3 DESIGN OF SOUTH MASONRY WALL – SECTION 2-2

11.3.1 SLENDER WALL

MSJC Code Section 3.3.5.4 provides a method to design walls with high aspect ratios (height to thickness). To qualify for this method the factored axial load must be less than 0.20 f'_m for H/T less than or equal to 30 and 0.05 H/T for greater than 30.

The wall is 22 ft tall with a 2.5 foot parapet. It is conservative to neglect the fixity (reduction in moment) caused by the parapet. The wall will be designed as a 22 foot simple span between the foundation and the roof using the slender method.

Assume an 8 in. CMU wall. Use 4 foot section of wall.

The loading from the roof is small, since the joists span parallel to the wall. The loading is less than 100 plf and will be neglected for now. The primary dead load is the weight of the wall. Assuming the weight at 78 lbs per square foot, the dead load on 4 foot section of wall at 11 foot above the base is:

$$[(2.5)\,(78) + (11)\,(78)]\,(4) = 4212 \text{ lbs}$$

The controlling load combinations from the IBC are equations 16-4 and 16-6.

$$1.2D + 1.6W + 0.5L_r \qquad \text{(IBC Eq 16-4)}$$

$$0.9D + 1.6W \qquad \text{(IBC Eq 16-6)}$$

The larger dead load equation will be used to check dead load limits on the method and equation 16-6 will be used for the amount of reinforcement required.

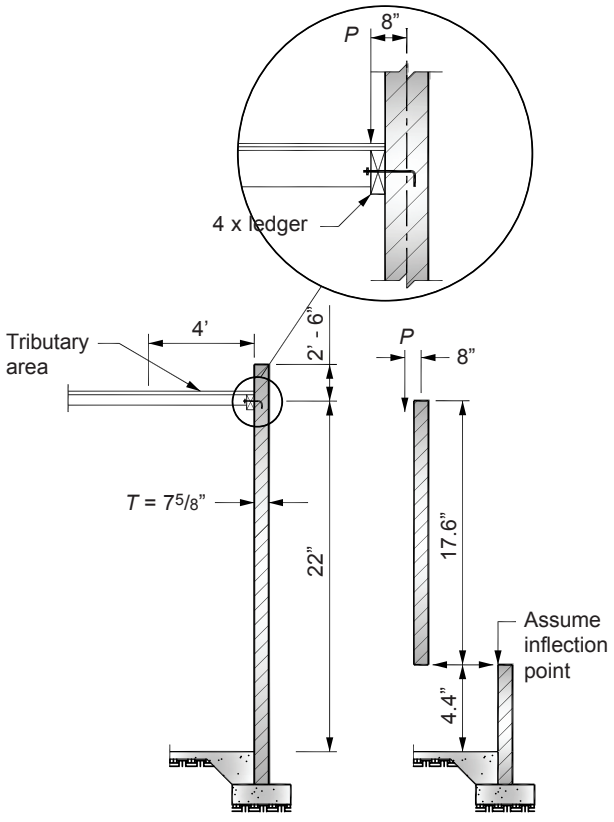


FIGURE 11.8 South wall Section 2-2.

Check the limiting axial load:

H/T = (22)(12/8) = 33 (The code allows the nominal thickness to be used)

$$\frac{P_u}{A_g} = \frac{(1.2)(4212)}{(48)(7.625)} = 13.8 \text{ psi} < 0.05\, f'_m = (0.05)(1800)$$

$$= 90 \text{ psi}$$

The method can be used.

The ultimate moment is given by MSJC Code equation 3-24.

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u$$

Where:

P_u = $P_{uw} + P_{uf}$

P_u = the factored axial load.

P_{uw} = the factored axial load from the roof or floor.

P_{uf} = the factored wall dead load at the point of moment.

δ_u = The out-of-plane deflection of the wall at the point of ultimate moment.

e_e = The eccentricity of the roof or floor applied load.

The method requires iteration of the δ_u deflection. Assume an initial value of the code provided limit on service loading of 0.007

δ_u = (0.007) (22) (12) = 1.8 in.

Using load combination 16-6:

$$M_u = \left[(16.7)(1.6)\frac{(22)^2}{8}\right] + (4)(12) + 0 + (0.9)(4212)(1.8)$$

= 77,600 + 6,800 = 84,400 lb-in./4 foot

Approximate the area of reinforcement:

$$A_s \cong \frac{M_u}{(\phi)(0.9)(d)\left(F_y\right)} = \frac{84{,}400}{(0.9)(0.9)(3.8125)(60{,}000)}$$

$$= 0.46 \text{ in.}^2$$

Try No. 6 at 48 in. o.c.

b = 48 in. (The maximum allowed by code is 6 times the nominal thickness of the wall or the bar spacing).

d = 3.8125 in.

A_s = 0.44

Calculate the nominal moment:

$$M_n = \left(A_s F_y + P_u\right)\left(d - \frac{a}{2}\right)$$

$$a = \frac{\left(A_s F_y + P_u\right)}{0.8\, f'_m\, b}$$

$$a = \frac{(0.44)(60{,}000) + (0.9)(4212)}{(0.8)(1800)(48)}$$

= 0.436

$$M_n = [(0.44)(60{,}000) + (0.9)(4212)]\left(3.8125 - \frac{0.436}{2}\right)$$

= 108,500 lb-in/4 foot

$M_u < \phi M_n$ = (0.9)(108,500) = 98,000 lb-in./4 foot OK

We must now verify our assumption about the deflection. The code provides equations for the deflection at the center of the wall.

If the wall is uncracked the equation is:

$$\delta_s = \frac{5 M_u h^2}{48 E_m I_g}$$

If the wall is cracked:

$$\delta_s = \frac{5 M_u h^2}{48 E_m I_g} + \frac{5\left(M_u - M_{cr}\right)h^2}{48 E_m I_{cr}}$$

Where:

$$I_{cr} = n A_{se}\,(d - c)^2 + \frac{bc^3}{3}$$

$$A_{se} = A_s + \frac{P_u}{F_y}$$

$$c = \frac{A_{se} F_y}{0.64\, f'_m\, b}$$

The gross moment of inertia is:

$$I_g = \left(\frac{1}{12}\right)(48)(7.625)^3 = 1{,}773 \text{ in.}^4$$

$$A_{se} = 0.44 + \frac{3791}{60{,}000} = 0.503$$

$$c = \frac{(0.503)(60{,}000)}{(0.64)(1800)(48)} = 0.545$$

$$I_{cr} = (17.9)(0.503)(3.8125 - 0.545)^2 + (48)\left(\frac{(0.545)^3}{3}\right)$$

$$= 98.7 \text{ in.}^4$$

Determine if the wall is cracked. From MSJC Code Table 3.1.8.2.1 the modulus of rupture of the CMU, solidly grouted for tension across the bed joint is 163 psi. The cracking moment is:

$$M_{cr} = 2\left[f_r + \frac{P_u}{A}\right]\left[\frac{I_g}{t}\right]$$

$$= 2\left[163 + \frac{3791}{(48)(7.625)}\right]\left[\frac{1773}{7.625}\right]$$

$$= 80{,}600 \text{ lb-in./4 ft}$$

The wall is cracked, therefore the deflection is:

$$\delta = \left[\frac{(5)(84{,}400)[(22)(12)]^2}{(48)(1{,}620{,}000)(1773)}\right] + \left[\frac{(5)[(84{,}400 - 80{,}600)][(22)(12)]^2}{(48)(1{,}620{,}000)(98)}\right]$$

$$= 0.21 + 0.17 = 0.38 \text{ in.}$$

The deflection is less than the 1.8 in. assumed, therefore the design is adequate. The service requirement of the defection being less than 0.077 *h* is also obviously satisfied.

11.4 DESIGN OF LINTEL BEAM SOUTH WALL – SECTION 3-3

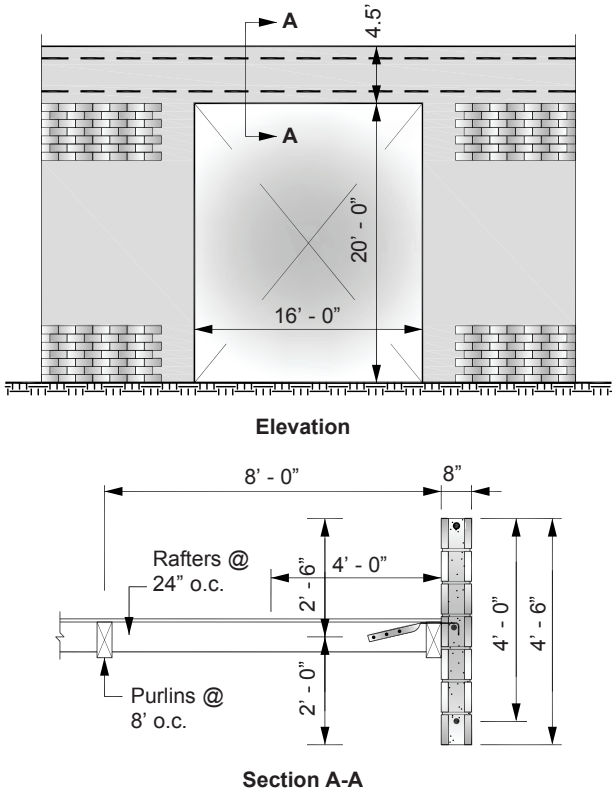


FIGURE 11.9 Lintel beam.

11.4.1 FLEXURAL DESIGN

Dead load from roof = 11 psf x 4 ft = 44 plf

Live load from roof = 20 psf x 4 ft = 80 plf

Dead load of wall = 78 psf x 4.5 ft = 351 plf

475 plf

b = 7.625 in.

d = 48 in

$$M = \frac{wL^2}{12} = \left(\frac{(475)(16)^2(12)}{12}\right) = 122{,}000 \text{ lb} - \text{in.}$$

Estimate the required area of reinforcement:

$$A_s \cong \frac{M}{0.9 F_s d} = \frac{(122{,}000)}{(0.9)(24000)(48)} = 0.12 \text{ in.}^2$$

Estimate 0.9 for *j* and the allowable stress of 24,000 is not increased by 1/3.

Try (1) No. 4.

$$\rho = \frac{A_s}{bd} = \frac{0.2}{(48)(7.625)} = 0.00054$$

$$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{(900)(1800)} = 17.9$$

$n\rho = 0.0098$

$$k = \sqrt{n\rho + 2n\rho} - n\rho = 0.13 \qquad j = \left(1 - \frac{k}{3}\right) = 0.96$$

$$M_t = A_s jdF_s = (0.2)(0.96)(48)(24{,}000) = 221{,}000 \text{ lb - in.}$$

$$M_c = \frac{bd^2}{2} kjF_b = \frac{(7.625)(48)^2}{2}(0.13)(0.96)\left(\frac{1800}{3}\right)$$

$$= 658{,}000 \text{ lb-in.}$$

Tension controls and is more than the applied moment.

Check for Shear:

$V = (475)(8) = 3800$ lb

The computed shear stress (MSJC Code Equation 2-19) is:

$$f_v = \frac{V}{bd} = \frac{3800}{(7.625)(48)} = 10.4 \text{ psi}$$

The allowable shear stress in a beam is given by MSJC Code Equation 2-20.

$$F_v = \sqrt{f'_m} = \sqrt{1800} = 42.4 \text{ psi} \le 50 \text{ psi}$$

No shear reinforcement is required.

11.4.2 LATERAL WIND LOAD ON BEAM

Moment

$$M = \frac{wL^2}{12} = \frac{(16.7)(4.5)(16)^2}{12} = 1603 \text{ lb - ft}$$

b = 54 in.

d = 3.8125 in.

A_s = 0.4 in.^{2}

n = 17.9

k = 0.243

j = 0.919

M_t = 2.800 lb - in.

M_c = 3.900 lb - in.

The horizontal #4 bars are adequate.

11.4.3 DEEP LINTEL BEAMS

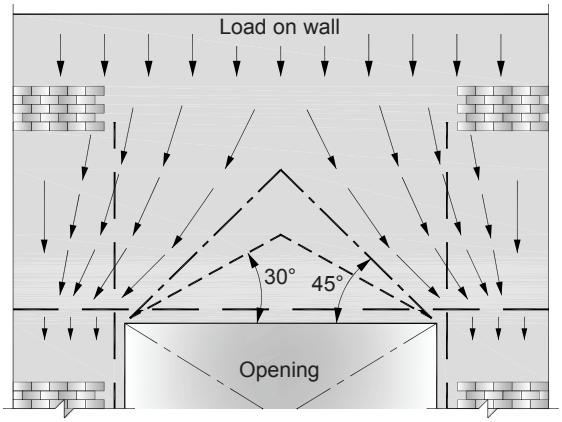


FIGURE 11.10 Distribution of load over an opening.

The wall over a lintel will tend to create an arch over the opening leaving the area under the arch to be carried by the lintel beam. A conservative arch angle would be 45°; although an angle of 30° would probably be more realistic to the true arch action of reinforced masonry.

11.5 DESIGN OF FLUSH WALL PILASTER NORTH WALL – SECTION 4-4. DESIGNED AS A WALL NOT A COLUMN

11.5.1 LOADS

Tributary area = (55/2)(24) = 660 sq ft

Live load = 20 psf. Reduce as permitted in ASCE 7 to 12 psf.

Live load from roof	= 12 x 660 =	7,920 lbs
Dead load from roof	= 14 x 660 =	9,240 lbs
Total load (*DL* + *LL*)	=	17,160 lbs

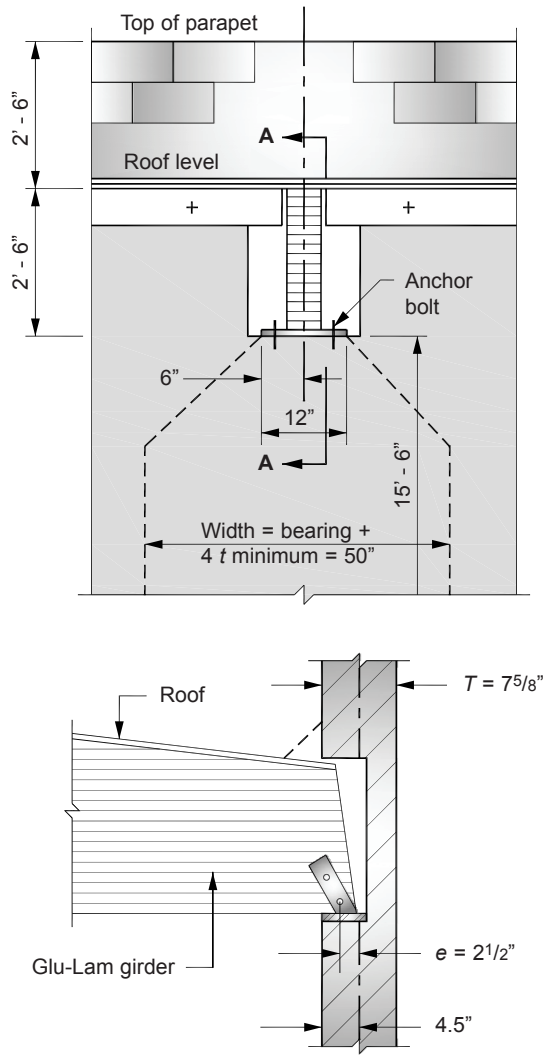


FIGURE 11.11 Flush pilaster width.

11.5.2 BEARING PLATE DESIGN

Bearing plate under Glu-Lam roof girder.

Allowable masonry bearing stress on the full area per MSJC Code Section 2.1.9.

$F_{bv} = 0.25f'_m = 0.25 \times 1800 = 450$ psi

Required area of bearing plate = 17160/450

= 38.1 in.^{2}

Glu-Lam beam assumed $6\frac{3}{4}$ in. x 30 in.

Width of bearing plate, assume 5 in. This leaves sufficient width for the CMU face shell to pass.

Required length of plate = 38.1/5 = 7.6 in.

Use a 12 in. plate. This is larger than required, but we want to anchor the plate to the CMU. The cell spacing is 8 in., and the added 2 in. on each side is to provide tolerance and edge distance for the anchor bolts.

To determine the bending moment on the bearing plate, assume that the maximum moment or the point of fixity is at the center of the Glu-Lam beam, half way between the ends of the bearing plate and that stresses are evenly distributed.

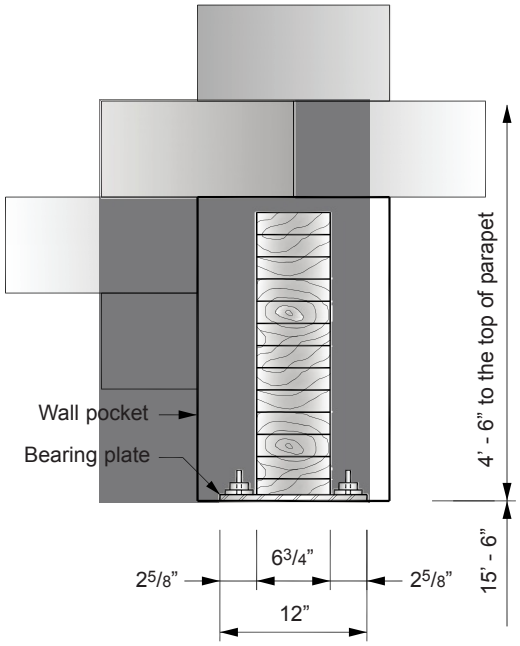


FIGURE 11.12 Glu-Lam beam bearing plate.

Uniform pressure under the bearing plate

$$= \frac{17{,}160}{(12)(5)} = 286 \text{ psi}$$

$$\text{Moment} = (286)(5)\left(\frac{6^2}{2}\right)$$

= 25,740 in. lbs

Assume $F_s = 0.66F_y = 0.66(36{,}000) = 24{,}000$ psi

The bearing stress between the plate and the masonry is:

$$\frac{17{,}160}{(12)(5)} = 286 \text{ psi}$$

The bearing stress between the Glue-Lam and the plate is:

$$\frac{17{,}160}{(6.75)(5)} = 508.4 \text{ psi}$$

The moment in the bearing plate is:

$$\frac{(286)(5)(6)^2}{2} - \frac{(508.4)(5)(3.375)^2}{2} = 11{,}263 \text{ lb - in.}$$

The required plate thickness is:

$$t = \sqrt{\frac{6M}{F_s b}} = \sqrt{\frac{(6)(11{,}263)}{(24{,}000)(5)}} = 0.75 \text{ in.}$$

Use a 3/4 x 5 x 1'-0" plate.

The anchor bolts need to be sized. Uplift wind loads could control. Assume 20 psf uplift wind loading (a more accurate value can be determined in ASCE 7, but this should be conservative).

Uplift is: (660)(20) = 13,200 lbs

The dead load from the roof is 9240 lbs. For overturning situations, the code typically uses 2/3 of the dead load to relieve wind overturning. This is not an overturning situation and the 2/3 is probably too conservative. But it shouldn't result in bolts that are abnormally large. Use the 2/3.

Uplift = 13,200 - (9,240) (2/3) = 7,040 lbs

Try (2) 3/4 in. anchor bolts embedded 18 in. Use MSJC Code Section 2.1.4.

$$B_a = 0.5A_p\sqrt{f'_m}$$

$$= (0.5)(18 + 9 + 18)(7.625)\sqrt{1800}\left(\frac{4}{3}\right) = 9704 \text{ lbs}$$

$$B_a = 0.2A_b f_y$$

$$= (0.2)(2)(0.44)(0.85)(36{,}000)\left(\frac{4}{3}\right) = 7{,}180 \text{ lbs OK}$$

The value of A_p is the projected area of the cone on the surface of the masonry. The 3/4 in. bolt was reduced in area by 85% of account for the threads. Since this is a wind load condition, the allowables are increased by the 1/3 stress increase.

11.6 DESIGN OF SECTION 5-5 FOR VERTICAL AND LATERAL LOADS

Out-of -Plane Design.

The out-of-plane loading is either wind or seismic. The wind force controls and is 16.7 psf. The maximum moment occurs where the shear is zero. Summing moments about the base of the wall results in the reaction at the roof-wall intersection:

$$R_1 = \left[\frac{(16.7)\frac{(20.5)^2}{2}}{18}\right] = 195 \text{ lb/ft}$$

$$R_2 = \left[\frac{(16.7)\left[\frac{(18.0)^2}{2} - \frac{(2.5)^2}{2}\right]}{18}\right] = 147.0 \text{ lb/ft}$$

The maximum moment occurs at:

$$X = \frac{147.0}{16.7} = 8.8 \text{ ft}$$

This is slightly above the window. Design for the maximum moment at the window head. The tributary area is:

$$W = \frac{8}{2} + 2 + \frac{8}{2}$$

And the moment is:

$$M_{max} = \frac{(147.0)(8.8)}{2}(10) = 6470 \text{ lb - ft}$$

The dead load from the roof:

(11)(12)(10) = 1320 lb/ft

The dead load of the wall is:

[(4)(10.5) + (6)(12.5)] (78) = 9,126 lbs

The live load is:

(20)(12)(10) = 2,400 lbs

The following table summarizes the loading on the pier:

TABLE 11.1 West Elevation Pier Loading

Load	Axial Load (lbs)	Out-of-Plane Moment (lb-ft)
Dead Load	10,450	0
Live Load	2,400	0
Wind Load	0	6,470

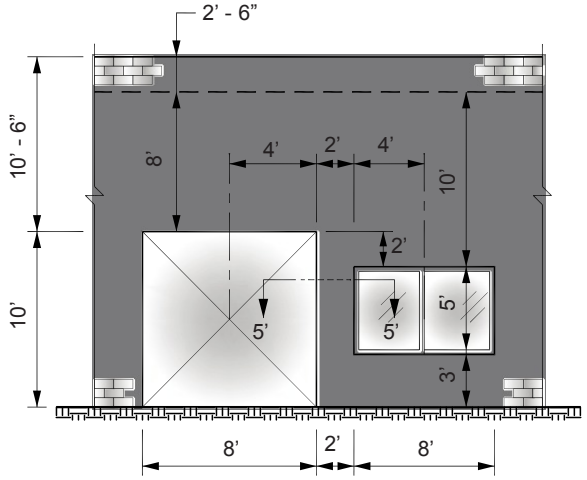


FIGURE 11.13 Detail elevation of west wall.

Compression Limiting

Load combination Equation 16-4 will govern:

$$1.2D + 1.6W + 0.5L_r$$

P = (1.2)(10,450) +(0.5)(2,400) = 13,740 lbs (Assume no snow load).

M = (1.6)(6,470) = 10,350 lb-ft

Estimate the area of the reinforcement. A good design would be (2) No. 5 bars at each jamb. Try (4) No. 5.

A_s = 1.24 in.^{2}

The wall is cracked. Thus:

$$k = \frac{(A_s F_y + P)}{(0.8)(0.8)(bdf'_m)}$$

$$= \frac{(1.24)(60{,}000) + 13{,}740}{(0.8)(0.8)(24)(3.8125)(1800)}$$

$$= 0.84$$

And the moment is:

$$M_u = A_s F_y\left(1 - \frac{0.8k}{2}\right)d + P\left(1 - \frac{0.8k}{2}\right)d$$

$$M_u = (1.24)(60{,}000)\left(1 - \frac{(0.8)(0.84)}{2}\right)(3.8125)$$

$$+ 13{,}740\left(1 - \frac{(0.8)(0.84)}{2}\right)(3.8125)$$

= 223,100 lb in.

$$\phi M_n = \frac{(0.9)(223{,}100)}{12{,}000} = 16{,}700 \text{ lb - ft} \quad \text{OK}$$

Tension Limiting

Check load combination 16-6 for required reinforcement:

$P_u = 0.9D$ = (0.9)(10,450) = 9,405 lbs

M_u = (1.6)(6,470) = 10,352 lb-ft

Continue with the (4) No. 5 bars:

$$k = \frac{(A_s F_y + P)}{(0.8)(0.8)(bdf'_m)}$$

$$= \frac{(1.24)(60{,}000) + 9{,}405}{(0.8)(0.8)(7.625)(12)(1800)} = 0.80$$

$$M_u = A_s F_y\left(1 - \frac{0.8k}{2}\right)d + P\left(1 - \frac{0.8k}{2}\right)d$$

$$M_u = (1.24)(60{,}000)\left(1 - \frac{(0.8)(0.8)}{2}\right)(3.8125)$$

$$+ (9{,}405)\left(1 - \frac{(0.8)(0.8)}{2}\right)(3.8125)$$

= 217,300 lb in.

$$\phi M_u = \frac{(0.9)(217{,}300)}{12{,}000} = 16{,}300 \text{ lb - ft} \quad \text{OK}$$

Limits on Reinforcement

MSJC Code Section 3.3.5 limits the amount of reinforcement allowed in the wall.

The following equation defines the limit on the reinforcement.

$$A_s = \rho bd \frac{0.64bdf'_m \dfrac{\varepsilon_{mo}}{\left(\varepsilon_{mo} + \alpha\varepsilon_y\right)} - P}{F_y}$$

α = 1.5 MSJC Code Section 3.3.3.5.1

MSJC Code Section 3.3.3.5.1(d) defines the amount of axial load to use in the reinforcement limiting equation as:

$P = D + 0.75L + 0.525Q_E$

= 10,450 + (0.75)(2,400)

= 12,250 lbs

This axial load is an attempt to define a service load condition at 75% of expected loads.

$$A_s = \frac{(0.64)(24)(3.8125)(1800)\dfrac{0.0025}{\left(0.0025 + 1.5\dfrac{60{,}000}{29{,}000{,}000}\right)} - 12{,}250}{60{,}000}$$

= 0.57 in.^{2}

This is less reinforcement assumed in design. At 0.57 in. of reinforcement, there is not enough reinforcement to resist the moment.

The solution is to make the pier a pilaster or a column. The easiest method is to make the wall 16 in. thick at this point. Place the No. 5 bars in the corners and tie the bars in accordance with the column tie requirement.

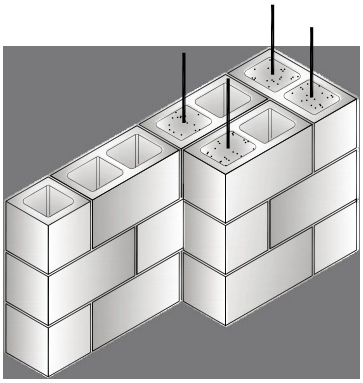
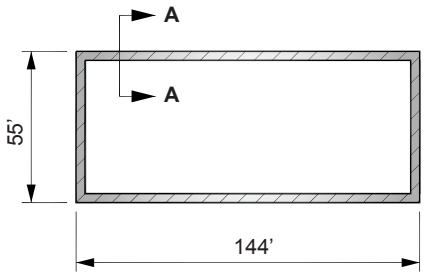


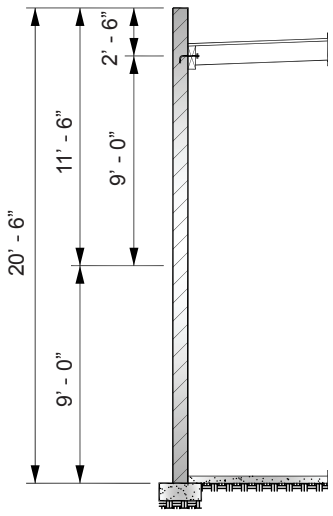
FIGURE 11.14 Pilaster/Pier

This member is not a column. It is a pilaster because it is part of a wall. The MSJC code has no provisions for pilasters in strength design. The definition of a column is "An isolated member whose horizontal dimension measured at right angles to its thickness does not exceed three times its thickness and whose height is greater than 4 times its thickness". Because the door opening is only 8 ft and subject to impact, it is a good idea to provide ties as though it is a column. Provide No. 4 ties at 8 in. on center.

11.7 WIND AND SEISMIC FORCES ON TOTAL BUILDING

By inspection, it is apparent that the east and west walls will have the highest shear stresses since they are relatively short but receive large loads from a N-S earthquake force. Therefore, check the shear capacity of these walls.





Section A-A

FIGURE 11.15 Plan of building and section thru wall.

11.7.1 LOADS

Load in North-South Direction

Wind:

$$\frac{(16.7)(20.5)^2}{(2)(18)} = 195 \text{ psf}$$

Seismic:

Weight of structure acting on roof diaphragm

= 2(1/2 wall ht. + parapet) + roof dead load

= 2(1/2 x 18 + 2.5) (78 psf) + (14 psf x 55 ft)

= 1794 + 770 = 2564 plf

Using the equivalent lateral force method assume, C_s equals 0.102.

$V = 0.102W$

$V = (0.102)(2564) = 262$ plf on the building

Seismic forces govern over wind forces

262 plf > 195 plf (or lb/ft), and

The roof diaphragm acts as horizontal beam with the end shear walls serving as the reactions. The ledger and bond beams act as the flanges that resist the tension and compression forces in this beam.

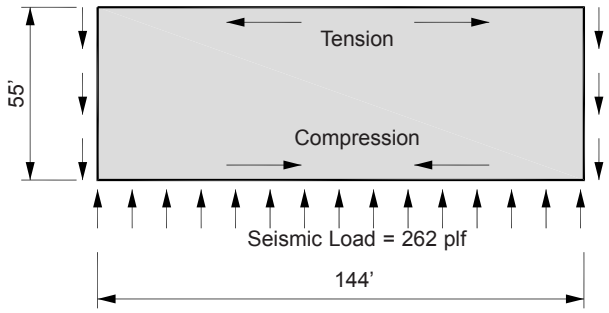


FIGURE 11.16 Seismic load to roof diaphragm.

$$\text{Moment} = \frac{wl^2}{8} = \frac{(262)(144)^2}{8}$$

$$= 679{,}104 \text{ ft lbs}$$

$$\text{Flange stress} = \frac{M}{d} = \frac{679{,}104}{(55 - 0.25)} = 12{,}404 \text{ lbs}$$

Chord steel or tension steel in bond beam

$$A_s = \frac{T}{f_s} = \frac{12{,}404}{(24{,}000)\left(\frac{4}{3}\right)} = 0.39 \text{ sq in.}$$

Use 1 - #6 or 2 - #4 bars in bond beam at ledger

Seismic force to end transverse shear walls

$$\text{Seismic force} = (262)\frac{144}{2} = 18{,}864 \text{ lbs}$$

Seismic shear per foot on transverse shear walls at roof ledger

$$= \frac{18{,}864}{55} = 343 \text{ plf}$$

Additional seismic force due to weight of end walls. For simplicity assume no openings in the walls.

Weight of walls = 20.5 x 78 psf = 1599 plf

Seismic force = 0.102 x 1599 = 163 plf

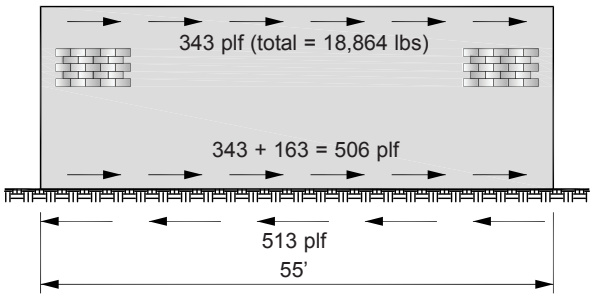


FIGURE 11.17 Seismic shear on wall and foundation.

Shear per linear foot at connection between wall and footing

343 + 163 = 506 plf

Shear stress at the base of the wall:

$$V = \frac{506}{(7.625)(12)} = 5.5 \text{ psi}$$

Shear friction and steel dowels will resist this stress.

11.7.2 LEDGER BOLT AND LEDGER BEAM DESIGN

Ledger bolts and joist anchors tie the roof and/or floor diaphragms to the masonry walls. They transmit the vertical and lateral shear loads from the roof and/or floor system to the load bearing, shear resisting wall elements. IBC Section 1604.8.2 requires a minimum of 280 lb/ft or the calculated amounts. ASCE 7 Section 12.11.1 requires the force to be a minimum of the following:

1. $0.4S_{DS}IW$ or (0.4)(0.33)(1.0)(78) = 10.3 psf. Proportioning from the wind load reaction force results in a force of [10.3/16.7] (198) = 122 lb/ft
2. (400) $S_{DS}I$ = (400) (0.33)(1.0) = 132 lb/ft
3. 280 lb/ft

Section 12.11.2.1 of ASCE 7 for flexible diaphragms requires a force of $F_p = 0.8S_{DS}IW = 20.6$ psf. Again proportioning from the wind load results in a force of [20.6/16.7] (198) = 245 lb/ft

It appears the 280 lb/ft controls

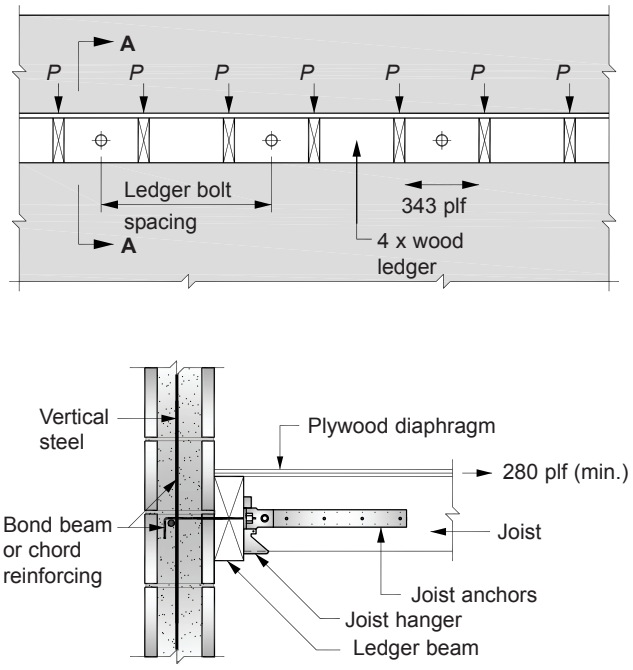


FIGURE 11.18 Connection of ledger beam to wall.

The diaphragm must be connected in a positive manner by means of joist anchors to the wall. The consideration of cross grain tension on the ledger to resist forces perpendicular to the wall is not permitted.

Joist anchors (straps) must be spaced 48 in. o.c. to resist 280 plf x 4 ft = 1120 lbs. These anchors must be properly fastened by means of bolts or lag screws into the joists.

Shear forces parallel to the shear wall are transmitted from the wood diaphragm to the ledger beam by means of proper nailing. The ledger beam then transmits the shear force to the wall through the ledger bolts.

Ledger bolts

E (shear parallel to wall) = 343 plf

D (vertical) = 11 psf x 12 ft = 132 plf

The controlling load combination is IBC Section 16-4 (modified), $1.2D + 1.0E + 0.5L_r$

Vertical load = 1.2 x 132 + 0.5 x [12 x 20] = 278 lbs

Horizontal Force = 343 lbs

MSJC Code Section 3.1.6 contains equations for allowable shear in headed bolts.

$$B_{vn} = 4A_{pv}\sqrt{f'_m} \text{ or } B_{vn} = 0.6A_bf_y$$

$$A_{pv} = \frac{\pi l_{be}^2}{2}$$

The term l_{be} is the distance to the edge of the masonry and is large in this case. Assume 3/4 in. diameter bolts at 48 in. on center. Use 33,000 yield strength for an assumed A7 bolt. The shear in the bolt is the limiting amount and equal to:

B_{vn} = 0.6 x 0.44 x 33,000 = 8,700 lbs

The value is too high. Strength design does not provide for failure of a bolt in shear due to bearing on the masonry. This is often the limiting condition. Resorting to Chapter 2, allowable stress design provides the following equations:

$$B_v = 350\sqrt[4]{f'_m A_b} \text{ or } B_v = 0.12A_bf_y$$

$$B_v = 350\sqrt[4]{(1800)(0.44)} = 1856 \text{ lbs or}$$

$$B_v = (0.12)(44)(33{,}000) = 1742 \text{ lbs}$$

The net unfactored bolt shear is:

$$V = \sqrt{343^2 + 278^2} = 442 \text{ lbs OK}$$

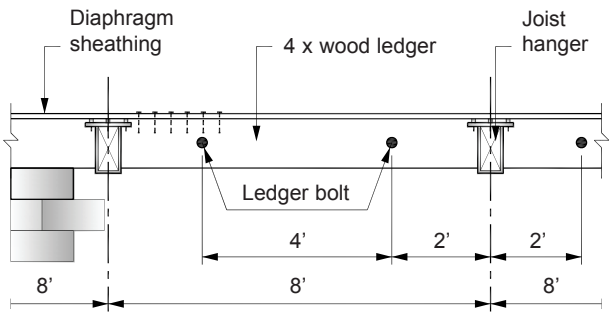


FIGURE 11.19 Bolt pattern in ledger.

The wood bearing stress must also be checked to assure the members are adequate.

11.8 DISTRIBUTION OF SHEAR FORCE IN END WALLS

The lateral forces on a building are distributed by the flexible wood diaphragm to the shear resisting walls. These walls carry the forces to the foundation and if the wall has openings due to doors and windows, the piers are subjected to shear forces in proportion to their respective rigidities. Rigidity of a pier is inversely proportional to its flexibility and deflection. This deflection is made up of both moment and shear deflection for cantilever and fixed piers.

Total shear and moment deflection for a cantilever pier is:

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

Total shear and moment deflection for a fixed pier is:

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

The modulus of elasticity of masonry is based on the value 1,000,000 psi. Table ASD-89a-g gives values per inch of thickness for Δ_F; Δ_C; $1/\Delta_F$ and $1/\Delta_C$ for various h/d ratios to facilitate determining the deflection and rigidity of piers in a wall. The tables (although labeled ASD) also apply to the distribution of lateral loads for strength design distribution. Adjustment of the moment of inertia is usually made and it is common to reduce the moment to 80% of the uncracked section.

If the unit shear stress exceeds the capacity of the masonry with no shear reinforcement, the pier must be reinforced and the horizontal shear reinforcement must be capable of resisting the loads in excess of the masonry capacity.

Seismic lateral force at top of wall from ledger = 18,864 lbs.

Calculate the seismic force from the weight of the wall.

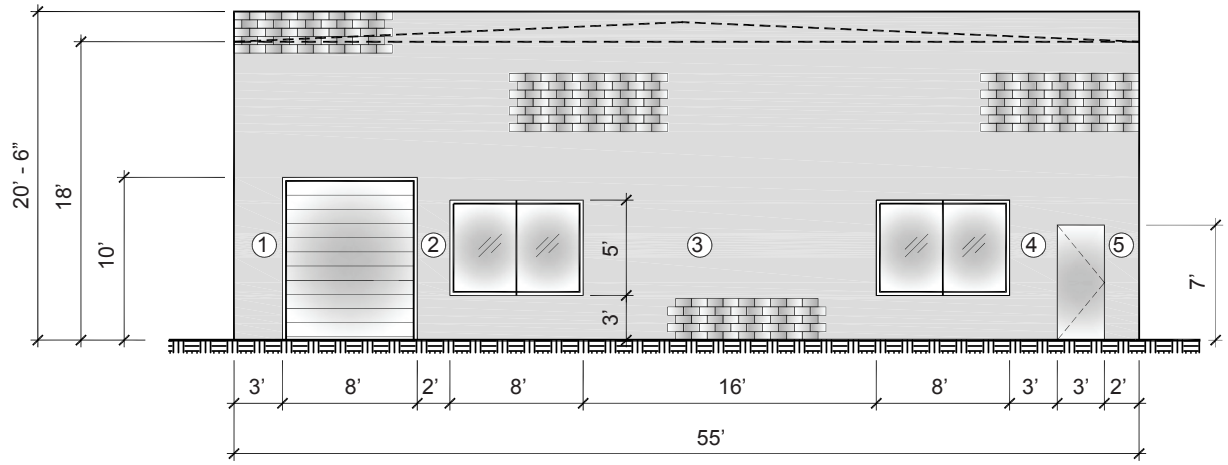


FIGURE 11.20 West elevation.

TABLE 11.2 Relative Rigidities of Piers – West Wall

Pier No.	Height *h* (ft)	Length *l* (ft)	*h*/*l* Ratio (all piers fixed)	Relative Rigidity Table ASD-89 Fixed Piers[2]	Percentage Lateral Force to Each Pier	Force *V* to each Pier (pounds)	Unit Shear f_v, in each pier $= \frac{V}{tl}$ (psi)
1	10	3	3.33	0.213	1.5	353	1.1
2	5	2	2.50	0.75	5.0	1,172	5.5
3	5	18	0.28	11.602	81.2	19,050	9.9
4	4	3	1.33	1.577	11.0	2,580	8.0
5	7	2	3.50	0.187	1.3	306	1.4
				$\Sigma = 14.329$	100%	$\Sigma = 23{,}460$ pounds[1]	

1. It would be conservative and quite usual to use the base shear, V = 26,700 lbs as the force on the wall and distribute this amount to all piers. The approach here is a little more detailed.

2. In Section 11-6 pier number 2 was increased in thickness to 15.626 inches. The stiffness is increased in the above table accordingly; 0.432 x 15.625/9 = 0.75.

Dead weight of wall, w

w = 78 [(20.5)(55) - (10)(8) - (2)(5)(8) - (7)(3)]

= 78[946.5] = 73,827 lbs

V_{wall} = 0.102(73,827) = 7530 lbs

Total base shear at the foundation = 18,864 + 7530 = 26,400 lbs

Seismic lateral force at 8 ft above the foundation

V_{wall} = 18,864 + (0.102) [(78)(10.5)(55)] = 23,460 lbs

The shear stresses in the piers (see Table 11-1) do not exceed the capacity for the masonry from Table ASD-5 and Diagram ASD-5 and only require nominal minimum reinforcing steel. It is good practice to place 2 - #4 bars or 1 - #5 bar on each side, above and below all openings.

11.8.1 DESIGN OF SHEAR REINFORCEMENT IN PIERS 3 AND 4

Pier 3

Calculate the shear force in the wall:

The capacity of the masonry without reinforcement is:

$$V_m = \left[4.0 - 1.75\frac{M_u}{V_u d_v}\right] A_n \sqrt{f'_m} + 0.25P_n \le 4A_n\sqrt{f'_m}$$

$$V_m = [4.0 - (1.75)(1.0)](7.625)(18)(12)\sqrt{1800} + (0.25)(78)(10.5) = 157{,}400 \text{ lbs}$$

Or

$$V_m \le (4)(7.625)(18)(12)\sqrt{1800} = 279{,}500 \text{ lbs}$$

ϕV_m = (0.8)(157,400) = 125,920 lbs

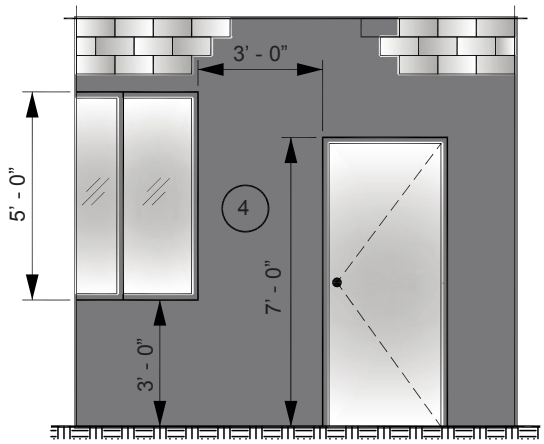
There is adequate capacity to resist the loads without adding shear reinforcement.

Analysis of Pier 4 is similar.

11.9 QUESTIONS AND PROBLEMS

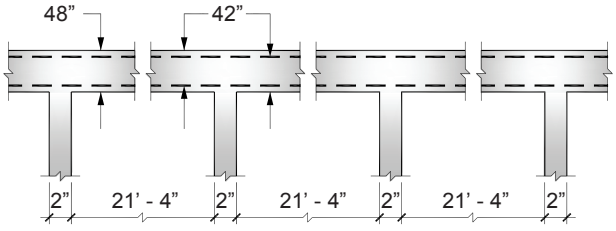
11-1 Determine the required reinforcing steel for the shear and overturning forces on pier 4 of the West wall of the industrial building in this Section.

The lateral force at the top of the pier is 15 kips, vertical dead load is 20 kips. f'_m = 2000 psi and f_y = 60,000 psi.

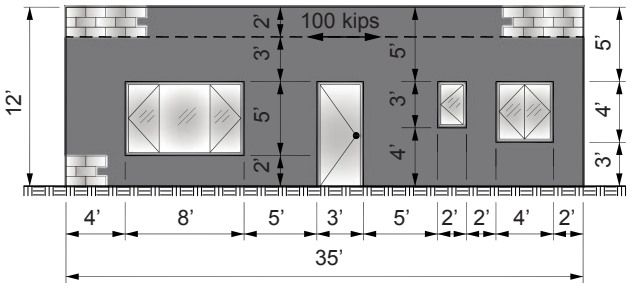


11-2 Design and detail the flexural and shear reinforcing steel for the solid grouted continuous masonry beam shown below.

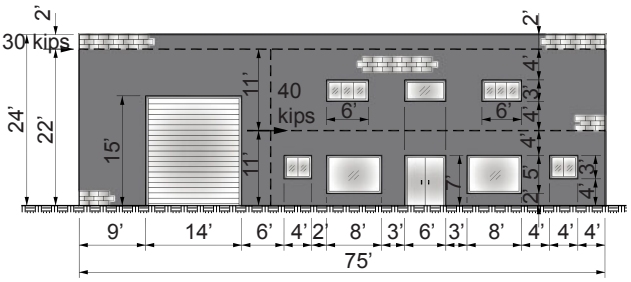
The 8 in. concrete masonry units have a strength of 2800 psi and Type S mortar will be used. Determine f'_m from Table 2.2B. Assume the wind load is 20 psf and the structure is located in Seismic Design Category E. Vertical *LL* = 1400 plf, *DL* = 600 plf.



11-3 Determine the shear stress in each 8 in. thick grouted CMU piers. If f'_m = 2000 psi. Will shear reinforcing steel be necessary in any of the piers?



11-4 Design the piers for the end wall shown below if f'_m = 1500 psi and Grade 60 reinforcing steel is used. Distribute the lateral force to each of the piers in relation to their rigidity. The lateral force is 30 kips at the roof level and 40 kips at the second floor level. Note the lateral force for the piers on the first floor must resist the total lateral force.



CHAPTER 12

DESIGN OF SEVEN–STORY MASONRY LOAD BEARING WALL APARTMENT BUILDING

12.1 GENERAL

In this example problem, the dead, live and seismic loads for two walls in the seven-story building will be calculated. The walls will then be designed for the imposed axial, shear and overturning forces. The design will be based on the 2006 IBC and TMS 402-05, Building Code Requirements for Masonry Structures.

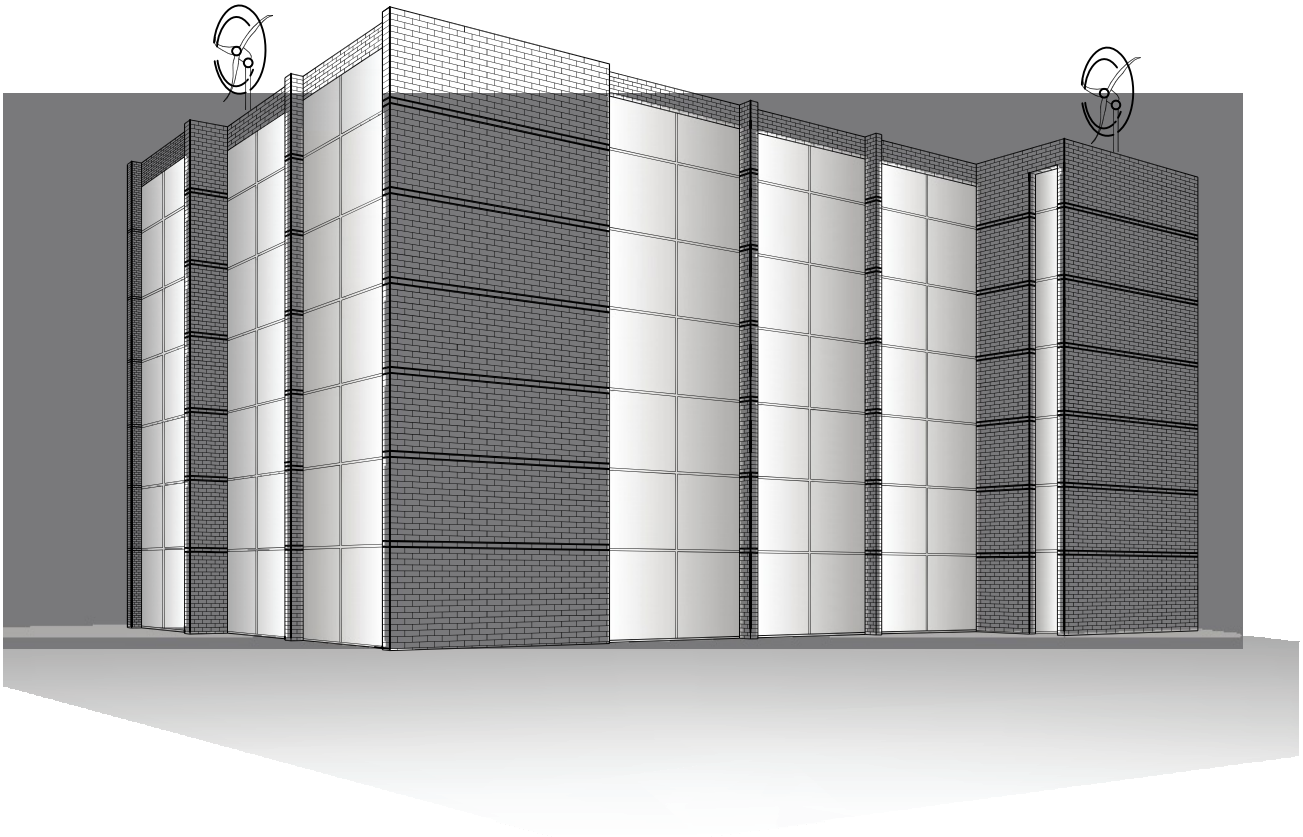


FIGURE 12.1 **Seven-story apartment building.**

FIGURE 12.2 Typical architectural floor plan.

12.1.1 DESIGN CRITERIA, ELEVATION AND PLAN

The building is located in a high seismic region (SDC D). From the 2006 IBC seismic maps (1613.5) the values for S_S and S_1 are 1.5 and 0.4 respectively and ρ = 1.0. The occupancy category is II and the soil classification is D. The wind loading is assumed to be less than the seismic loading and is not controlling the design, but will be checked. The building is 71 feet tall, basic wind speed (3 second gust) is 90 MPH, exposure B, importance factor of 1.0 with an internal pressure coefficient GC_{pi} = 0.18.

12.1.2 FLOOR AND ROOF SYSTEMS

A 6 in. precast prestressed concrete plank with a 2 in. concrete topping is selected for the floor system. This system weighs 73 psf according to manufacturers technical information. The roof system is 6 in. precast prestressed concrete plank, but without the topping. It weighs 48 psf.

Floor Loads:		**Roof Loads:**	
Slab, 8"	= 73 psf	Slabs, 6"	= 48 psf
Misc.	= 5 psf	Roofing	= 5.5 psf
(Piping electrical and other Misc.)		(ASCE 7, Bituminous Gravel)	
Dead load	= 78 psf	Dead load	= 53.5 psf

Live Loads:

40 psf Apartments	20 psf Roof
100 psf Corridor	Snow load not considered

15 psf Partition (ASCE 7, Section 4.2.2)

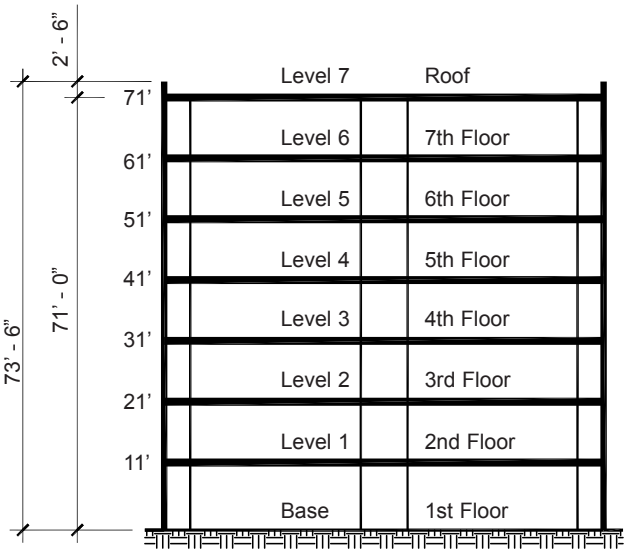


FIGURE 12.3 Transverse cross-section.

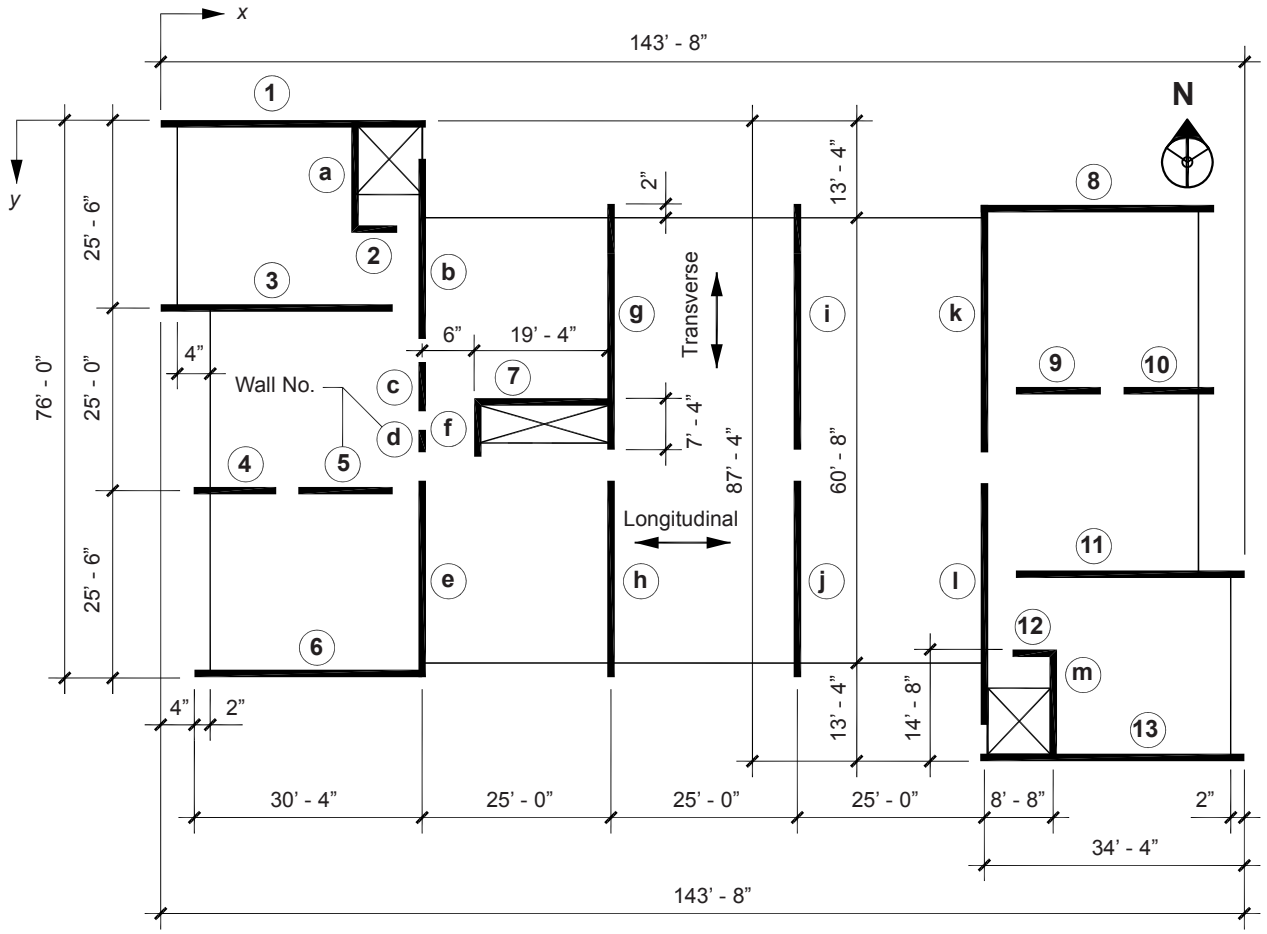


FIGURE 12.4 Typical structural floor plan.

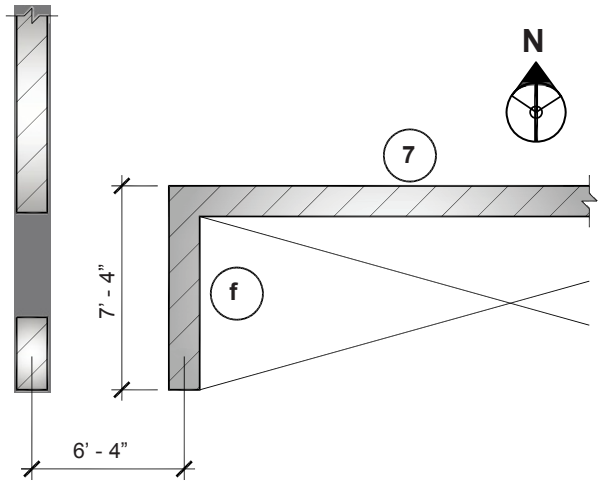


FIGURE 12.5 Wall f.

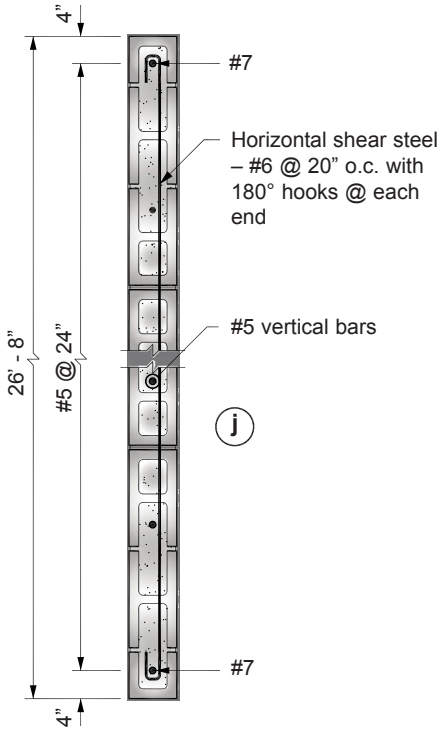


FIGURE 12.6 Detail of reinforcing steel in wall j at first floor, base level.

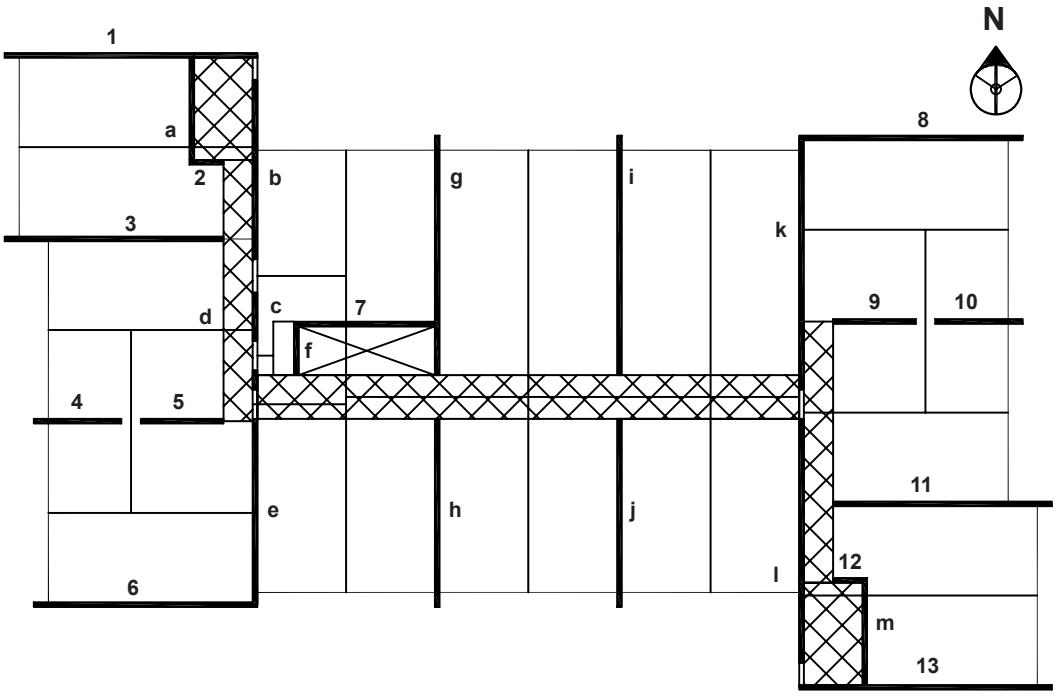


FIGURE 12.7 Floor load map.

12.1.3 STRUCTURAL WALL SYSTEM

8 in. medium weight concrete block masonry units (105 to 125 pcf) – 8 in. x 8 in. x 16 in. units or 8 in. hollow clay brick.

Wall weight = 78 psf (See Table GN-3a, Weight of Walls, assumes grouted solid).

Masonry walls are to be solid grouted for maximum STC (Sound Transmission Coefficient) values and fire ratings.

The structure is located in SDC D. The inspection level is determined as Level 2, therefore the inspection requirements of IBC Table 1704.5.3 apply.

Values of f'_m, specified strength of masonry, are to be determined based on the magnitude of vertical and lateral forces.

12.1.4 DEAD AND LIVE LOADS ON THE MASONRY WALLS

The dead and live loads on the walls are typically determined by the tributary area method. It is assumed that splitting the distances between supporting elements can approximate the loading applied to the wall. Some judgment and understanding of the structural system is required to do this properly. In this example, the precast planks span the typical 25 ft between supporting walls. Walls perpendicular to this do not support significant floor weight. The following figure shows the breakup of the tributary areas for the typical floor. The hatched area designates the 100 psf corridor floor live load areas.

The calculation of dead and live loading using tributary area can take many forms. The following tables for walls **j**, **f** and total for the building follow:

For wall j:

TABLE 12.1 Gravity Loads on Wall j

Load Criteria	Design Factor
Roof Dead Load	53.5 psf
Floor Dead Load	78.0 psf
Wall Dead Load[1]	78.0 psf
Partition	15.0 psf
Roof Live Load	20.0 psf
Floor Live Load - Apartments	40.0 psf
Floor Live Load - Corridors	100.0 psf
Tributary Area - Apartments	580.2 sq ft
Tributary Area - Corridors	75.0 sq ft
Average Floor LL[2]	46.9 lb/sf
Wall Length	25.8 ft
Maximum LL Reduction	60.0 Percent
Live Load Reduction Methodology	IBC Section 1607.9.2

[1] "Wall Dead Load" is applied to the vertical wall area, not the horizontal floor area, unlike the other loads provided on this table.

[2] The Average Floor LL is calculated [(580.2)(40) + (75) (100)]/ - (580.2 + 75)

TABLE 12.2 Gravity Load Distribution for Wall j

FLOOR	Trib Area	Wall Height	Floor D	Wall D	Live L	Sum Area	LL Reduction	Sum D	Sum L
R	655.2		35.1		13.1	655.2	40.0		
		10.0		20.1				55.2	7.9
7	655.2		60.9		30.7	655.2	40.4		
		10.0		20.1				136.3	26.2
6	655.2		60.9		30.7	1310.4	60.0		
		10.0		20.1				217.4	32.4
5	655.2		60.9		30.7	1965.6	60.0		
		10.0		20.1				298.4	44.7
4	655.2		60.9		30.7	2620.8	60.0		
		10.0		20.1				379.5	57.0
3	655.2		60.9		30.7	3276.0	60.0		
		10.0		20.1				460.6	69.3
2	655.2		60.9		30.7	3931.2	60.0		
		11.0		22.2				543.7	81.6
Ground									

For wall f:

TABLE 12.3 Gravity Loads on Wall f

Load Criteria	Design Factor
Roof Dead Load	53.5 psf
Floor Dead Load	78.0 psf
Wall Dead Load[1]	78.0 psf
Partition	15.0 psf
Roof Live Load	20.0 psf
Floor Live Load - Apartments	40.0 psf
Floor Live Load - Corridors	100.0 psf
Penthouse DL	4.7 K
Tributary Area - Apartments	20.7 sq ft
Tributary Area - Corridors	0.0 sq ft
Average Floor LL	40.0 lb/sf
Wall Length	7.3 ft
Maximum LL Reduction	60.0 Percent
Live Load Reduction Methodology	IBC Section 1607.9.2

[1] "Wall Dead Load" is applied to the vertical wall area, not the horizontal floor area, unlike the other loads provided on this table.

TABLE 12.4 Gravity Load Distribution for Wall f

FLOOR	Trib Area	Wall Height	Floor D	Wall D	Live L	Sum Area	LL Reduction	Sum D	Sum L
R	20.7		1.1		0.4	20.7	0.0		
		10.0		10.5				11.6	0.4
7	20.7		1.9		0.8	20.7	0		
		10.0		5.7				19.2	1.3
6	20.7		1.9		0.8	41.4	0		
		10.0		5.7				26.8	2.2
5	20.7		1.9		0.8	62.1	0		
		10.0		5.7				34.5	3.1
4	20.7		1.9		0.8	82.8	0		
		10.0		5.7				42.1	3.9
3	20.7		1.9		0.8	103.5	0		
		10.0		5.7				49.8	4.7
2	20.7		1.9		0.8	124.2	0		
		11.0		6.3				58.0	5.5
Ground									

The gravity load of the entire building is required to determine the building seismic forces. The tables for walls **j** and **f** can be added to similar tables for all the walls, adding weights for the penthouse and other miscellaneous results in the following dead loads for the entire building.

TABLE 12.5 Gravity Loads on Building

Load Criteria	Design Factor
Roof Dead Load	53.5 psf
Floor Dead Load	78.0 psf
Wall Dead Load[1]	78.0 psf
Partition	15.0 psf
Roof Live Load	20.0 psf
Floor Live Load - Apartments	40.0 psf
Floor Live Load - Corridors	100.0 psf
Tributary Area - Apartments	7666.0 sq ft
Tributary Area - Corridors	961.0 sq ft
Average Floor LL[2]	46.7 lb/sf
Wall Length	545.0 ft
Maximum LL Reduction	60.0 Percent
Live Load Reduction Methodology	IBC Section 1607.9.2

1. "Wall Dead Load" is applied to the vertical wall area, not the horizontal floor area, unlike the other loads provided on this table.

2. The Average Floor LL is calculated [(7666)(40) + (961) (100)]/ (7666 + 961)

TABLE 12.6 Gravity Load Distribution for Building

FLOOR	Trib Area	Wall Height	Floor D	Wall D	Live L	Sum Area	LL Reduction	Sum D	Sum L
R	8627		462		173	8627			
		10		533				994	125
7	8627		802		402	8627			
		10		425				2221	402
6	8627		802		402	17253			
		10		425				3448	514
5	8627		802		402	25880			
		10		425				4676	632
4	8627		802		402	34507			
		10		425				5903	788
3	8627		802		402	43134			
		10		425				7130	948
2	8627		802		402	51760			
		11		467				8399	1108
Ground									

12.1.5 SEISMIC LOADING

Based on 2006 IBC (ASCE 7 2005)

In recent years, the seismic design of even simple structures has become complex. There are many steps:

Step 1 Seismic Design Category:

1. Adjust seismic Maximum Considered Earthquake (MCE) coefficients for soil, ASCE 7 Tables 11.4-1 and 11.4-2.

Since the building is on site classification D.

$$S_{MS} = F_a S_s = (1.0)(1.5) = 1.50$$

$$S_{M1} = F_v S_1 = (1.6)(0.4) = 0.64$$

2. Spectral Design Parameters

The design parameters are 2/3 of the values above.

$$S_{DS} = (2/3)(1.5) = 1.00$$

$$S_{D1} = (2/3)(0.64) = 0.43$$

3. Importance Factor

Since the occupancy is II, the importance factor is 1.0 (ASCE 7 Table 11.5-1)

4. Seismic Design Category

From ASCE 7 Tables 11.6-1 and 11.6-2 the building is Seismic Design Category D.

Step 2 Detailing Requirements, Response Modification Factor; System Overstrength Factor; Deflection Amplification Factor and Structural System Limitations:

Table 12.2-1 of ASCE-7 line A.7, refers to detailing requirements in ASCE 7 Sections 14.4 and 14.4.3. The detailing requirements: ASCE 7 Section 14.4 and 14.4.3, modify the Masonry Code (MSJC) and they may or may not override the provision of MSJC Code. In some cases the IBC has different provisions than ASCE 7 and in these cases the IBC overrides the provisions of ASCE 7.

IBC Section 2106.5.2

2106.5.2 Shear wall shear strength. For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.

For all cross sections within a region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall, the nominal shear strength shall be determined by Equation 21-1.

$$V_n = A_n \rho_n f_y \qquad \textbf{(Equation 21-1)}$$

The required shear strength for this region shall be calculated at a distance L_w /2 above the base of the shear wall, but not to exceed one-half story height.

For the other region, the nominal shear strength of the shear wall shall be determined from Section 2108.

1. ASCE 7 Modification to Chapter 1 of the MSJC Code:

   ASCE 7 changes the width of the flange considered effective in resisting overturning from 6 times the flange thickness or the actual flange length for both compression and tension, to the same for compression but for tension the width is 0.75 times the floor to floor wall height or the actual, which ever is less.

2. ASCE 7 Modification to Chapter 2 of the MSJC Code:

   a. One Third Stress Increase

   This modification relates to the 1/3 stress increase for load combinations containing wind or seismic loading when using the ASCE 7 allowable stress design load combinations. There is an alternative set of allowable stress design load combinations contained in the IBC. We will use the IBC alternate allowable stress design load combinations, which allow the 1/3 stress increase. ASCE 7 allowable stress load combinations are not appropriate for this type of building because of the 0.6 factor applied to dead load. As of this publication, there is movement within ASCE 7 to align the requirements more closely to those in IBC.

   b. Bar Size

   For ASD (MSJC Code Chapter 2) reinforcement must be smaller than a No. 9 bar or 1/8 of the nominal wall thickness. We intend to use 8 in. CMU or 8 in. brick, so the maximum bar size is a No. 8.

c. Maximum Reinforcement

For ASD (MSJC Code Chapter 2) the maximum flexural reinforcement permitted in shear walls is one half the balanced reinforcement ratio.

3. ASCE 7 Modifications to Chapters 2 and 3 of the MSJC Code:

For ASD and SD (MSJC Code Chapter 2 and 3) lap slices, weld splices and mechanical type 1 splices are not allowed in a plastic hinge zone. This prohibition on lap splices applies to both horizontal and vertical reinforcement. For this building splices will be required to transfer loads from the wall reinforcement to the foundation reinforcement. Only Type 2 mechanical connectors can be used in the plastic hinge region. An example of this splice type is shown in Figure 12.8.

FIGURE 12.8 Mechanical coupler connection.

4. ASCE 7 Modifications to Chapter 3 of the MSJC Code:

For SD (MSJC Code Chapter 3) shear keys between the foundation and the walls may be required.

5. Detailing requirements: IBC 2006

For ASD and SD (Chapter 2 and 3) lap slices do not need to conform to MSJC Code Equation (2-9) or Equation (3-15). Alternative equations are provided.

From ASCE 7 Table 12.2-1

The Response Modification Factor of Special Reinforced Masonry Shear Walls is 5.0.

The System Overstrength Factor is 2.5.

The Deflection Amplification Factor is 3.5.

The system is limited to a height of 160 ft in Seismic Design Category D.

Step 3 Run a Computer Model of the Structure:

There are many computer programs available to determine the loading on the elements of the structure. A model for E-TABS was created and run. The model is a dynamic model combining the many modes of the building response. The responses (moments, shear, and axial loads) to each of the modes are combined by using the square root of the sum of the squares. There are two consequences of combining the model effects that need to be understood in interpreting the resulting output. First, all the output has a positive sign, as any negative sign is lost when the responses are squared. Thus all results need to be considered as possibly having a positive or a negative sign. Second, the output is not statically consistent because the effects of the different modes on the shear and moment response will be somewhat different.

Output included shear, moment and axial loads on each wall at each floor. The model was run for the seismic elastic response producing a base shear of 7,407 kips in the East-West direction and 7,299 kips in the North-South Direction. These values represent the elastic response and need to be scaled for inelastic response. The scaling factor can be determined using the equivalent lateral force method of ASCE 7 Section 12.8.

S_{D1} = 0.4 and S_{DS} = 1.0

The period may be taken as:

$T = C_t h_n^x$ ASCE 7 Eq 12.8-7

C_t = 0.02 ASCE 7 Table 12.8-2

H_n = 71 ft

x = 0.75 ASCE 7 Table 12.8-2

T = 0.49 seconds

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{1.0}{\left(\frac{5}{1}\right)} = 0.2$$ ASCE 7 Eq 12.8-2

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.43}{0.49\left(\frac{5}{1}\right)} = 0.175$$

TABLE 12.7 E-TABS Output

Level	Wall	Load	Loc	Axial Load P	V2 (kips)	V3 (kips)	T (kips)	M2 (kips)	M3 (kip-in.)
STORY 7	J	ATEQX	Top	0	1.4	0.0	26.3	0	0
STORY 7	J	ATEQX	Bottom	0	1.4	0.0	26.3	1	167
STORY 7	J	ATEQY	Top	0	2.4	0.0	44.0	0	0
STORY 7	J	ATEQY	Bottom	0	2.4	0.0	44.0	1	280
STORY 7	J	X	Top	0	39.7	0.2	58.0	0	0
STORY 7	J	X	Bottom	0	39.7	0.2	58.0	23	4766
STORY 7	J	Y	Top	0	53.7	0.1	91.1	0	0
STORY 7	J	Y	Bottom	0	53.7	0.1	91.1	16	6445
STORY 6	J	ATEQX	Top	0	4.9	0.0	27.7	1	167
STORY 6	J	ATEQX	Bottom	0	4.9	0.0	27.7	-1	758
STORY 6	J	ATEQY	Top	0	8.3	0.0	46.3	1	280
STORY 6	J	ATEQY	Bottom	0	8.3	0.0	46.3	-2	1271
STORY 6	J	X	Top	0	124.5	0.2	62.1	23	4766
STORY 6	J	X	Bottom	0	124.5	0.2	62.1	15	18725
STORY 6	J	Y	Top	0	179.7	0.2	95.8	16	6445
STORY 6	J	Y	Bottom	0	179.7	0.2	95.8	8	26488
STORY 5	J	ATEQX	Top	0	7.1	0.0	26.8	-1	758
STORY 5	J	ATEQX	Bottom	0	7.1	0.0	26.8	-3	1611
STORY 5	J	ATEQY	Top	0	11.9	0.0	44.9	-2	1271
STORY 5	J	ATEQY	Bottom	0	11.9	0.0	44.9	-4	2700
STORY 5	J	X	Top	0	193.8	0.2	60.4	15	18725
STORY 5	J	X	Bottom	0	193.8	0.2	60.4	32	41490
STORY 5	J	Y	Top	0	273.7	0.1	93.4	8	26488
STORY 5	J	Y	Bottom	0	273.7	0.1	93.4	20	58044
STORY 4	J	ATEQX	Top	0	8.7	0.0	25.0	-3	1611
STORY 4	J	ATEQX	Bottom	0	8.7	0.0	25.0	-5	2656
STORY 4	J	ATEQY	Top	0	14.6	0.0	41.9	-4	2700
STORY 4	J	ATEQY	Bottom	0	14.6	0.0	41.9	-8	4451
STORY 4	J	X	Top	0	246.2	0.3	56.6	32	41490
STORY 4	J	X	Bottom	0	246.2	0.3	56.6	63	70568
STORY 4	J	Y	Top	0	347.5	0.2	87.2	20	58044
STORY 4	J	Y	Bottom	0	347.5	0.2	87.2	43	97951
STORY 3	J	ATEQX	Top	0	10.2	0.0	21.2	-5	2656
STORY 3	J	ATEQX	Bottom	0	10.2	0.0	21.2	-6	3886
STORY 3	J	ATEQY	Top	0	17.1	0.0	35.5	-8	4451
STORY 3	J	ATEQY	Bottom	0	17.1	0.0	35.5	-10	6508
STORY 3	J	X	Top	0	290.7	0.1	48.0	63	70568
STORY 3	J	X	Bottom	0	290.7	0.1	48.0	75	104889
STORY 3	J	Y	Top	0	418.0	0.1	73.9	43	97951
STORY 3	J	Y	Bottom	0	418.0	0.1	73.9	52	145717
STORY 2	J	ATEQX	Top	0	12.1	-0.1	17.7	-6	3886
STORY 2	J	ATEQX	Bottom	0	12.1	-0.1	17.7	-13	5334
STORY 2	J	ATEQY	Top	0	20.1	-0.1	29.5	-10	6508
STORY 2	J	ATEQY	Bottom	0	20.1	-0.1	29.5	-22	8927
STORY 2	J	X	Top	0	336.9	1.0	39.9	75	104889
STORY 2	J	X	Bottom	0	336.9	1.0	39.9	184	144646
STORY 2	J	Y	Top	0	498.9	0.7	61.3	52	145717
STORY 2	J	Y	Bottom	0	498.9	0.7	61.3	129	202843
STORY 1	J	ATEQX	Top	0	15.8	0.1	4.2	-13	5334
STORY 1	J	ATEQX	Bottom	0	15.8	0.1	4.2	0	7427
STORY 1	J	ATEQY	Top	0	26.5	0.2	7.0	-22	8927
STORY 1	J	ATEQY	Bottom	0	26.5	0.2	7.0	0	12420
STORY 1	J	X	Top	0	404.8	1.4	10.5	184	144646
STORY 1	J	X	Bottom	0	404.8	1.4	10.5	0	197227
STORY 1	J	Y	Top	0	614.6	1.0	15.3	129	202843
STORY 1	J	Y	Bottom	0	614.6	1.0	15.3	0	281360

Values from E-TABS analysis and results are not statically consistent because they use many models.

Using the weight from Table 12.6, the base shear is:

$V = (0.175)(8399) = 1470$ kip

The elastic base shear from the model was 7407 kip East-West and 7299 kip North-South. Thus the model scaling factors are:

East-West = 1470/7407 = 0.198

North-South = 1470/7299 = 0.201

These factors are used to scale the model results to the design levels that take into account inelastic response.

Step 4 Find the Wall Lateral Loads:

The bottom of the wall values are used in the load combinations. The following loads are extracted from the data. Note that these are still the elastic response of the structure and will need to be reduced for the inelastic response.

ATEQX are loads for accidental torsion from seismic forces in the "X" (E-W direction) and ATEQY are loads for accidental torsion from seismic forces (EQ) in the "Y" (N-S direction).

These are the basic loads required to design wall **j**.

ASCE 7 Section 12.5.3 for SDC C and above requires 30% of the orthogonal loading to be combined with the main direction loading for shear wall buildings.

Combining the orthogonal requirements and applying the scaling factors of the previous step results in the loadings for wall **j**. For example, the shear force at the base of wall **j** is the larger of:

V_y = (614.6)(0.201) + (0.30)(404.8)(0.198) + ABS(26.5)(0.201) = 152.9 kip

V_y = (404.8)(0.198) + (0.3)(614.6)(0.201) + ABS(15.8)(0.198) = 120.3 kip

And the moment is:

M_y = (281,360)(0.201) + (0.30)(197,227)(0.198) + ABS(12,420)(0.201) = 70,765 kip-in.

M_y = (197,227)(0.198) + (0.30)(281,360)(0.201) + ABS(7427)(0.198) = 57,487 kip-in.

TABLE 12.8 Primary Shears(kips)

Level	Load	Shear Y	Level	Load	Shear X
STORY 7	Y	53.7	STORY 7	X	49.7
STORY 6	Y	179.7	STORY 6	X	124.5
STORY 5	Y	273.7	STORY 5	X	193.8
STORY 4	Y	347.5	STORY 4	X	246.2
STORY 3	Y	418.0	STORY 3	X	290.7
STORY 2	Y	498.9	STORY 2	X	336.9
STORY 1	Y	614.6	STORY 1	X	404.8

TABLE 12.9 Shears from Accidental Torsion (kips)

Level	Load	Shear AY	Level	Load	Shear AX
STORY 7	ATEQY	2.4	STORY 7	ATEQX	1.4
STORY 6	ATEQY	8.3	STORY 6	ATEQX	4.9
STORY 5	ATEQY	11.9	STORY 5	ATEQX	7.1
STORY 4	ATEQY	14.6	STORY4	ATEQX	8.7
STORY 3	ATEQY	17.1	STORY 3	ATEQX	10.2
STORY 2	ATEQY	20.1	STORY 2	ATEQX	12.1
STORY 1	ATEQY	26.5	STORY 1	ATEQX	15.8

TABLE 12.10 Moment from Primary Shear (kip-in.)

Level	Load	Moment Y	Level	Load	Moment X
STORY 7	Y	6445	STORY7	X	4766
STORY 6	Y	26488	STORY6	X	18725
STORY 5	Y	58044	STORY5	X	41490
STORY 4	Y	97951	STORY4	X	70568
STORY 3	Y	145717	STORY3	X	104889
STORY 2	Y	202843	STORY2	X	144646
STORY 1	Y	281360	STORY1	X	197227

TABLE 12.11 Moment from Accidental Torsion (kip)

Level	Load	Shear AY	Level	Load	Shear AX
STORY 7	ATEQY	280	STORY7	ATEQX	167
STORY 6	ATEQY	1271	STORY6	ATEQX	758
STORY 5	ATEQY	2700	STORY5	ATEQX	1611
STORY 4	ATEQY	4451	STORY4	ATEQX	2656
STORY 3	ATEQY	6508	STORY3	ATEQX	3886
STORY 2	ATEQY	8927	STORY2	ATEQX	5334
STORY 1	ATEQY	12420	STORY1	ATEQX	7427

Wall **f** has axial load because it is connected to wall 7. Wall **j** has no axial loading because it is not connected to another wall. The coupling provided by the floor is not included in the model and is small.

TABLE 12.12 Seismic Loads on Wall j

	Shear N-S (Kip)	Shear E-W (Kip)	Moment N-S (Kip-in)	Moment E-W (Kip-in)	Axial N-S (Kip)	Axial E-W (Kip)
STORY 7	13.6	11.4	1635	1365	0	0
STORY 6	45.2	36.5	6692	5455	0	0
STORY 5	68.9	56.3	14674	12034	0	0
STORY 4	87.4	71.4	24775	20405	0	0
STORY 3	104.7	84.8	36828	30324	0	0
STORY 2	124.3	99.2	51158	41927	0	0
STORY 1	152.9	120.3	70765	57487	0	0

Using the same development process, values for wall **f** are contained in Table 12.13.

TABLE 12.13 Seismic Loads on Wall f

	Shear N-S (Kip)	Shear E-W (Kip)	Moment N-S (Kip-in)	Moment E-W (Kip-in)	Axial N-S (Kip)	Axial E-W (Kip)
STORY 7	5.4	6.2	166	204	12.0	13.2
STORY 6	10.7	11.0	370	364	37.4	40.7
STORY 5	14.1	14.9	598	538	72.8	79.4
STORY 4	16.7	17.8	836	714	115.4	126.4
STORY 3	18.5	20.0	1091	896	162.8	179.4
STORY 2	19.4	21.3	1321	1058	213.1	235.9
STORY 1	25.6	22.0	2688	1990	268.1	294.1

TABLE 12.14 Wind Loads

			MWFRS		BASE SHEAR	
Range	**Height**	**K_z[1]**	**Windward (psf)**	**Total Pressure (psf)**	**Height of Zone (ft)**	**Force (Kips)**
0-15	15	0.57	6.8	13.5	15	29
15-20	20	0.62	7.4	14.1	5	10
20-25	25	0.66	7.9	14.6	5	10
25-30	30	0.70	8.4	15.1	5	11
30-40	40	0.76	9.1	15.8	10	23
40-50	50	0.81	9.7	16.4	10	24
50-60	60	0.85	10.2	16.9	10	24
60-70	70	0.89	10.7	17.4	10	25
70-71	71	0.93	10.7	17.4	1	3
			Leeward pressure = -6.7 psf		Base Shear	159
			Sideward pressure = -9.4 psf			

1. ASCE 7-05 Table 6-3, Exposure B, Case 2

12.1.6 WIND DESIGN

The wind load provisions of ASCE 7 are as complex as the seismic provision. A comparison of the wind base shear to that of the seismic loading base shear will determine which loading conditions apply. This comparison will be based on wind loads applied to the broad face of the building as this has the largest surface area for wind. The wind speed is 90 mph and the exposure is "B". ASCE 7 method 2 will be used. The building conforms to the limitations of method 2.

The mean roof height is 71 ft.

V = 90 mph (from wind maps, ASCE 7-05, Figure 6-1)

The wind importance factor is 1.0.

It is an "enclosed" building.

The internal pressure coefficient GC_{pi} = 0.18

K_d = 0.85 (ASCE 7-05, Table 6-4)

K_{zt} = 1.0 (ASCE 7-05, Sec. 6.5.7.2)

K_h = 0.89 (ASCE 7-05, Table 6-3)

$q_z = (17.6)(K_z)$ psf

q_h = 15.8 psf

Clearly the seismic lateral forces will control the design.

12.2 DESIGN OF WALL "j" ON FIRST STORY, BASE LEVEL – ALLOWABLE STRESS DESIGN

Wall **j** carries vertical loads and in-plane seismic loads.

Masonry dimensions of the wall are:

b = 7.5 in. (Brick masonry is laid with a 1/2 in. mortar joint)

L = 25 ft - 10 in. = 310 in.

TABLE 12.15 Loads on Wall j

	Axial Load (Kip)	Shear Load (Kip)	Moment (Kip-in)
Dead Load	543.7	0	0
Live Load	81.6	0	0
Seismic Load (E-W)	0	120.4	57,487
Seismic Load (N-S)	0	152.9	70,765

12.2.1 LOAD COMBINATIONS

We will use the alternative load combinations contained in the 2006 IBC:

$D + L + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-16)**

$D + L + (\varpi W)$ **(Equation 16-17)**

$D + L + \varpi W + S/2$ **(Equation 16-18)**

$D + L + S + \varpi W/2$ **(Equation 16-19)**

$D + L + S + E/1.4$ **(Equation 16-20)**

$0.9D + E/1.4$ **(Equation 16-21)**

The 1/3 stress increase for load combinations containing wind or seismic loads is allowed when using these load combinations. As of this publication ASCE Load Combinations are moving closer to the IBC Combinations provided here.

In ASCE 7 Section 12.4.2.3, the above load combinations 16-20 and 16-21 are expanded to include the effects of vertical seismic forces.

$E = E_h + E_v$

$E = \rho Q_E$

$E_v = (0.2S_{DS})(D)$

12.2.2 SHEAR

Check the shear capacity first. This will usually determine the thickness of the masonry required. IBC Section 2106.5.1 requires the seismic shear force to be increased by 1.5.

IBC Section 2106.5.1

2106.5.1 Loads for shear walls designed by the working stress design method. When calculating in-plane shear or diagonal tension stresses by the working stress design method, shear walls that resist seismic forces shall be designed to resist 1.5 times the seismic forces required by Chapter 16. The 1.5 multiplier need not be applied to the overturning moment.

The maximum shear loading on the wall is:

$$V = \frac{(152.9)(1.5)}{1.4} = 163.8 \text{ Kip}$$

Assume 3 bars at the end of the wall.

Brick typically uses a 12 in. module while CMU or block uses a 16 in. module. Brick typically uses a 1/2 in. mortar joint while CMU or block uses a 3/8 in. mortar joint.

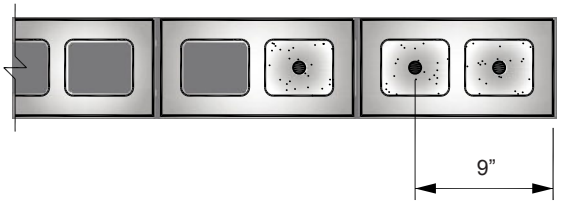


FIGURE 12.9 End of wall j.

The computed shear stress is:

d = 310 - 9 = 301 in.

$$f_v = \frac{V}{bd} = \frac{163{,}800}{(7.5)(301)} = 72.6 \text{ psi}$$

MSJC Code Eq (2-19)

The allowable shear stress is:

$$\frac{M}{Vd} = \frac{70{,}765{,}000}{(152{,}900)(301)} = 1.54 \geq 1.0$$

$$F_v = 1.5\sqrt{f'_m} \leq 75 \text{ psi} \quad \text{MSJC Code Eq (2-25)}$$

With the 1/3 stress increase:

$$F_v = \left(\frac{4}{3}\right)(1.5)\sqrt{f'_m} \leq \left(\frac{4}{3}\right)(75)$$

$$= \left(\frac{4}{3}\right)(1.5)\sqrt{2600} = 102 \text{ psi} \geq \left(\frac{4}{3}\right)(75 \text{ psi}) = 100 \text{ psi}$$

$f_v < F_v$ OK

The 8 in. nominal brick will work.

The resistance to the shear should be provided by the reinforcement.

$$V_n = A_n \rho_n f_y \qquad \text{(IBC Eq 21-1)}$$

The Code defines ρ_n as the "Ratio of distributed shear reinforcement on [a] plane perpendicular to [the] plane of A_{mv}". The Code does not provide a definition of A_{mv}. The intent is that the entire shear is taken by reinforcement. We have a long wall. The 45 degree crack length is limited by the height of the wall or 11'-0". Thus:

$$A_s = \frac{V_n}{f_y} = \frac{152{,}900}{60{,}000} = 2.54 \text{ in.}^2 \text{ / height of wall}$$

Two No. 5 bars at 24 in. on center will be sufficient.

Minimum reinforcement requirement is 0.0007 times the net area of the masonry, or:

(0.0007)(11)(12)(7.5) = 0.69 in^2. OK.

The axial load on the wall is compression. The wall will be governed by either the maximum compression loading or minimum compression loading.

12.2.3 COMPRESSION LIMIT: EQUATION 16-20

Loading combination Equation 16-21 will result in the maximum compression:

N-S Direction:

$$\left(1.0 + \frac{0.2S_{DS}}{1.4}\right)(D) + \frac{\rho Q_E}{1.4} + L + S$$

$$\left(1.0 + \frac{(0.2)(1.0)}{1.4}\right)(543.7) + 0.0 + 81.6 = 703 \text{ K}$$

(we have assumed no snow load and seismic N-S controls)

The compression stress on the wall is:

$$\frac{703{,}000}{(7.5)(310)} = 302 \text{ psi compression.}$$

The corresponding N-S in-plane moment in the wall is:

$$M = \frac{70{,}765{,}000}{1.4} = 50{,}546{,}000 \text{ lb - in.}$$

The bending tension stress from this moment is:

$$\frac{Mc}{I} = \frac{50{,}546{,}000\left(\frac{310}{2}\right)}{\frac{1}{(12)}(7.5)(310)^3} = 420$$

This is more than the 302 psi compression, so the wall is cracked for this load combination. But, the minimum axial loading will control the amount of reinforcement at the end of the wall.

12.2.4 TENSION LIMIT: EQUATION 16-21

Loading combination Equation 16-20 will result in the maximum tension:

$$P = \left(0.9 - \frac{0.2S_{DS}}{1.4}\right)(D) + \frac{\rho Q_E}{1.4}$$

$$= \left(0.9 - \frac{0.2}{1.4}\right)(543.7) + 0.0 = 411{,}700 \text{ lbs}$$

$$\frac{411{,}700}{(7.5)(310)} = 177 \text{ psi compression}$$

An estimate of the area of reinforcement required to resist the tension load can be found by splitting the axial load to the two ends of the wall, and resisting the moment with a couple using 70% (allowable stress design - strength design uses 80%) of the wall length.

$$A_s \cong \frac{\frac{M}{0.7L} - \frac{P}{2}}{F_s} = \frac{\frac{70{,}765{,}000}{(1.4)(0.7)(310)} - \frac{411{,}700}{2}}{24{,}000\left(\frac{4}{3}\right)} = 0.84 \text{ in}^2$$

Try 2 No. 6 bars. The distance to the centroid of the reinforcement is now 304 inches.

In ASD the wall can be either limited by the allowable stress in the reinforcement or the allowable stress in the masonry. If limited by the allowable stress in the reinforcement, the location of the neutral axis is:

$$k^2 = \left(2n\rho + 2\frac{\varepsilon_{mo}}{\varepsilon_s}\right)k - \left(2n\rho + 2\frac{\varepsilon_{mo}}{\varepsilon_s}\right) = 0$$

Where:

$$\varepsilon_{mo} = \frac{P}{E_m bd} = \frac{411{,}700}{(1{,}820{,}000)(7.5)(304)} = 0.000099$$

$$\frac{\varepsilon_{mo}}{\varepsilon_s} = \frac{0.000099}{\frac{(24{,}000)\left(\frac{4}{3}\right)}{29{,}000{,}000}} = 0.897$$

$$n\rho = \left(\frac{E_s}{E_m}\right)\left(\frac{A_s}{bd}\right) = \frac{29{,}000{,}000}{1{,}820{,}000}\frac{0.66}{(7.5)(304)} = 0.00615$$

The solution to the quadratic is k = 0.352 and j = 0.883

The resulting moment that is limited by the reinforcement is:

$$M_t = A_s F_s jd + P\left[1 - \frac{k}{3} - \left(1 - \frac{d}{2L}\right)\right]d$$

$$M_t = (0.88)(24{,}000)\left(\frac{4}{3}\right)(0.883)(304) + (411.700)\left[1 - \frac{0.352}{3} - \left(1 - \frac{304}{(2)(310)}\right)\right]304$$

M_t = 66,850,000 lb in. OK

If limited by the allowable stress in the masonry, the location of the neutral axis is:

$$k^2 + \left(2n\rho - 2\frac{\varepsilon_{mo}}{\varepsilon_m}\right)k - (2n\rho) = 0$$

Where:

$$\varepsilon_m = \frac{\left(\frac{2600}{3}\right)\left(\frac{4}{3}\right)}{1{,}820{,}000} = 0.000635$$

The solution to the quadratic is k = 0.559 and j = 0.814

The resulting moment that is limited by the masonry is:

$$M_c = \frac{F_b kjbd^2}{2} - P\frac{d}{2L}d$$

$$M_c = \frac{\left(\frac{2600}{3}\right)\left(\frac{4}{3}\right)(0.559)(0.814)(7.5)(304)^2}{2} - (411{,}700)\left(\frac{304}{(2)(310)}\right)(304) = 120{,}800{,}000 \text{ lb in. OK}$$

12.2.5 LIMITS ON REINFORCEMENT

The IBC 2006 Section 2107.8 limits the amount of reinforcement for special reinforced masonry shear walls to the following:

$$\rho_{\max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'_m}\right)} = \frac{(15.9)(2600)}{(2)(60{,}000)\left(15.9 + \frac{60{,}000}{2600}\right)}$$

= 0.00884

A_s = (0.00884)(7.5)(310) = 20.5 in.^{2} > 0.88 in.^{2} OK

12.3 DESIGN OF WALL "j" ON FIRST STORY, BASE LEVEL – STRENGTH DESIGN

Wall **j** carries vertical loads and in-plane seismic loads.

Masonry dimensions of the wall are:

b = 7.5 in. (Brick masonry is laid with a 1/2 in. mortar joint)

L = 25 ft - 10 in. = 310 in.

TABLE 12.16 Loads on Wall j

	Axial Load (Kip)	Shear Load (Kip)	Moment (Kip-in)
Dead Load	543.7	0	0
Live Load	81.6	0	0
Seismic Load (E-W)	0	120.4	57,487
Seismic Load (N-S)	0	152.9	70,765

12.3.1 LOAD COMBINATIONS

The load combinations contained in the 2006 IBC are:

1.4 (D + F) **(Equation 16-1)**

1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R) **(Equation 16-2)**

1.2D + 1.6(L_r or S or R) + (f_1L or 0.8W) **(Equation 16-3)**

1.2D + 1.6W + f_1L + 0.5(L_r or S or R) **(Equation 16-4)**

1.2D + 1.0E + f_1L + f_2S **(Equation 16-5)**

0.9D + 1.6W + 1.6H **(Equation 16-6)**

0.9D + 1.0E + 1.6H **(Equation 16-7)**

In ASCE 7 Section 12.4.2.3, the above IBC load combinations 16-5 and 16-7 are expanded to include the effects of vertical seismic forces.

5. $(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$

7. $(0.9 - 0.2S_{DS})D + \rho Q_E + 1.6H$

12.3.2 SHEAR

Check the shear capacity first. This will usually determine the thickness of the masonry required.

V_u = 152,900 lbs

M_u = 70,765,000 lb-in.

d_v = 310 in. (MSJC Code definition: "actual depth of masonry in direction of shear considered")

$V_n = V_m + V_s$ (MSJC Code Eq 3-18)

$$\frac{M}{Vd} = \frac{70{,}765{,}000}{(152{,}900)(310)} = 1.49 > 1.0$$

$V_n < 4A_n\sqrt{f'_m}$ (MSJC Code Eq 3-20)

$$= (4)(7.5)(310)\sqrt{2600} = 474{,}200 \text{ Kip} \quad \text{OK}$$

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u$$

(MSJC Code Eq 3-21)

$\frac{M_u}{V_u d_v}$ need not exceed 1.0

P_u is the factored axial load. Use the P_u that results in the lowest value.

However, Section 2106.5.2 of the 2006 IBC requires that the entire shear be resisted by reinforcement in the plastic hinge zone.

$V_n = A_n\rho_n f_y$

The Code defines ρ_n as the "Ratio of distributed shear reinforcement on [a] plane perpendicular to [the] plane of A_{mv}". The Code does not provide a definition of A_{mv}. The intent is that the entire shear is taken by reinforcement. We have a long wall, so the 45 degree crack length is limited by the height of the wall or 11'-0". Thus:

$$A_s = \frac{V_n}{f_y} = \frac{152.9}{60} = 2.54 \text{ in.}^2 \text{ / height of wall}$$

Two No. 5 bars at 24 in. on center will be sufficient.

Minimum reinforcement is 0.0007 times the net area of the masonry, or:

0.0007 x 11 x 12 x 7.5 = 0.69 in.^{2} OK.

Checking the shear in accordance with the MSJC Code and assuming the shear reinforcement is (2) No. 5 bars at 24 in. on center results in:

$$V_s = 0.5\left(\frac{A_v}{s}\right)f_y d_v \qquad \text{(MSJC Code Eq 3-22)}$$

$$V_s = 0.5\left(\frac{0.62}{24}\right)(60{,}000)(310) = 240{,}200 \text{ lbs}$$

ϕV_n = (0.8) (240,200) = 192,200 lbs > 152,900 lbs OK

The 8 in. brick wall is OK for shear.

The axial load on the wall is compression. The wall will be governed by either the maximum compression loading or minimum compression loading.

12.3.3 COMPRESSION LIMIT

Load combination Equation 16-5 will govern:

N-S Direction will control.

$(1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L + 0.2S$

= (1.2 + (0.2) (1.0)) (543.7) + (0.5) (81.6)

= 802,000 lbs (we have assumed no snow load)

The stress on the wall is:

$$\frac{802{,}000}{(7.5)(310)} = 345 \text{ psi compression.}$$

The corresponding N-S in-plane moment in the wall is:

M_u = 70,765,000 lb-in.

The bending tension stress from this moment is:

$$\frac{Mc}{I} = \frac{\frac{(70{,}765{,}000)(310)}{2}}{\frac{1}{12}(7.5)(310)^3} = 589 \text{ psi}$$

This is more than the 345 psi compression, so the wall is cracked for this load combination.

12.3.4 TENSION LIMIT

Check load combination 16-7 for required reinforcement:

$P_n = (0.9 - 0.2S_{DS})D + \rho Q_E$

= [0.9 - (0.2) (1.0)] (543.7) = 380,600 lbs

M_n = 70,765,000 in.-lb

An estimate of the area of reinforcement required to resist the tension load is can be found by splitting the axial load to the two ends of the wall, and resisting the moment with a couple using 80% of the wall length.

$$A_s \cong \frac{\frac{M}{0.8L} - \frac{P}{2}}{\phi F_y} = \frac{\frac{70{,}765{,}000}{(0.8)(310)} - \frac{380{,}600}{2}}{(0.9)(60{,}000)} = 1.75 \text{ in.}^2$$

Try 3 No. 7 bars.

As long as there is enough moment to yield the reinforcement, the moment allowed for this applied axial load is:

$$M_u = A_s F_y\left(1 - \frac{0.8k}{2}\right)d + P\left(1 - \frac{0.8k}{2} - \Delta\right)d$$

$$k = \frac{(A_s F_y + P)}{(0.8)(0.8)(bdf'_m)} = \frac{(1.75)(60{,}000) + 380{,}600}{(0.8)(0.8)(7.5)(301)(2600)}$$

= 0.130

Where:

$$\Delta = \left(1 - \frac{L/2}{d}\right) = \left(1 - \frac{310/2}{301}\right) = 0.485$$

$$M_u = (1.75)(60{,}000)\left(1 - \frac{(0.8)(0.13)}{2}\right)(301) + (380{,}600)$$

$$\left(1 - \frac{(0.8)(0.13)}{2} - 0.485\right)(301) = 83{,}859{,}000 \text{ lb in.}$$

ϕM_u = (0.9) (83,859,000) = 75,473,000 OK

12.3.5 LIMITS ON REINFORCEMENT

MSJC Code Section 3.3.5 limits the amount of reinforcement allowed in the wall. Often MSJC Code Section 3.3.6.5 can be used to circumvent Section 3.3.5 requirements, but in this case, the compression stresses are too high, and boundary elements would be required under MSJC Code Section 3.3.6.5.

The following equation defines the limit on the reinforcement.

$$A_s = \rho bd = \frac{0.64bdf'_m \dfrac{\varepsilon_{mo}}{(\varepsilon_{mo} + \alpha\varepsilon_y)} - P}{F_y}$$

α = 4.0 (MSJC Code Section 3.3.3.5.3)

MSJC Code Section 3.3.3.5.1(d) defines the amount of axial load to use in the reinforcement limiting equation as:

$P = D + 0.75L + 0.525Q_E$

= 543,700 + (0.75) (81,600) + (0.525) (0.0)

= 604,900 lbs

This axial load is an attempt to define a service load condition at 75% of expected loads. The 0.525 comes from the 75% times a factor of 0.7 intended reduce the seismic force to the ASD design level. The value of Q_E was taken is zero. ASCE 7 Section 12.14.1.4 defines Q_E as "The effect of horizontal seismic forces". Since wall **j** is not coupled with other walls, the effect is zero. It is not clear why the seismic vertical acceleration affect is not included in the axial load. Perhaps the code writers overlooked it.

$$A_s = \frac{(0.64)(7.5)(301)(2600)\dfrac{0.0035}{\left(0.0035 + (4)\dfrac{60{,}000}{29{,}000{,}000}\right)} - 604{,}900}{60{,}000}$$

= 8.5 in.^{2}

This is more reinforcement than is used for the in-plane loading [(3) No. 7's].

12.4 DESIGN OF WALL "f" ON FIRST STORY, BASE LEVEL

12.4.1 GENERAL

Wall **f** carries relatively low vertical loads and therefore overturning controls the design. Neglect any elevator loads imposed on wall **f**.

Masonry properties and dimensions of the wall are:

f'_m = 2600 psi (It is assumed the first floor is construction of hollow brick masonry. Assume an f'_m of 2600 psi. Values for brick can be significantly higher than this, but for now a lower value is used).

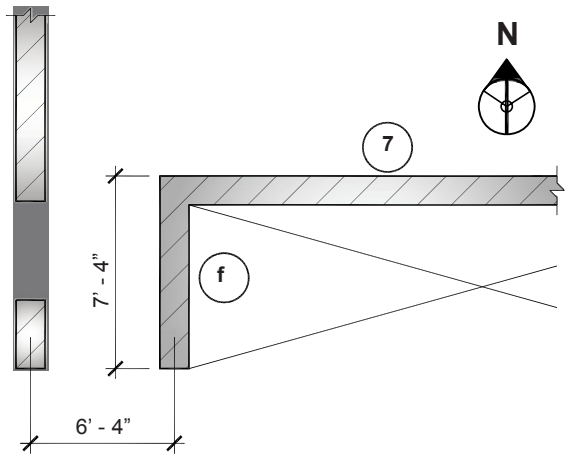


FIGURE 12.10 Wall f.

E_m = 700 f'_m = 1,820,000 psi

b = 7.5 in. (Brick masonry is laid with a 1/2 in. mortar joint)

L = 88 in.

TABLE 12.17 Loads on Wall f

	Axial Load (Kip)	Shear Load (Kip)	Moment (Kip-in)
Dead Load	58	0	0
Live Load	5.5	0	0
Seismic Load (E-W)	294.1	22.0	1990
Seismic Load (N-S)	268.1	25.6	2688

Two methods of design will be used, Allowable Stress Design using Chapter 2 of the 2005 MSJC Code and Strength Design using Chapter 3 of the 2005 MSJC Code.

12.4.2 ALLOWABLE STRESS DESIGN

We will use the alternative load combinations contained in the 2006 IBC. As of this publication ASCE Load Combinations are moving closer to the IBC Combinations provided here.

$D + L + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-16)**

$D + L + (\varpi W)$ **(Equation 16-17)**

$D + L + \varpi W + S/2$ **(Equation 16-18)**

$D + L + S + \varpi W/2$ **(Equation 16-19)**

$D + L + S + E/1.4$ **(Equation 16-20)**

$0.9D + E/1.4$ **(Equation 16-21)**

The 1/3 stress increase for load combinations containing wind or seismic loads is allowed when using these load combinations.

In ASCE 7 Section 12.4.2.3, the above IBC load combinations 16-20 and 16-21 are expanded to include the effects of vertical seismic forces.

$E = E_h + E_v$

$E_h = \rho Q_E$

$E_v = 0.2 S_{DS} D$

Check the shear capacity first. This will usually determine the thickness of the masonry required. IBC Section 2106.5.1 requires the seismic shear force to be increased by 1.5.

IBC Section 2106.5.1

2106.5.1 Loads for shear walls designed by the working stress design method. When calculating in-plane shear or diagonal tension stresses by the working stress design method, shear walls that resist seismic forces shall be designed to resist 1.5 times the seismic forces required by Chapter 16. The 1.5 multiplier need not be applied to the overturning moment.

The maximum shear loading on the wall is:

$$V = \frac{(25.6)(1.5)}{1.4} = 27.4 \text{ kip}$$

The computed shear stress is:

Assume 3 bars at each end of the wall.

Brick typically uses a 12 in. module while block uses a 16 in. module. Brick typically uses a 1/2 in. mortar joint while block uses a 3/8 in. mortar joint.

$d = 88 - 9 = 79$ in.

$$f_v = \frac{V}{bd} = \frac{27{,}400}{(7.5)(79)} = 46.2 \text{ psi (MSJC Code Eq 2-19)}$$

The allowable shear stress is:

$$\frac{M}{Vd} = \frac{2{,}688{,}000}{(25{,}600)(79)} = 1.32 \geq 1.0$$

$$F_v = 1.5\sqrt{f'_m} \leq 75 \text{ psi}$$

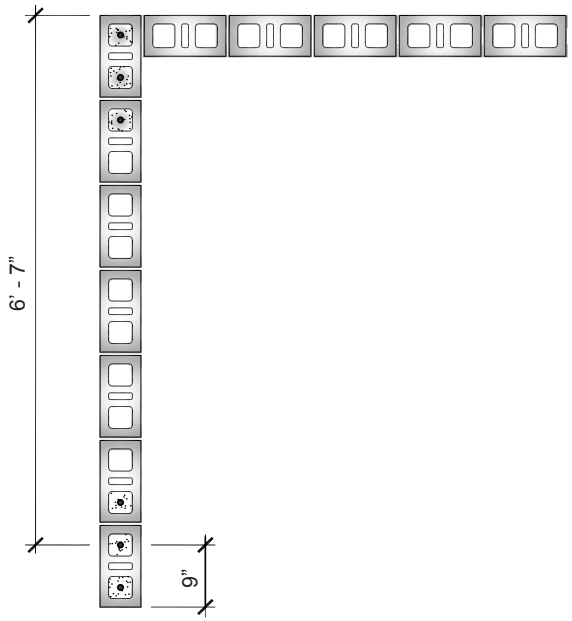


FIGURE 12.11 Detail of reinforcing steel in wall f.

with the 1/3 stress increase:

$$F_v = \left(\frac{4}{3}\right)1.5\sqrt{f'_m} \leq \left(\frac{4}{3}\right)75$$

$$= \left(\frac{4}{3}\right)1.5\sqrt{2600} = 102 \text{ psi} \geq \left(\frac{4}{3}\right)75 \text{ psi} = 100 \text{ psi}$$

$f_v < F_v$ OK

The 8 in. nominal brick will work.

The axial load on the wall can be either tension or compression. The load results from the connections to wall 7 resulting in coupling between wall **f** and **g**. Wall f needs to be designed for both the tension and compression conditions.

Compression Equation 16-20

Loading combination Equation 16-20 will govern:

E-W Direction:

$$\left(1.0 + \frac{02 S_{DS}}{1.4}\right)D + \frac{\rho Q_E}{1.4} + L + S$$

$$= \left(1.0 + \frac{(0.2)(1.0)}{1.4}\right)(58) + \left(\frac{294.1}{1.4}\right) + 5.5 = 282 \text{ K}$$

(we have assumed no snow load and the seismic E-W controls).

The stress of the wall is:

$$\frac{282{,}000}{(7.5)(88)} = 427 \text{ psi compression.}$$

The corresponding E-W in-plane moment in the wall is:

$$M = \frac{1{,}990{,}000}{1.4} = 1{,}421{,}000 \text{ lb - in.}$$

The bending tension stress from this moment is:

$$\frac{Mc}{I} = \frac{1{,}421{,}000\left(\frac{88}{2}\right)}{\frac{1}{12}(7.5)(88)^3} = 146 \text{ psi}$$

This is less than the 427 compression, so the wall remains uncracked for this load combination.

Maximum compression stress is 427 + 146 = 573 psi

N-S Direction

A check of this load combination for the seismic N-S directions results in:

$$\left(1.0 + \frac{0.2S_{DS}}{1.4}\right)D + \frac{\rho Q_E}{1.4} + L + S$$

$$= \left(1.0 + \frac{(0.2)(1.0)}{1.4}\right)(58) + \frac{268.1}{1.4} + 5.5 = 263 \text{ K}$$

$$\frac{263{,}000}{(7.5)(88)} = 399 \text{ psi}$$

The corresponding N-S in-plane moment in the wall is:

$$M = \frac{2{,}688{,}000}{1.4} = 1{,}920{,}000 \text{ lbs - in.}$$

The bending tension stress from this moment is:

$$\frac{Mc}{I} = \frac{1{,}920{,}000\left(\frac{88}{2}\right)}{\frac{1}{12}(7.5)(88)^3} = 198 \text{ psi}$$

This is less than the 399 psi compression, so the wall remains uncracked for this load combination.

The maximum compression stress occurs in the N-S direction and is:

399 + 198 = 597 psi

There is no reinforcement required for the load combination 16-20, since there is no net tension.

Tension Equation 16-21

Check load combination 16-21 for required reinforcement for E-W direction:

$$P = \left(0.9 - \frac{02S_{DS}}{1.4}\right)D - \frac{\rho Q_E}{1.4}$$

$$= \left(0.9 - \frac{(0.2)(1.0)}{1.4}\right)58 - \frac{(1.0)(294.1)}{1.4}$$

= -166,200 lbs tension

$$M = \frac{1{,}990{,}000}{1.4} = 1{,}421{,}000 \text{ in. - lb}$$

For the N-S direction:

$$P = \left(0.9 - \frac{0.2S_{DS}}{1.4}\right)D - \frac{\rho Q_E}{1.4}$$

$$= \left(0.9 - \frac{(0.2)(1.0)}{1.4}\right)58 - \frac{(1.0)(268.1)}{1.4}$$

= -147,600 lbs tension

$$M = \frac{2{,}688{,}000}{1.4} = 1{,}920{,}000 \text{ in. - lb}$$

An estimate of the area of reinforcement required to resist the tension load is:

$$A_s = \frac{166{,}200}{24{,}000\left(\frac{4}{3}\right)} = 5.2 \text{ in.}^2$$

Use (8) No. 8 bars. Assume there are three at the end of the wall and two near the center. Now that we have an estimated reinforcement, an approximate bending and axial interaction diagram can be produced, and the load combinations checked.

Point 1: No moment

$$r = \sqrt{\frac{I}{A}} = 0.288t \text{ for solid grouted masonry}$$

$$\frac{h}{r} = \frac{(11)(12)}{(0.288)(7.5)} = 61 < 99$$

$$P = 0.25f'_m A_n\left[1-\left(\frac{h}{140r}\right)^2\right]\frac{4}{3} = (0.25)(2600)(7.5)$$

$$(88)\left[1-\left(\frac{132}{(140)(2.17)}\right)^2\right]\frac{4}{3} = 464{,}000 \text{ lbs}$$

Compression was not considered in the calculation because it is not tied.

Point 2: No axial

d = 88 - 9 = 79

b = 7.5

$$n = \frac{29{,}000{,}000}{(700)(2{,}600)} = 15.9$$

A_s = 3 #8 bars = 2.37 in.^{2}

$$\rho = \frac{A_s}{bd} = \frac{2.37}{(7.5)(79)} = 0.0040$$

$n\rho$ = 0.0636

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho = 0.30$$

$$j = \left(1-\frac{k}{3}\right) = \left(1-\frac{0.3}{3}\right) = 0.90$$

Limited by the compression allowable:

$$M_c = \frac{bd^2}{2}(kj)F_b = \frac{(7.5)(79)^2}{2}(0.30)(0.90)$$

$$\left(\frac{2600}{3}\right)\left(\frac{4}{3}\right) = 7{,}302{,}000 \text{ lb - in.}$$

Limited by the tension allowable:

$$M_t = A_s jdF_s = (2.37)(0.9)(79)(24{,}000)\left(\frac{4}{3}\right)$$

= 5,392,000 lb - in.

Point 3: Balanced compression and tension limit:

The point where the tension allowable and compression allowable are reached is the balance point. The location of the neutral axis is:

$$k = \frac{\varepsilon_m}{\varepsilon_m + \varepsilon_s} = \frac{\frac{F_b}{E_m}}{\frac{F_b}{E_m} + \frac{F_s}{E_s}} = \frac{1}{1+\frac{F_s}{nF_b}} = \frac{1}{1+\frac{24{,}000}{(15.9)\left(\frac{2600}{3}\right)}}$$

= 0.365

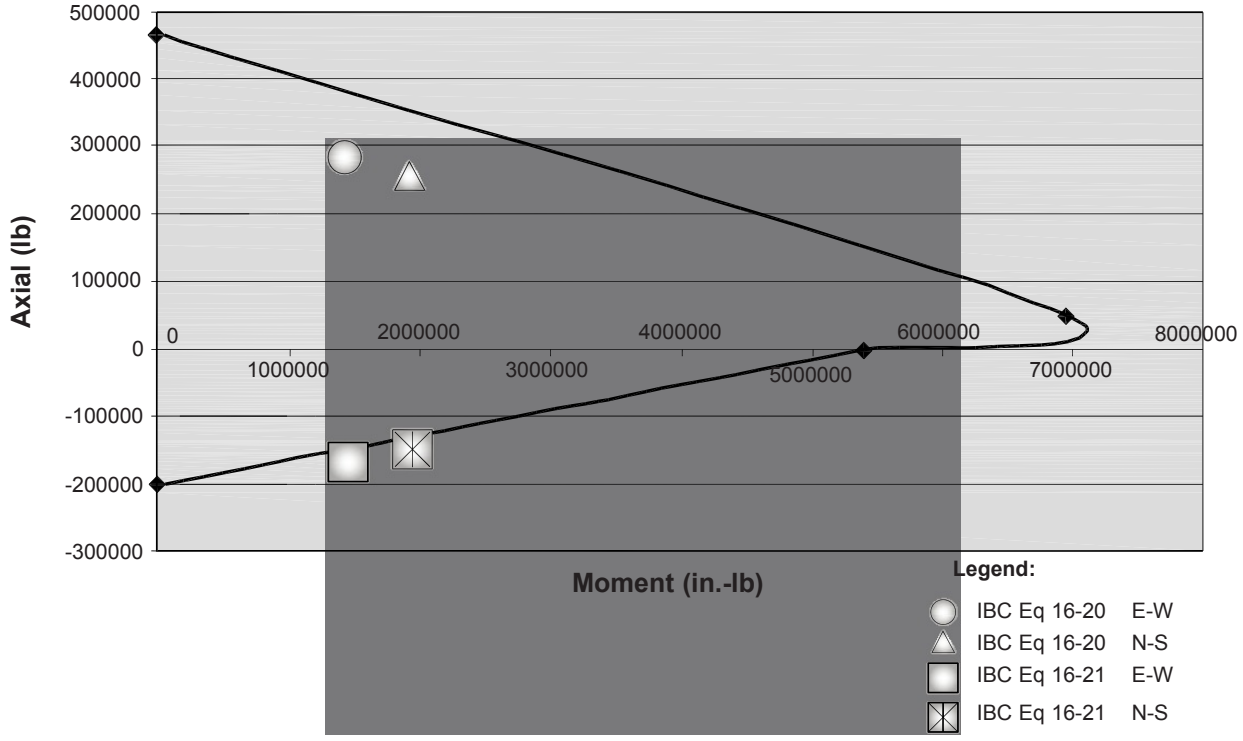


FIGURE 12.12 Wall f approximate interaction ASD.

The axial load is:

$$P_{balanced} = \frac{bkd}{2} F_b - A_s F_s$$

$$= \left(\frac{(7.5)(0.365)(79)}{2}\right)\left(\frac{2600}{3}\right)\left(\frac{4}{3}\right) -$$

$$(2.37)(24{,}000)\left(\frac{4}{3}\right) = 49{,}100 \text{ lbs}$$

where Δd is the distance from the centroid of reinforcement to the line of centroid of axial load:

$$M_{c-balanced} = \frac{bd^2}{2} kjF_b - P\Delta d = \frac{(7.5)(79)^2}{2}(0.365)$$

$$\left(1 - \frac{0.365}{3}\right)\left(\frac{2600}{3}\right)\left(\frac{4}{3}\right) - (49{,}100)\left(1 - \frac{88}{(2)(79)}\right)(79)$$

= 6,952,000 lb - in.

Point 4: Axial tension only:

$$P_t = (8)(0.79)(24{,}000)\left(\frac{4}{3}\right) = 202{,}200 \text{ lbs}$$

The simple interaction diagram with the load combinations included indicates the design is marginal, but acceptable. Adding a vertical reinforcing bar at the center of wall will increase the conservatism of design.

12.4.3 LIMITS ON REINFORCEMENT

The IBC 2006 Section 2107.8 limits the amount of reinforcement in special reinforced masonry shear walls to the following:

$$\rho_{max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'_m}\right)} = \frac{(15.9)(2600)}{(2)(60{,}000)\left(15.9 + \frac{60{,}000}{2600}\right)}$$

= 0.00884

The compression in wall **f** is due to in-plane bending of wall **f** and coupling with wall **g**. Separating the two and using some engineering judgement results in the following approximate analysis:

In-plane

For tension reinforcement the following equation is the limit on the reinforcement.

A_s = (0.00844) (7.5) (88) = 5.83 > (3) (0.79) = 2.37

If we consider the coupling forces and apply the equation between walls **f** and **g**, and neglecting the compression contribution of the web (wall 7) and assuming the neutral axis beyond the flange (wall **f** is all in compression), an approximation of the maximum can be found. The *d* in the calculation is now the distance between wall **f** and wall **g** and *b* is the length of wall **f**.

A_s = (0.00844) (88) (232)

= 172 in.^{2} > (8) (0.79) = 6.32 in.^{2} OK

This appears satisfactory.

12.5 STRENGTH DESIGN

12.5.1 LOAD COMBINATIONS

The load combinations contained in the 2006 IBC Section 1605.2.1 are:

1.4 (D + F) **(Equation 16-1)**

1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R) **(Equation 16-2)**

1.2D + 1.6(L_r or S or R) + (f_1L or 0.8W) **(Equation 16-3)**

1.2D + 1.6W + f_1L + 0.5(L_r or S or R) **(Equation 16-4)**

1.2D + 1.0E + f_1L + f_2S **(Equation 16-5)**

0.9D + 1.6W + 1.6H **(Equation 16-6)**

0.9D + 1.0E + 1.6H **(Equation 16-7)**

In ASCE 7 Section 12.4.2.3, the above IBC load combinations 16-5 and 16-7 are expanded to include the effects of vertical seismic forces.

5. (1.2 + 0.2S_{DS})D + ρQ_E + L + 0.2S

7. (0.9 - 0.2S_{DS})D + ρQ_E + 1.6H

12.5.2 SHEAR

Check the shear capacity first. This will usually determine the thickness of the masonry required.

The maximum shear loading on the wall is 25.6 kip (N-S). The shear stress is:

V_u = 25,600 lbs

M_u = 2,688,000 lb-in

d_v = the actual depth of the masonry in the direction of shear.

d_v = 88 in

$V_n = V_m + V_s$ (MSJC Code Eq 3-18)

For $M/Vd > 1.0$

$$V_n < 4A_n\sqrt{f'_m} = (4)(7.5)(79)\left(\sqrt{2600}\right) = 120.8\,\text{Kip OK}$$

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right]A_n\sqrt{f'_m} + 0.25P_u$$

(MSJC Code Eq 3-21)

$\frac{M_u}{V_u d_v}$ need not exceed 1.0

The code is not clear about using the axial load if it is in tension. Clearly the intent of the 0.25 factor is that axial load should only be included if compression. But for coupled walls, the seismic load is both tension and compression.

Engineering judgment is required. The wall has a net tension, so a rational approach is to neglect the contribution to shear strength by the masonry (V_m - 0).

Assume shear reinforcement of (2) No. 4 bars at 24 in. on center results in:

$$V_s = 0.5\left(\frac{A_v}{s}\right)f_y d_v$$

$$V_s = 0.5\left(\frac{0.4}{24}\right)(60{,}000)(88) = 44{,}000\ \text{lbs}$$

ϕV_n = (0.8) (44,000) = 35,200 lbs OK

The 8 in. brick wall is OK for shear.

12.5.3 COMPRESSION LIMITING

Load combination Equation 16-5 as expanded by ASCE-7 will govern:

E-W Direction

$(1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L + 0.2S$

= [1.2 + (0.2) (1.0)] (58) + 294.1 + (0.5) (5.5)

= 378,000 lbs (we have assumed no snow load and seismic E-W controls)

The stress on the wall is:

$$\frac{378{,}000}{(7.5)(88)} = 573\ \text{psi compression.}$$

The corresponding E-W in-plane moment in the wall is:

M_u = 1,990,000 lb-in.

The bending tension stress from this moment is:

$$\frac{Mc}{I} = \frac{(1{,}990{,}000)\left(\frac{88}{2}\right)}{\left(\frac{1}{12}\right)(7.5)(88)^3} = 206\ \text{psi}$$

This is less than the 573 compression, so the wall remains uncracked for this load combination.

N-S Direction

A check of this load combination for the seismic N-S directions results in:

$(1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L + 0.2S$

= (1.2 + 0.2) (58) + 268.1 + (0.5) (5.5)

= 352,000 lbs or 533 psi.

The corresponding N-S in-plane moment in the wall is:

M_u = 2,688,000 lb-in.

The bending tension stress from this moment is:

$$\frac{Mc}{I} = \frac{(2{,}688{,}000)\left(\frac{88}{2}\right)}{\left(\frac{1}{12}\right)(7.5)(88)^3} = 278\ \text{psi}$$

This is less than the 533 psi compression, so the wall remains uncracked for this load combination.

The maximum compression stress occurs in the N-S direction and is:

278 + 533 = 811 psi

There is no reinforcement required for the load combination 16-5, since there is no net tension force.

12.5.4 TENSION

Check IBC load combination 16-7 for required reinforcement:

$P_n = (0.9 - 0.2S_{DS})D - \rho Q_E$

$= (0.9 - 0.2)\ (58) - (1.0)\ (294.1)$

$= -253{,}500$ lbs tension

$M_n = 1{,}990{,}000$ in-lbs

Or

$P_n = (0.9 - 0.2S_{DS})D - \rho Q_E$

$= (0.9 - 0.2)\ (58) - (1.0)\ (268.1)$

$= -227{,}500$ lbs tension

$M_n = 2{,}688{,}000$ in-lbs

The area of reinforcement required to resist the tension load is:

$$A_s = \frac{253{,}500}{(0.9)(60{,}000)} = 4.7 \text{ in.}^2$$

Use 6 No. 8 bars.

Now that we have an estimated reinforcement, an approximate bending and axial interaction diagram can be produced, and the load combinations checked.

Point 1: Axial compression only, compression reinforcement not considered in calculation because it is not tied.

$$\frac{h}{r} = \frac{(11)(12)}{(0.288)(7.5)} = 61 < 99$$

$$P_n = 0.80\left(0.80f'_m A_n\left[1 - \left(\frac{h}{140r}\right)^2\right]\right)$$

$$\phi P_n = (0.9)(0.8)(0.8)(2600)(7.5)(88)\left[1 - \left(\frac{61}{140}\right)^2\right]$$

$= 800{,}800$ lbs

Point 2: No axial

The distance to the neutral axis of the reinforcement is:

$d = 88 - 3 - 6 = 79$ in. (brick uses a 12 in. module)

$L = 88$ in.

$A_s = 2.37$ (3 No. 8 bars)

$b = 7.5$ in.

$= 9{,}527{,}000$ in-lb

$$M_n = A_s F_y\left[1 - \left(\left(\frac{1}{2}\right)\frac{A_s F_y}{0.8bdf'_m}\right)\right]d$$

$$\phi M = (0.9)(2.37)(60{,}000)\left[1 - \left(\left(\frac{1}{2}\right)\frac{(2.37)(60{,}000)}{(0.8)(7.5)(79)(2600)}\right)\right](79)$$

Point 3: Sufficient axial load so the reinforcement is just at yield.

The location of the neutral axis is;

$$k = \left(\frac{\varepsilon_{mo}}{\varepsilon_{mo} + \frac{F_y}{E_m}}\right)$$

$$k = \frac{0.0035}{\left[0.0035 + \left(\frac{60{,}000}{29{,}000{,}000}\right)\right]} = 0.628$$

The axial load is:

$P_n = (0.8)(08)f'_m bkd - A_s F_y$

$\phi P_n = (0.9)\ [(0.8)\ (0.8)\ (2600)\ (7.5)\ (0.628)\ (79)$
$- (2.37)\ (60{,}000)] = 429{,}300$ lbs

The moment is:

$$M_n = (0.8)(0.8)f'_m kbd\left(1 - \frac{0.8k}{2}\right)d - P\left(1 - \left(\frac{L}{2d}\right)\right)d$$

$$\phi M = (0.9)(0.8)(0.8)(2600)(0.628)(7.5)(79)\left[1 - \left(\frac{(0.8)(0.628)}{2}\right)\right](79) - (477{,}000)\left[1 - \left(\frac{(88)}{(2)(79)}\right)\right](79) = 17{,}938{,}000 \text{ lb in.}$$

Point 4: Axial tension only.

$\phi P_n = (0.9)\ (4.74)\ (60{,}000) = 255{,}900$ lbs tension

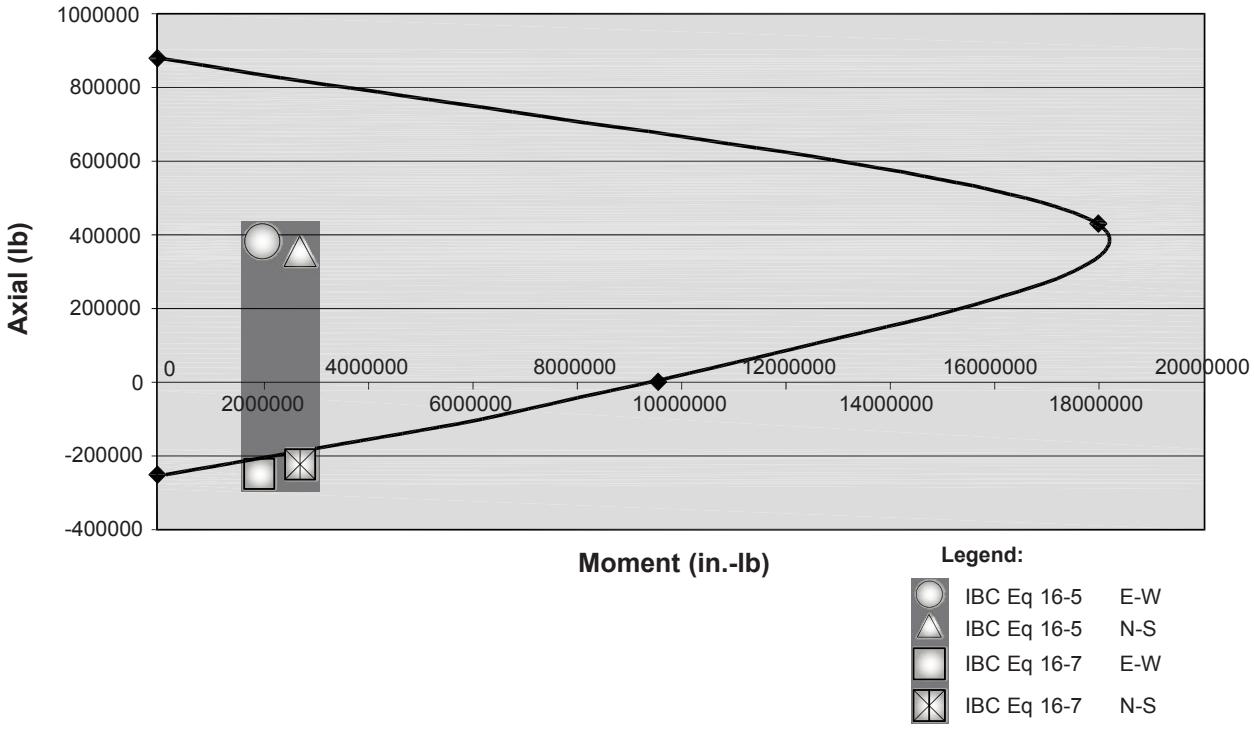


FIGURE 12.13 **Wall f approximate interaction - 6 #8 bars.**

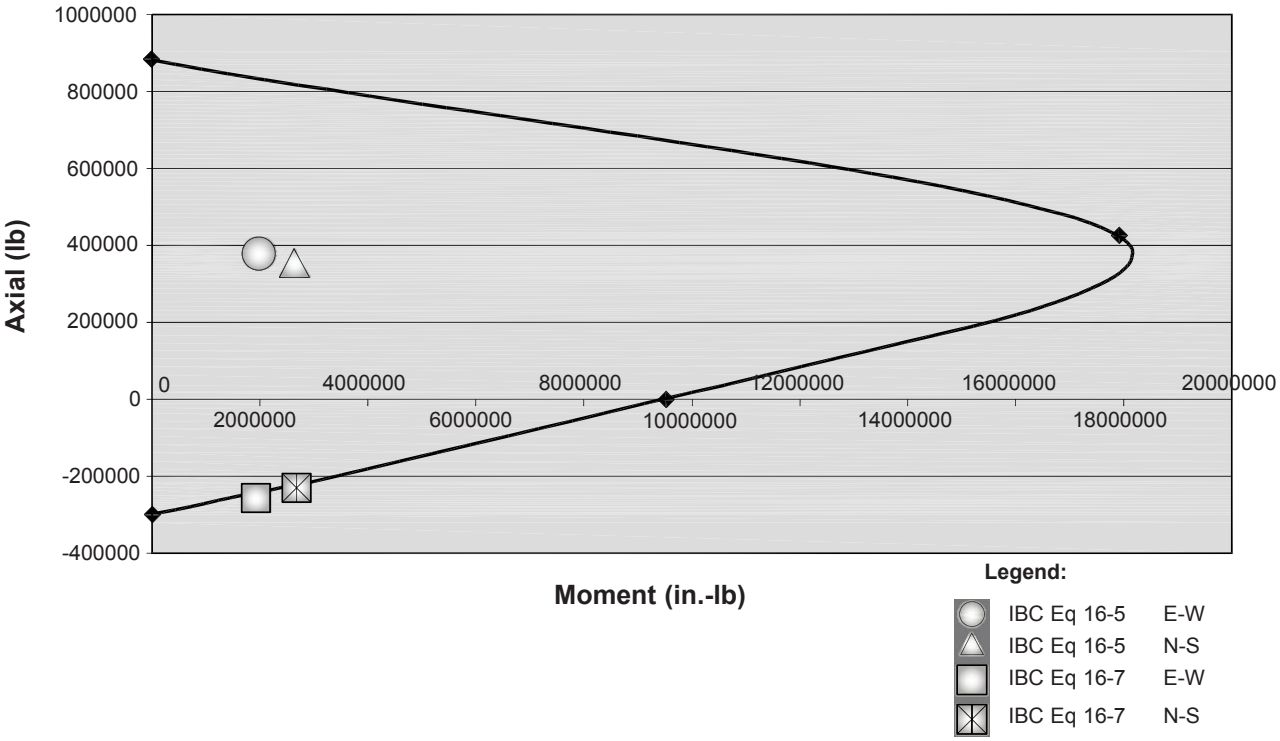


FIGURE 12.14 **Wall f approximate interaction - 6 #8 Bars plus 1 #8 Bar at wall center.**

The simple interaction diagram with the load combinations included indicates the design is marginal.

The addition of another No. 8 bar at the center of the wall will not affect the previous calculations since only the reinforcement at *d* was used. However, it will provide additional axial tension loading:

$\phi P_n = (0.9)(7)(0.79)(60{,}000) = 298{,}600$ lbs tension

12.5.5 LIMITS ON REINFORCEMENT

MSJC Code Section 3.3.5 limits the amount of reinforcement allowed in the wall. Often MSJC Code Section 3.3.6.5 can be used to circumvent Section 3.3.3.5 requirements, but in this case, the compression stresses are too high, and boundary elements would be required under Section 3.3.6.6.

However, MSJC Code Section 3.3.5 assumes cantilevered/uncoupled shear walls. As can be seen in the following figure, our situation is much more complex.

The compression in wall **f** is due to in-plane bending of wall **f** and coupling with wall **g**. Separating the two and using some engineering judgment results in the following approximate analysis:

In-plane

For tension reinforcement the following equation is the limit on the reinforcement (MSJC Code Commentary 3.3.3.5).

$$A_s = \rho bd = \frac{(0.8)(0.8)bdf'_m \dfrac{\varepsilon_{mo}}{(\varepsilon_{mo} + \alpha\varepsilon_y)} - P}{F_y}$$

α = 4.0 MSJC Code Section 3.3.3.5.3

$P = D + 0.75L + 0.525\ Q_E$

$= 58{,}000 + (0.75)(5{,}500) + (0.525)(0.0)$

$= 62{,}125$ lbs

This axial load is an attempt to define a service load condition at 75% of expected loads as required by MSJC Code Section 3.3.5.1(d). The 0.525 comes from the 75% times 0.7 to reduce the seismic to the design level. The value of Q_E was taken as zero, because we are only looking at the in-plane condition of wall **f**.

$$A_s = (0.8)(.8)(7.5)(79)(2600)\left[\left(\frac{0.0035}{0.0035+4}\right)\left(\frac{60{,}000}{29{,}000{,}000}\right)\right] - \frac{62{,}125}{60{,}000} = 3.8 \text{ in.}^2$$

This is more reinforcement than is used for the in-plane loading [(3) No. 8's].

If we consider the coupling forces and apply the equation between walls **f** and **g**, the value of *P* is:

$P = D + 0.75\ L + 0.525\ Q_E$

$= 58{,}000 + (0.75)(5{,}500) + (0.525)(294{,}100)$

$= 216{,}500$ lbs

And, neglecting the compression contribution of the web (wall 7) and assuming that the 4 times yield places the neutral axis beyond the flange (wall **f** is all in compression), an approximation of the maximum reinforcement is:

$$A_s = \frac{(0.8)(0.8)(88)(7.5)(2600) - 216{,}500}{60{,}000} = 14.7 \text{ in.}^2$$

This appears to be satisfactory.

The reinforcement causing compression in wall **f** is in wall **g**. The provision is intended to limit the reinforcement in wall **g**.

12.6 HISTORY OF WALL j

The previous edition of Reinforced Masonry Engineering Handbook had the building weigh in at 8409 kips compared to 8399 kips. It is encouraging to be that close. The base shear for seismic using the old Uniform Building Code equivalent lateral force method was 1538 kip compared to the 1470 kips used in this design. Previous editions distributed the loads to the walls using the tedious hand methods instead of the computer model. It is interesting to compare the results.

For wall **j**, the hand solution in previous editions resulted in a seismic shear force of 161 kips compared to the computer model result of 152.9 kip. The hand calculated overturning moment was 80,736 kip-in. compared to the computer models 70,765 kip-in.

The end of the wall reinforcement in previous editions is (4) No. 9 bars compared to the current

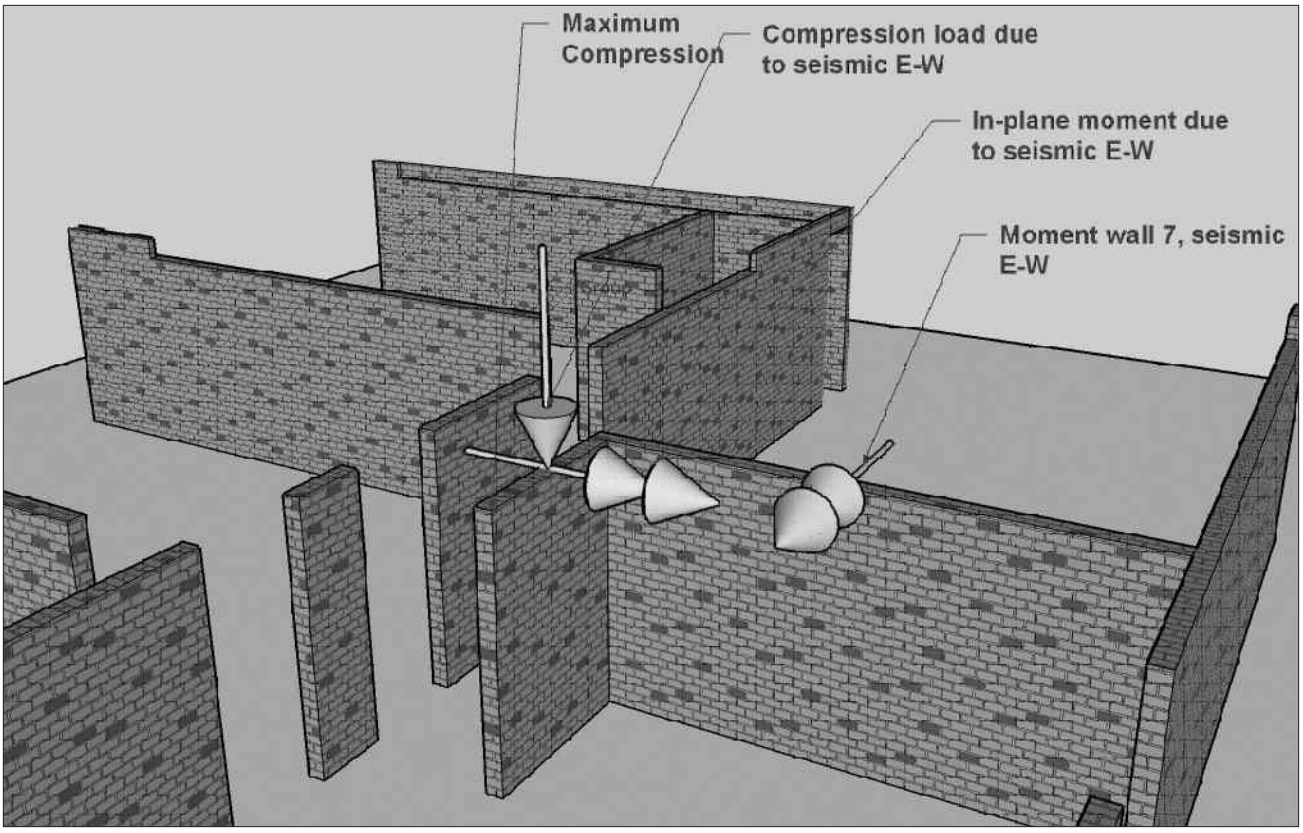


FIGURE 12.15 Wall f and 7, Loading

design. The current design resulted in (2) No. 6 bars for allowable stress design and (3) No. 7 bars for strength design. The amount of reinforcement at the end of the wall is sensitive to the ratio of moment to dead load. This explains the diffence between the previous editions results and the current analysis. The difference between the strength allowable stress design reinforcement and strength design reinforcement is explained by the differences in the load combinations and methods of analysis.

Over the years, wall **j** has been used as a trial design. The following table compared the loading on the walls between the current analysis and the trial design.

Load	This Analysis	Trial Designs
Dead	543.7	560
Live	72	81.6
Seismic Shear	152.9	143
Seismic Moment	70,765	74,100

The calculated end of the wall reinforcement (trim steel) from the trial design is as follows:

Trial Design	Trim Steel
1	(4) No. 5
2	(1) No. 5
3	None
4	None
5	(6) No. 5
6	None
7	(1) No. 7
8	(6) No. 6
9	(6) No. 5

A report on the trial designs can be found at the Western States Clay Products Association web site.

12.7 ADDITIONAL CONSIDERATIONS IN THE DESIGN OF MULTI-STORY SHEAR WALL STRUCTURES

The concepts outlined in this book provide information on the design and detailing of a multi-story shear wall structure. In addition, there are alternate concepts and considerations that should be examined.

Repetition of floor layout from floor to floor will speed construction. The contractor and mason will become familiar with the layout and production will increase as the building is constructed.

If possible, walls should be continuous to the foundation. This will eliminate discontinuities or the need to develop a shear platform to transfer the shear to other walls.

It is advantageous to have as uniform and symmetrical floor plans as possible. This will reduce torsional forces in the building and simplify design, detailing and construction.

Buildings with irregular plans such as T, Z, U and L layouts should be designed with floor diaphragms sufficiently stiff to transmit the torsional forces to the various shear wall elements or there should be a separation of each wing so that each section will act independently. The separation should be adequate to prevent impact between building sections and should be at least the computed drift due to wind or earthquake forces plus a clearance allowance.

Buildings can react as shear structures in which the major drift, lateral deflection, due to lateral loads is caused by shear deformation of the walls rather than moment deflection of the frame and shortening of the column or boundary members due to overturning forces. Shear structures in which this phenomenon dominates generally have a height to width ratio of 1.5 or less.

When the height to width ratio of a building is 1.20 or more it can react as a moment or flexural structure in which the major drift due to lateral forces is caused by moment deformation, joint rotation (in the case of a frame building) and column or boundary member shortening due to overturning moment and flange stresses. The shear walls in these relatively flexible structures should be designed for their full height as walls loaded laterally and cantilevering from the foundation as in the design example in this section.

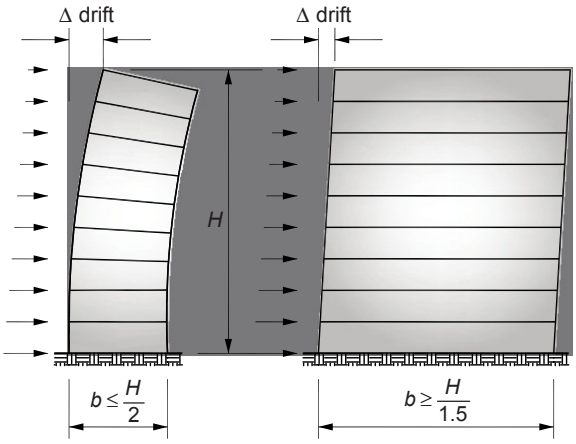


FIGURE 12.16 Drift of a flexible moment frame structure and a rigid shear wall structure.

Shear wall buildings are generally stiff buildings and do not distort as much as frame buildings. This means that there will be little or no damage to non-structural, architectural elements, and no damage to the structural walls in buildings subjected to small earthquakes, and only slight damage in medium earthquakes. Shear walls which resist the lateral forces, can vary greatly in stiffness depending on the placement of openings in the wall. Figure 12-17 compares the rigidity of a wall with no openings to walls of the same size but with various opening patterns.

By staggering the openings, the wall acts as a solid wall with scattered openings. Walls that have the openings stacked on top of each other act as independent elements and the total rigidity is significantly reduced. This comparison assumes that the floor system connecting the walls has no resistance to transfer shear or moment.

The strength of the connecting member between the wall elements can also significantly influence the total strength of the wall. The effect of the stiffness of a connecting member on the rigidity of the system produces a coupled wall mechanism. The connecting members, floor slabs or spandrel or lintel beams in coupled shear walls should be investigated and designed for the stresses, moments and shear forces induced in them.

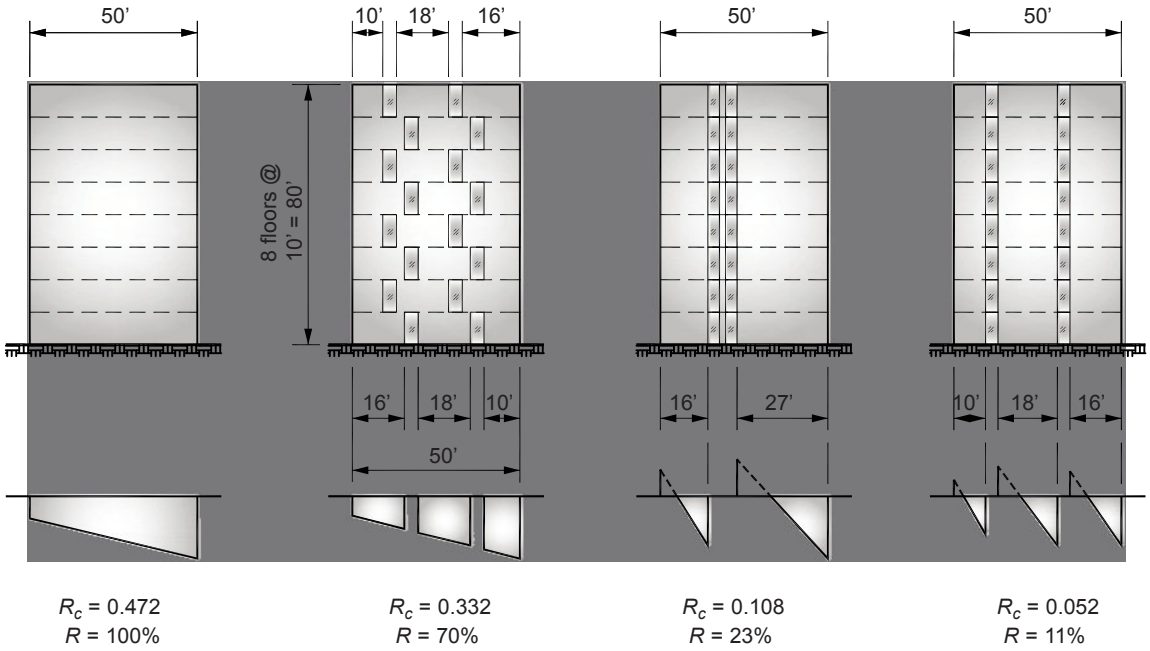
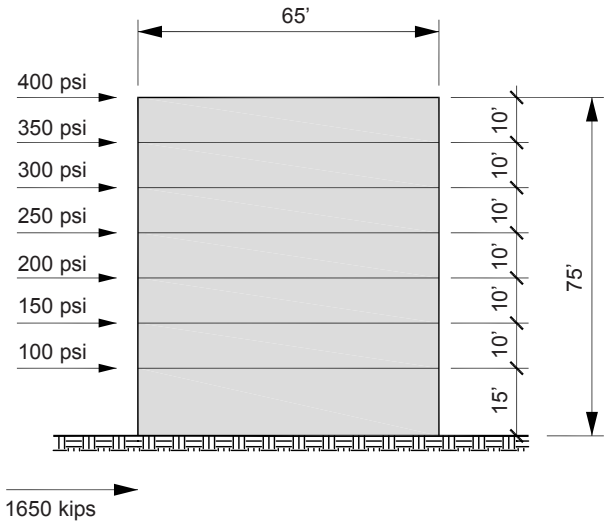


FIGURE 12.17 **Relative rigidities of walls with various opening patterns.**

The moment capacity of shear walls can be greatly increased when acting as vertical cantilever elements by using returns at the ends which function as flanges and increased area to resist compression forces. The flanges will increase the moment of inertia of the wall section, decrease the flexural stresses and facilitate the placement and efficiency of reinforcing steel. These flanges can be readily incorporated at door returns. They also significantly reduce shear and moment stresses and thus reduce the amount of steel required for overturning forces.

12.8 QUESTIONS AND PROBLEMS

12-1 What is the overturning moment and shear on the 7 story building at each floor level given the force distribution shown?



CHAPTER 13

RETAINING WALLS

13.1 GENERAL

One of the many applications of masonry is retaining walls. Masonry walls inherently provide visual beauty along with the required structural strength to resist imposed vertical and lateral loads.

FIGURE 13.1 Reinforced masonry wall retaining eight feet of backfill.

13.2 TYPES OF RETAINING WALLS

There are four basic types of retaining walls, gravity walls, counterfort or buttress walls, cantilever walls, and supported walls. Selection of a particular type of wall will depend on the site, size of wall, loads, soil conditions, use and economics of construction.

13.2.1 GRAVITY WALLS

Gravity walls are designed so that no tension stresses develop in the wall under most loading conditions. In some instances, low tension stresses are permitted by providing reinforcement in the wall. Partially reinforced walls are considered semi-gravity walls. Gravity walls, more commonly known as segmental retaining walls, have gained significant popularity in recent years.

There are two types of Segmental Retaining Walls (SRW's) (Figure 13.2).

- Conventional SRW's rely on self-weight to retain soil. This type of system is typically limited in height without engineering.

- Reinforced SRW's are tied into the soil using a geogrid system. This type of system relies on embedding geogrid into the soil in addition to the self-weight of the masonry units.

Both systems require a gravel fill behind the wall and a drainage system to minimize hydrostatic pressure on the retaining wall. Drainage of water from behind the wall is recommended for any type of retaining wall system, but is particularly important for a gravity wall system.

13.2.2 COUNTERFORT OR BUTTRESS WALLS

These walls span horizontally between vertical support members. If vertical supports are behind the wall and buried in the earth backfill, they are called

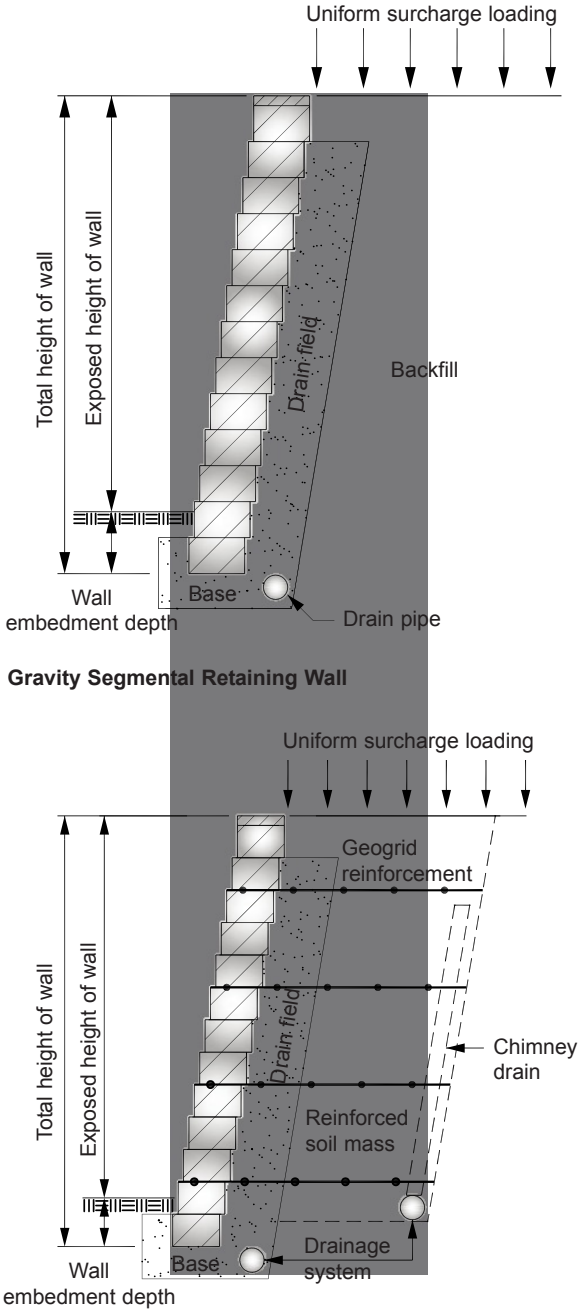


FIGURE 13.2 Segmental retaining wall.

counterforts and are tension members. If vertical supports are exposed in front of the wall, they are called buttresses and are compression members. In either case, the main wall is considered as a continuous member supported at each cross wall.

Counterfort and buttress retaining walls are used to retain soil up to 25 ft.

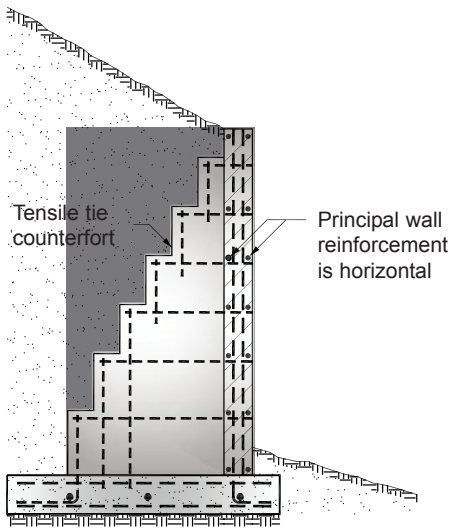


FIGURE 13.3 Counterfort retaining wall.

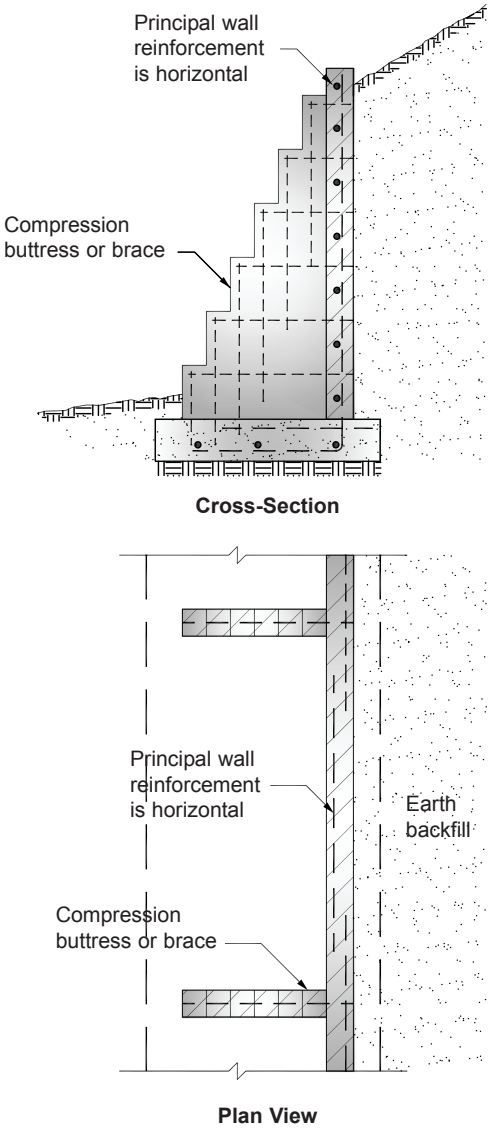


FIGURE 13.4 Buttress retaining wall.

13.2.3 CANTILEVER RETAINING WALLS

These walls are so named because the vertical stem wall is designed to cantilever from the base. The tension stresses which develop on the wall stem are resisted by reinforcing steel placed on the soil side (tension face) of the wall. The base resists sliding, overturning and rotating due to the lateral loading and must also be large enough to ensure that the bearing capacity of the soil is not exceeded.

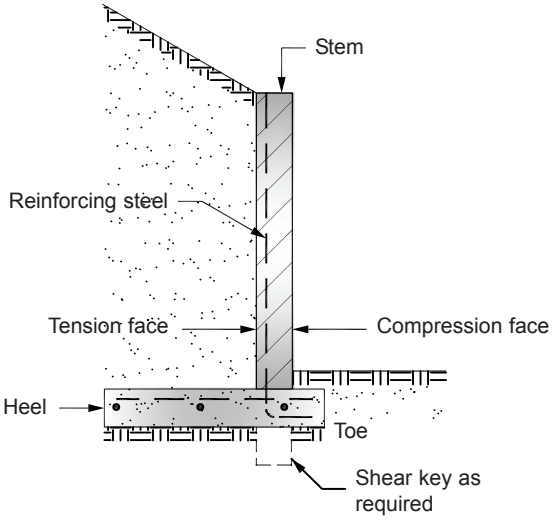


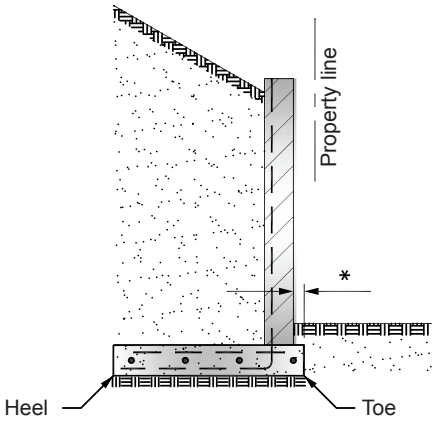
FIGURE 13.5 Cantilever retaining wall.

Cantilever retaining walls can be designed without a footing heel and are especially useful in limited space areas such as near property lines and existing utility lines. These walls require special attention to assure they can resist the lateral earth forces and overturning moment through their own weight and strength. Since there is no footing heel on most property walls, there is also no resisting soil mass and thus the wall foundation may be especially large and heavy.

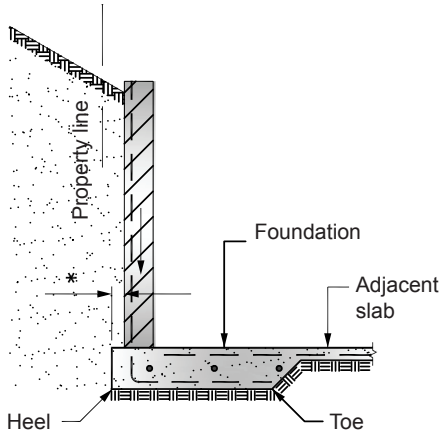
To reduce the need for such a large foundation, adjacent slabs are often tied into the foundation. These slabs help resist sliding forces and contribute somewhat to reducing the soil bearing pressure and the overturning forces.

13.2.4 SUPPORTED WALLS

Walls of basements and subterranean garages are often laterally supported at their tops by floor systems (Figure 13.7).



(a) *Retaining wall adjacent to property line.*



(b) *Retaining wall adjacent to property line with foundation tied into floor slab to increase the sliding resistance.*

* Provide at least a 2 in. footing extension to allow for construction tolerances

FIGURE 13.6 Property line type retaining walls.

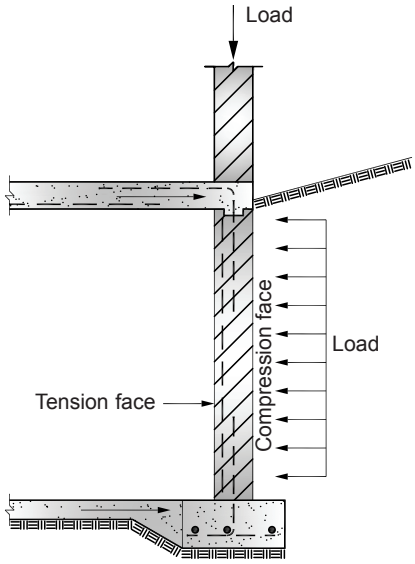
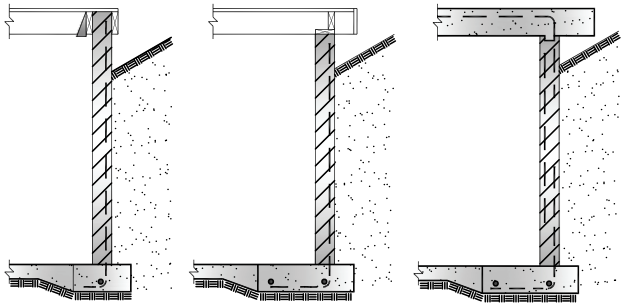


FIGURE 13.7 Supported retaining wall.

Depending on the type of support provided by the floor and foundation systems, a supported wall could be considered having either a fixed top and bottom, a fixed base with simply supported top or a simply supported top and bottom. Each wall type must be designed and reinforced accordingly. Continuity of the connections at the top and the bottom must be developed by proper reinforcement in order to provide the required degree of fixity.



(a) *Simply supported top and bottom*

(b) *Simply supported top: fixed at bottom*

(c) *Fixed at top and bottom*

FIGURE 13.8 Supported retaining walls with various end conditions.

Basement or subterranean garage walls are often subjected to both vertical and lateral loads since these perimeter walls support the building above as well as resist the earth pressure. The combined wall loading, vertical load plus lateral load, must be considered in the design of the wall.

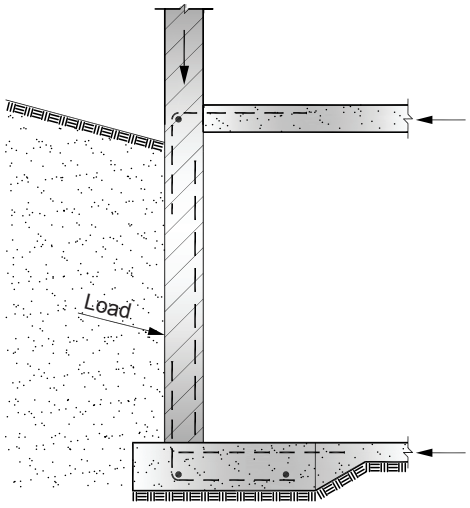


FIGURE 13.9 Vertical and lateral loads on a supported retaining wall.

13.3 DESIGN OF RETAINING WALLS

Masonry retaining walls may be designed by one of two methods. They can be designed as masonry walls using allowable masonry stresses or they can be designed assuming that masonry is a form for the concrete grout and allowable concrete stresses are used in the stress analysis.

Both design methods depend on loads imposed, height of the wall, limiting thickness of the wall, construction procedure and economics of the finished wall.

The resultant of the forces on a retaining wall should fall within the middle third of the footing. This results in the most efficient use of the footing. However, this general recommendation is not mandatory and the resultant force may fall outside of the middle third, provided the resulting soil pressure does not exceed the allowable soil bearing pressure and the overturning moment does not exceed the resisting moment within the allowable factor of safety.

13.3.1 EFFECT OF CORNERS ON LATERAL SUPPORTING CAPACITY OF RETAINING WALLS

Retaining walls that change direction create a condition of increased strength at the corner. Each direction of wall will support the opposite wall for a certain distance (Figure 13.10). This condition is common in basements and underground garages.

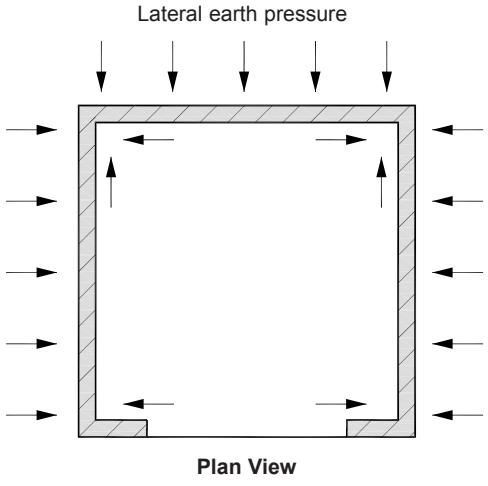


FIGURE 13.10 Intersecting walls brace retaining walls.

If the length of the wall is not more than about 3 times the height, the wall can be designed by a procedure similar to the one presented by the Portland Cement Association in its publication *Rectangular Concrete Tanks* (IS003). Using this procedure, walls may have four conditions of support:

1) fixed both sides, hinged top and bottom,
2) fixed both sides and bottom, free at top,
3) fixed both sides, hinged at bottom and free at top, and
4) hinged at all four edges.

Loading on the wall is assumed to be a lateral triangular earth load plus any surcharge load and possibly a vertical load due to the structure above. If the length of the wall is greater than 3 times its height, the influence of the corner is significantly reduced and it is usually considered effective over a distance equal to the height of the wall.

13.3.2 PRELIMINARY PROPORTIONING OF RETAINING WALLS

Retaining walls may be designed by initially selecting tentative dimensions which are analyzed for stability and structural requirements and then revised as required. Since this is a trial and error process, various solutions to the problem may be obtained, all of which may be satisfactory. Software specifically for retaining wall design is also available. One of the user-friendly programs is available from www.retainpro.com.

Dimensions for a retaining wall must be adequate for the structural stability of the wall and must also satisfy local building code requirements. Tentative dimensions of cantilever retaining walls are shown in Figure 13.11 and are based in part on experience of satisfactorily constructed walls. They may be used for the initial design proportions, although they can result in overly conservative designs.

The stem is typically constructed with masonry units and the base of the stem should be thick enough to satisfy shear requirements without the use of shear reinforcing steel.

The base-slab dimensions should be such that the resultant of the vertical loads falls within the middle one-third of the wall (Figure 13.12). If the resultant falls outside the middle one-third, toe pressures may be excessively large and only part of the footing will be effective.

When the resultant falls within the middle third, no uplift will occur and the following equations may be used for design:

$$\text{Eccentricity, } e = \frac{l}{2} - \left(\frac{M_R - OTM}{\Sigma W} \right), \text{ and}$$

$$\text{Soil Pressure} = \frac{W}{A} \pm \frac{We}{S}$$

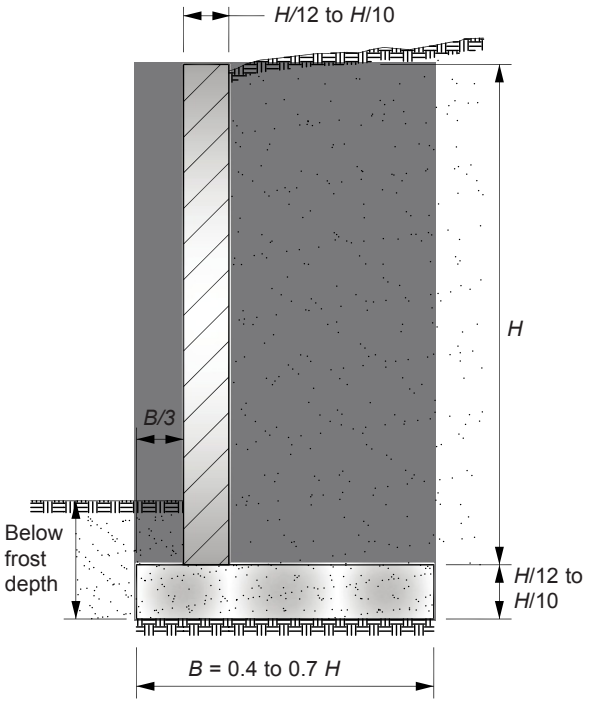


FIGURE 13.11 Initial design dimensions for a cantilever retaining wall.

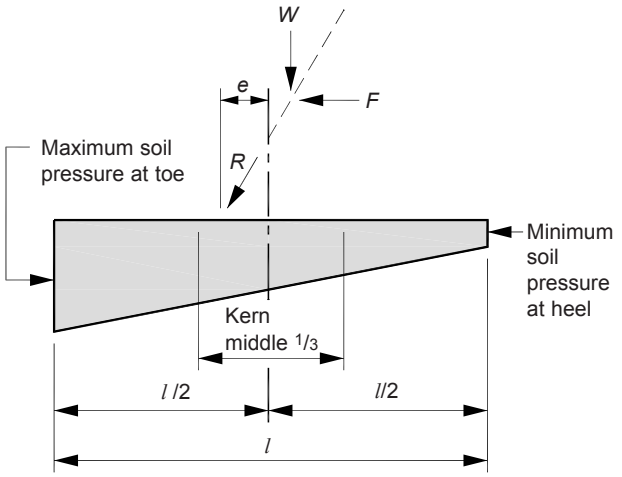


FIGURE 13.12 Soil bearing pressure distribution, resultant within middle third of the wall.

When the resultant of the forces falls outside the middle third, the above equations do not apply since only a portion of the footing is effective (Figure 13.13). For this condition, moments may be taken about the toe of the footing to find:

$$\text{Eccentricity, } e = \frac{l}{2} - \left(\frac{M_R - OTM}{\Sigma W} \right)$$

$$\text{Length of compress area} = l' = 3\left(\frac{l}{2} - e \right)$$

$$\text{Maximum soil pressure} = 2\left(\frac{\Sigma W}{l'} \right)$$

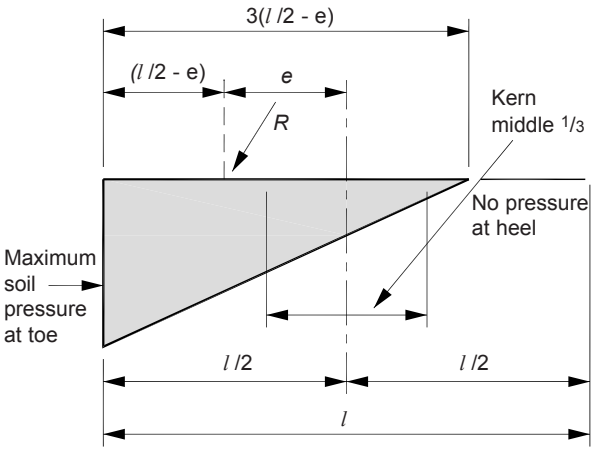


FIGURE 13.13 Soil bearing pressure distribution, resultant outside of middle third.

The stabilizing resisting moment should be at least 1½ times greater than the overturning moment and the allowable soil bearing pressure must not be exceeded. If the maximum soil pressure exceeds the allowable soil pressure, the toe or heel may be extended to decrease the eccentricity, *e*, and to increase the length of compression area.

Finally, the design of a retaining wall system must provide stability of the wall against sliding. Depending on the foundation soil, which may not be the same as the backfill material, the foundation may require a greater depth of material at the toe of the base or may require a key, which effectively provides additional toe depth, to provide enough resistance to sliding of the foundation. Static friction between the wall system may be considered with the soil beneath the base resisting lateral forces. The stabilizing resisting forces from friction and passive toe pressures should be at least 1½ times greater than lateral forces on the wall system.

Based on the estimated wall dimensions, carrying out stability analysis prior to wall design may save significant time. In cases where there are surcharges behind the wall or there is a significant backfill slope, lateral loads are dependent on the base dimension, which is determined in the stability analysis. With a significant backfill slope, the base dimension required can exceed the 0.7H upper limit. This is the case in the following design example.

13.4 CANTILEVER RETAINING WALL DESIGN EXAMPLE

13.4.1 DESIGN CRITERIA

Determine the required thickness and reinforcement in a retaining wall for balanced design conditions when constructed with:

Part 1 – Two wythe, grouted clay brick masonry wall: f'_m = 2500 psi.

Part 2 – Concrete block masonry wall; f'_m = 1500 psi.

Check the footing and other conditions of design for the concrete masonry stem as determined in Part 2.

The design for the cantilever retaining wall is based on a per foot length of wall.

Retaining wall – 10 ft high

Backfill slope – 1:3

Backfill soil – course-grained soil of low permeability due to admixture of particles of silt size, w_s = 110 pcf (Type 2 Soil)

Masonry – design of wall using two different masonry requirements.

Steel – Grade 60 steel, f_s = 24,000 psi, E_s = 29,000,000 psi

Footing – concrete, f'_c = 2500 psi

Lateral Pressure – Equivalent Fluid Pressure.

Soils analysis provides the following for given soil conditions: k_h = 40 pcf and k_v = 12 pcf.

For level backfill, soils analysis for horizontal pressure per foot of depth provides k_h = 36 pcf, and density, w_s = 110 pcf, to determine an active pressure coefficient.

$$K_a = \frac{k_h}{\gamma} = \frac{36}{110} = 0.327$$

Since $K_p = 1/K_a$, we can determine a passive pressure coefficient K_p = 3.06, and using an undisturbed soil density this results in a passive horizontal pressure per foot of depth, K_p = 367 pcf. Undisturbed soil can have a greater lateral resistance based on the angle of internal friction ϕ. The angle of internal friction ϕ of the undisturbed soil for this problem will be presumed to be 30° and have an undisturbed w_s = 120 pcf.

In order to use these values, weep holes or a drainage system must be provided to prevent hydrostatic pressure from building up behind the wall. These holes or systems must be located near the bottom of the wall or at the ground surface.

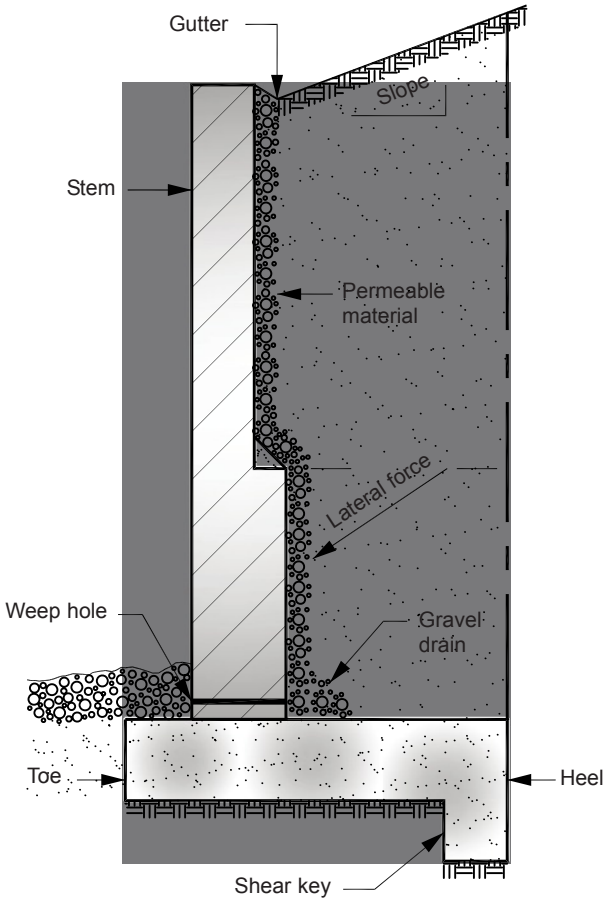


FIGURE 13.14 Cross-section of cantilever retaining wall.

13.4.2 STEM DESIGN

13.4.2.1 BRICK WALL STEM

For balanced design:

$f_b = f'_m/3 = 2500/3 = 833$ psi

f_s = 24,000 psi

Use a minimum width at the top of 10 in. to provide approximately 3 in. of grout space.

t = 10 in., use d = 5.5 in.

Enter Table ASD-26a for f'_m = 2500 psi; and n = 16.6. Balanced conditions occur when f_b = 833 psi and f_s = 24,000 psi:

K_b = 133.7; ρ_b = 0.00634

Balanced moment

$$M_b = \frac{K_b b d^2}{12} = \frac{(133.7)(12)(5.5)^2}{12}$$

= 4,044 ft lbs / ft

(Note, Table ASD-38 could also have been used to find M_b = 4.04 ft kips/ft).

Find the height of backfill where the actual moment will exceed M_b. For a cantilevered wall with a triangular (equivalent fluid) soil surcharge:

$$\text{Moment} = k_h \frac{h^3}{6} = 4{,}044 \text{ ft lbs/ft}$$

$$h = \sqrt[3]{\frac{6M}{k_h}} = \sqrt[3]{\frac{6(4{,}044)}{40}}$$

h = 8.47 ft

This is equivalent to 25 courses of brick with a 4 in. vertical module totaling 8.333 ft (8 ft - 4 in.).

$A_s = \rho_b b d$ = 0.00634(12)(5.5)

= 0.418 sq in./ft

From Table GN-20c

Use #6 at 12 in. o.c., A_s = 0.442 sq in./ft

From Table GN-22a, the basic development length for a Grade 60 #6 bar is 30.42 inches. Bar development length required would be:

$$l_d = l_{d(basic)}\left(\frac{A_{s(required)}}{A_{s(provided)}}\right) = (30.42)\left(\frac{0.418}{0.442}\right) = 28.8 \text{ in.}$$

Thus, given the height above the footing at which the bars are required to resist the flexural tension, the #6 bars must extend into the foundation at least:

$$28.8 - (10 - 8.333)(12) \approx 9 \text{ in.}$$

The minimum horizontal steel area required is

$A_s = 0.0007bt$

From Table GN-21a, #4 bars must be spaced no further apart than 29 in. o.c.

Use a more practical spacing of 24 in. o.c.

Determine the required wall thickness and reinforcement at the base of the wall where the moment is at its maximum. The moment at base is:

$$M = \frac{k_h h^3}{6} = \frac{(40)(10)^3}{6} = 6{,}667 \text{ ft lbs/ft}$$

$$bd^2 = \frac{M}{K_b} = \frac{(6{,}667)(12)}{133.7} = 599$$

Since b = 12 in.

$$d = \sqrt{\frac{599}{12}} = 7.06 \text{ in.}$$

Try t = 12.0 in. with d = 7.5 in.

$A_s = \rho_b bd = 0.00634(12)(7.5)$

= 0.57 sq in./ft

From Table GN-20c:

Use #7 at 12 in. o.c., (A_s = 0.601 sq in./ft)

This spacing matches the spacing of the reinforcement in the upper section of the wall, but requires significant embedment to develop the bar. From Table GN-22a, the basic development length for a Grade 60 #7 bar is 35.5 inches. Instead, use #5 bars at 6 in. o. c. from GN-20b (A_s = 0.62 sq in./ft). This allows every other bar to align with the bars in the upper portion of the wall and the required extension above the foundation, from Table GN-22a, is only 19.5 inches. A standard hook provides 7 in., so only an additional 19.5 - 7 = 12.5 in., embedment is required in the foundation.

The #5 hooked bars embedded in the foundation can be extended higher in the wall to eliminate the need for the #6 bars to be anchored into the foundation. The #5 bars would extend from the foundation for approximately the development length required of the #6 bars $l_d \approx 29$ in. to avoid the need to anchor the #6 bars into the foundation. This distance is conservative because the need for the #6 bar decreases as the transition in wall thickness is moved upward.

As an alternative the wall could be made 12 in. thick for the full height of the wall. In this case every other bar could be stopped at a height above the foundation where #5 bars at 12 in. o. c. is sufficient reinforcement. The level of reinforcement at this point will not provide a balanced condition, but the tension reinforcement will control the design.

$$\rho = \frac{0.31}{12(7.5)} = 0.00344$$

K_f can be determined by interpolation from Table ASD-26a and ρ as K_f = 74.82. The moment capacity is thus:

$$M = \frac{K_f bd^2}{12} = \frac{74.82(12)(7.5)^2}{12} = 4209 \text{ ft lb/ft}$$

The moment equals this value at a height h:

$$h = H - \sqrt[3]{\frac{6M}{K_h}} = 10 - \sqrt[3]{\frac{6(4208)}{40}}$$

= 1.422 ft (17.1 in.) < l_d = 19.5 in. (so use h = 20 in.)

The minimum horizontal steel area requirement for a 12 in. section is:

$A_s = 0.0007bt = 0.0007(12)(12) = 0.1008$ sq in./ft

Use #4 bars at 24 in. o.c. (A_s = 0.1 sq in./ft)

For the original solution with a change of section from 10 in. thickness for the top 8 ft - 4 in. of height to 12 in. thickness for the bottom 1 ft - 8 in., if the lateral force was large enough, the shear could split the wall vertically at the point of change of section. Determine the maximum shear force at that point.

Shear on wall

Shear 8 ft - 4 in. from top

$$V = \frac{k_h h^2}{2} = \frac{(40)(8.333)^2}{2} = 1389 \text{ lbs/ft}$$

Assume the shear is resisted by the grout and masonry where b = 12 in., d = 5.50 in. and t = 10 in.

$$v = \frac{V}{bd} = \frac{1389}{(12)(5.5)} = 21.04 \text{ psi}$$

From Table ASD-3 Allowable Stresses:

Allowable v_m = 50 psi > 21.04 psi OK

Shear at the base of the stem, at the footing:

$$V = \frac{k_h h^2}{2} = \frac{(40)(10)^2}{2} = 2{,}000 \text{ lbs/ft}$$

Assume shear is resisted by grout and masonry where b = 12 in., t = 12 in. and d = 7.5 in.

$$v = \frac{V}{bd} = \frac{2000}{12(7.5)} = 22.22 \text{ psi}$$

Allowable v_m = 50 psi > 22.22 psi OK

If the actual shear exceeds the allowable shear strength of the masonry, provide a shear key similar to those shown in Figure 13.15 and use concrete shear capacity values.

Reinforcement may be reduced because of the increased depth, d, of the deeper section:

$$K_f = \frac{M}{bd^2} = \frac{(6{,}667)(12)}{12(7.5)^2} = 118.5$$

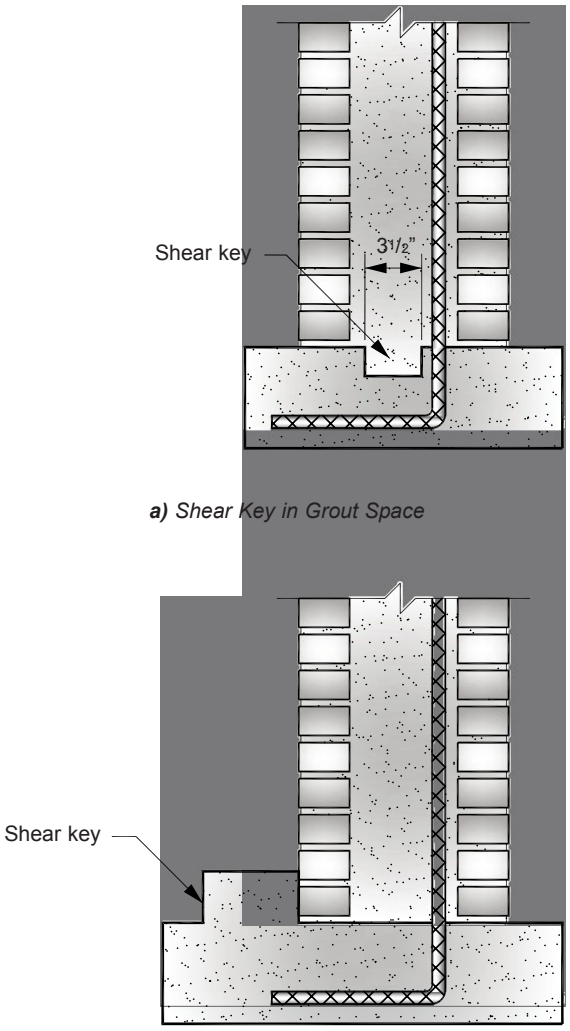
From Table ASD-26a for K_f = 118.5, interpolate to find ρ = 0.00559

$A_s = \rho bd$ = 0.00559(12)(7.5) = 0.503 sq in./ft

From Table GN-20b:

Use #5 at 6 in. o. c. (A_s = 0.62 sq in./ft)

The 6 in. spacing is used rather than the possible 7 in. spacing so that every other bar matches the reinforcement above the change in section size. Alternately, the same bar size and wall thickness could be used for the full height of the wall, that is, a 12 in. wall with #5 bars at 7 in. o. c. spacing at a depth d = 7.5 in. for the full height of the wall, or optionally as determined above every other #5 bar which could be terminated at a height of 20 inches.



a) *Shear Key in Grout Space*

b) *Shear Key in Front of Wall*

FIGURE 13.15 Connection of wall with footing.

The shear on a key shown in Figure 13.15a is:

$$f_v = \frac{V}{bd} = \frac{2000}{(12)(3.5)} = 47.6 \text{ psi}$$

The allowable

$$F_v = \sqrt{f'_m} = \sqrt{2500} = 50 \text{ psi} > 47.6 \text{ psi}$$

(MSJC Code Section 2.3.5.2.2)

Thus, the shear capacity of the 3.5 in. key is sufficient.

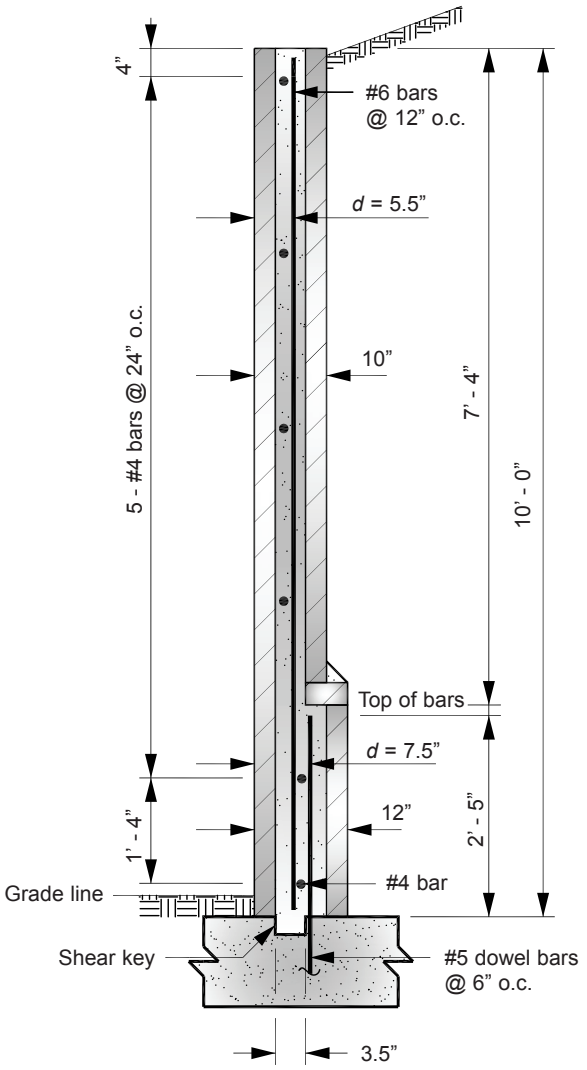


FIGURE 13.16 Detail of wall for Part (1), brick masonry stem, balanced design.

13.4.2.2 CONCRETE MASONRY STEM

Design the concrete masonry wall stem and determine the reinforcement for a wall with f'_m = 1500 psi.

Use balanced design conditions:

f_b = 500 psi and f_s = 24,000 psi. (Grade 60 steel).

From Table ASD-24b,

n = 21.5, K_b = 69.3 and ρ_b = 0.00322

Try 8 in. CMU for top section and place reinforcement approximately 1 in. inside the 1¼ in. face shell.

t = 7.625 in. and d = 5.375 in.

Through computation of depth of reinforcement, maintaining a ½ in. cover, the 'd' distance can be increased to 5.5 in., as long as the reinforcement is limited to #6 bars or smaller.

Balanced moment

$$M_b = K_b b d^2 = \frac{(69.3)(12)(5.5)^2}{12} = 2{,}096 \text{ ft lbs/ft}$$

$$\text{Moment} = k_h \frac{h^3}{6} = 2{,}096 \text{ ft lbs/ft}$$

$$h = \sqrt[3]{\frac{6M}{k_h}} = \sqrt[3]{\frac{6(2{,}096)}{40}}$$

h = 6.8 ft (81.6 in.) ≈ 80 in.

This is equivalent to 10 courses of 8 in. high CMU (80 in.)

$A_s = \rho_b b d$ = 0.00322(12)(5.5) = 0.2125 sq in./ft

From Table GN-20c

Use #5 at 16 in. o.c. (A_s = 0.230 sq in./ft)

Horizontal steel

From Table GN-21a, Minimum A_s = 0.064 sq in./ft

Use #4 bars @ 36 in. o. c.

For #4 bars, the spacing required would be 36 in. or less (32 in. for modular spacing). To obtain the spacing of 36 in. o. c., bond beam blocks may be inverted as shown in Figure 13.17. One bond beam will be required at the top of the wall (MSJC Code Section 1.14.2.2.2.1), one bond beam will be inverted 32 in. down from the top of the wall, and another bond beam will be required at the bottom of the wall section (72 in. from the top of wall).

Moment at base

M = 6,667 ft lbs/ft (from Part 1)

$$bd^2 = \frac{M}{K_b} = \frac{(6{,}667)(12)}{69.3} = 1155$$

Since b = 12 in.,

$$d = \sqrt{\frac{1155}{12}} = 9.81$$

Use 12 in. CMU, d = 11.63 in. - 1.5 in. - 1 in. $\approx$ 9 in.

$$K_f = \frac{M}{bd^2} = \frac{6{,}667(12)}{(12)(9)^2} = 82.31$$

From Table ASD-24b, for n = 21.5 and K_f = 82.31, interpolate to find:

ρ = 0.00529

From Table GN-23g, for d = 9 in. and ρ = 0.0053

Use #6 at 8 in. o. c. (A_s required = 0.00529(8)(9) = 0.381 < 0.44 sq in.)

Check if #5 bars can be used because of the embedment issue raised above. A larger d could reduce the A_s requirement and make the design closer to the balanced condition.

d = 11.63 in. - 1.5 in. (face shell) - 0.5 in. (grout cover, MSJC Code Section 1.13.3.5) - 1/2 (0.63 in.) = 9.32 in.

Use d = 9.25 in. (works for #6 bar also)

$$K_f = \frac{M}{bd^2} = \frac{6{,}667(12)}{(12)(9.25)^2} = 77.92$$

From Table ASD-24b, for n = 21.5 and K_f = 77.92, interpolate to find:

ρ = 0.00449

From GN-23g with d = 9.25, #5 bar and spacing = 8 in., ρ = 0.0042 < 0.00449, so not enough reinforcement is provided. The area required could also be checked:

A_s required = 0.00449(8)(9.25) = 0.333 > 0.31 sq in. NO GOOD

Use #6 @ 8 in. o. c. (at a depth of 9.25 in.)

Shear on wall

Shear 6 ft - 8 in. from top

$$V = \frac{k_h h^2}{2} = \frac{(40)(6.667)^2}{2} = 889 \text{ lbs/ft}$$

Assume the shear is resisted by the grout and masonry where b = 12 in., d = 5.5 in. and t = 7.63 in.

$$f_v = \frac{V}{bd} = \frac{889}{(12)(5.5)} = 13.47 \text{ psi}$$

From Table ASD-3 Allowable Stresses:

F_v = 38.73 psi > 13.47 psi OK

Shear at the base of the stem, at the footing:

$$V = \frac{k_h h^2}{2} = \frac{(40)(10)^2}{2} = 2{,}000 \text{ lbs/ft}$$

Assume shear is resisted by grout and masonry where b = 12 in., t = 11.63 in. and d = 9.25 in.

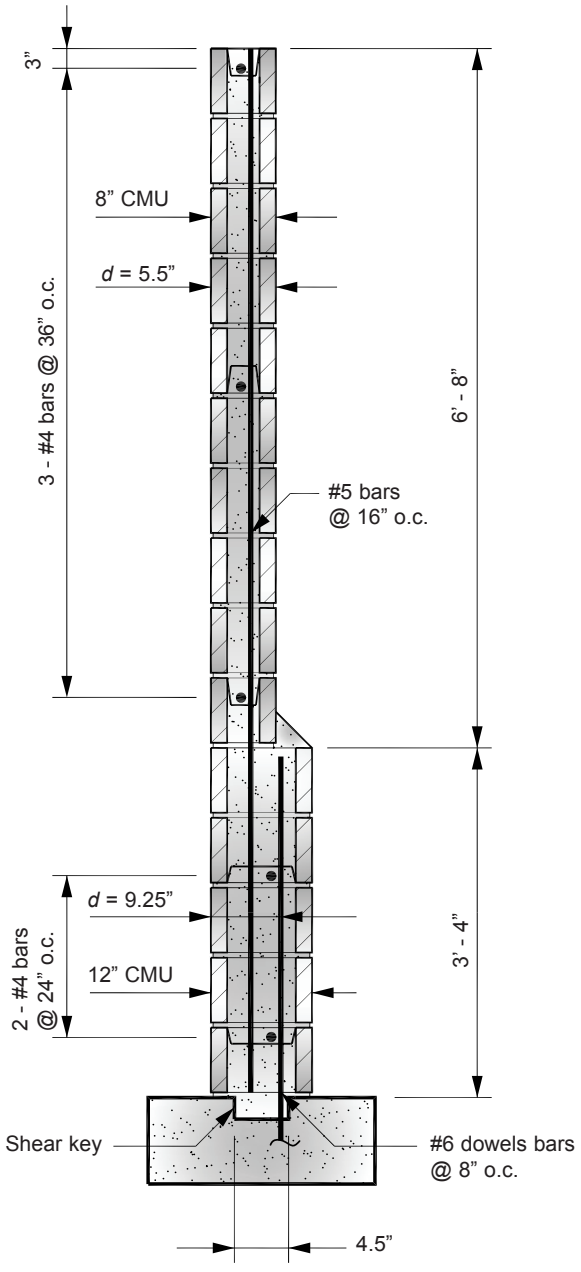


FIGURE 13.17 Detail of wall for Part 2, concrete block masonry.

$$f_v = \frac{V}{bd} = \frac{2000}{12(9.25)} = 18.02 \text{ psi}$$

F_v = 38.73 psi > 18.02 psi OK

The shear on a key shown in Figure 13.15a will be more critical due to the lower masonry strength f'_m. The allowable $F_v = \sqrt{f'_m} = \sqrt{1500} = 38.73$ psi (MSJC Code Section 2.3.5.2.2)

$$d = \frac{V}{bf_v} = \frac{2000}{(12)(38.73)} = 4.3 \text{ in.}$$

Thus, based on the shear capacity, a 4.5 in. shear key is sufficient.

Horizontal Steel

From Table GN-21a, Minimum A_s = 0.098 sq in./ft

Use #4 at 24 ft o. c.

13.4.3 FOOTING DESIGN

13.4.3.1 SOIL BEARING AND OVERTURNING

The concrete masonry stem, as designed in Section 13.4.2.2, will be used to complete the design of the retaining wall.

For the most efficient footing design, it is advisable to have the resultant of forces fall within the middle third of the footing. Using retaining wall proportions from Figure 13.11, the width of the base is between 4 and 7 feet. By trial a footing with a width, B, of 6 feet results in a stable wall with appropriate safety factors and according to the following computations. The toe width of B/3 = 2 ft results from the same proportions. From this configuration, the resulting dimensions are shown in Figure 13.18, as well as, zones or components (circled numbers) that relate to the forces acting on and associated with the wall.

Lateral earth pressure above toe, resulting from component 8:

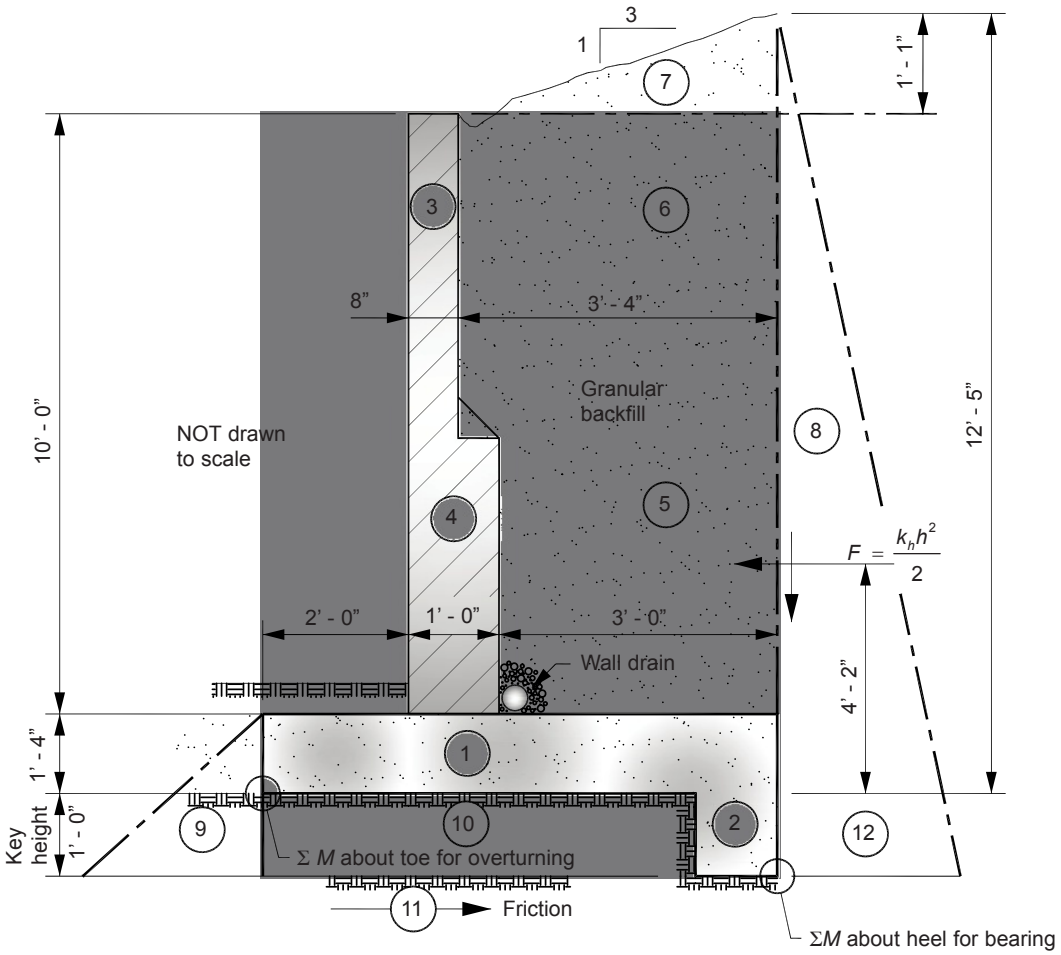


FIGURE 13.18 Forces on wall.

$$F = F_8 = \frac{k_h h^2}{2} = \frac{40(12.412)^2}{2} = 3{,}081 \text{ lbs/ft}$$

A shear key may be provided to add lateral resistance of the retaining wall system in order to prevent sliding. The soil pressures resisting the motion of the shear key can reach passive pressure levels if the lateral forces exceed the friction on the bottom of the footing, such that there is a small amount of movement of the footing. The undisturbed sub-base volume of Component 10 then contributes to the weight of the wall system to produce the friction of Component 11 in Figure 13.18.

The pressures on the shear key add to the overturning moment, but cannot exceed the lateral forces acting upon the wall. Table 13.1 includes a shear key that is one foot deep and one foot wide. Lateral forces, F_L, acting upon the retaining wall include only forces resulting from Components 8 and 12 in Figure 13.18. Lateral forces are resisted by the forces resulting from Components 9 and friction force 11. Soil above the footing on the left side of the wall in Figure 13.18 is ignored.

The maximum horizontal resistance, F_R, is:

$F_R = F_9 + F_{11}$

The resistance forces must equal the lateral forces, F_L, which is: $F_L = F_8 + F_{12}$

$$F_{12} = k_h h h_{key} + \frac{k_h h_{key}^2}{2} = 496 + 20 = 516 \text{ lbs/ft}$$

(Table 13.1 Component 12)

F_L = 3,081 + 516 = 3,597 lbs/ft

The footing sliding resistance on the sub-base is μN with μ being the coefficient of friction (0.35 for a concrete footing and a gravel sub-base from Table 13.2) and N being the normal force (sum of vertical forces):

$F_{\text{footing friction}}$ = 0.35(7,118) = 2,491 lbs/ft < 3,597 lbs/ft

Thus, movement will occur resulting in passive resistance:

$$F_9 = \frac{k_p h_{ftg}^2}{2} + k_p h_{ftg} h_{key} + \frac{k_p h_{key}^2}{2}$$

$$= 367\left[\frac{(1.333)^2}{2} + 1.333(1) + \frac{(1)^2}{2}\right] = 999 \text{ lbs/ft}$$

TABLE 13.1 Resisting Moment (Σ Moments at Toe of Footing)

Item	Factor		Height ft		Width ft		Unit Pressure psf		Weight lbs/ft		Moment Arm ft		Resisting Moment ft lbs/ft
(1) Footing			1.333	x	6	x	150	=	1,200	x	3.00	=	3,600
(2) Key			1.0	x	1.0	x	150	=	150	x	5.50	=	825
(3) Wall			6.667	x	0.636	x	120	=	508	x	2.32	=	1,178
(4) Wall			3.333	x	0.969	x	120	=	388	x	2.48	=	963
(5) Earth			3.333	x	3.031	x	110	=	1,111	x	4.48	=	4,984
(6) Earth			6.667	x	3.365	x	110	=	2,467	x	4.32	=	10,653
(7) Earth	1/2	x	1.083	x	3.365	x	110	=	200	x	4.88	=	976
					Total weight = 6,024								
(8)+(12) Vertical component of lateral earth pressure	1/2	x	13.412	x	13.412	x	k_v = 12	=	1,079	x	6.00	=	6,476
Total vertical force = 7,103													
(9) Horizontal resistance above toe	1/2	x	1.333	x	1.333	x	k_p = 367*	=	326	x	0.444	=	145
(12) Horizontal component of lateral earth pressure on key below toe			12.412	x	1	x	k_h = 40	=	496	x	0.5	=	248
	1/2	x	1	x	1	x	k_h = 40	=	20	x	0.667	=	13
Total Resisting Moment Σ*M*												=	**30,061**

*Passive earth pressure

TABLE 13.2 Allowable Foundation and Lateral Pressure (IBC Table 1804.2)

CLASS OF MATERIALS	ALLOWABLE FOUNDATION PRESSURE (psf)[4]	LATERAL BEARING (psf/f below natural grade)[4]	LATERAL SLIDING	
			Coefficient of friction[1]	Resistance (psf)[2]
1. Crystalline bedrock	12,000	1,300	0.70	–
2. Sedimentary and foliated rock	4,000	400	0.35	–
3. Sandy gravel and/or gravel (GW and GP)	3,000	200	0.35	–
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	–
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500[3]	100	–	130

For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m.

1. Coefficent to be multiplied by the dead load.
2. Lateral sliding resistance value to be multiplied by the contact area, as limited by IBC Section 1804.3.
3. Where the building official determines that in-place soils with an allowable bearing capacity of less than 1,500 psf are likely to be present at the site, the allowable bearing capacity shall be determined by a soils investigation.
4. An increase of one-third is permitted when using the alternate load combinations in IBC Section 1605.3.2 that include wind or earthquake loads.

Remaining resistance will be from the friction at the base of the wall system comprised of key-soil friction and soil-soil friction, and will depend on the bearing pressures. To avoid a sliding failure, the resisting force F_{11} must be at least:

$F_{11} = F_L - F_9 = 3{,}597 - 999 = 2{,}598$ lbs/ft

Overturning moment, assuming sliding failure does not occur:

$$OTM = \frac{k_h h^3}{6} + \frac{k_p h_{ftg} h_{key}^2}{2} + \frac{k_p h_{key}^3}{3} + F_{11} h_{key}$$

$$= \frac{40(12.412)^3}{6} + \frac{1.333(367)(1)^2}{2} + \frac{(367)(1)^3}{3} + 2{,}598(1)$$

= 15,713 ft lbs/ft

Overturning safety factor (M_R from Table 13.1)

$$= \frac{M_R}{OTM} = \frac{30{,}061}{15{,}713} = 1.91 > 1.5 \text{ OK}$$

Eccentricity

$$= \frac{L_{ftg}}{2} - \left[\frac{M_R - OTM}{\Sigma Wt}\right]$$

$$= \frac{6}{2} - \left[\frac{30{,}061 - 15{,}713}{7{,}103}\right] = 0.98 \text{ ft}$$

$$\text{Third point} = \frac{L_{ftg}}{6} = \frac{6}{6} = 1.0 \text{ ft}$$

Eccentricity, e = 0.98 ft < 1.0 ft, and thus is within the middle third.

Thus, there will be no uplift on the footing.

$$\text{Soil pressure under footing} = \frac{Wt}{L_{ftg}} \pm \frac{6(Wt)(e)}{L_{ftg}^2}$$

$$= \frac{7{,}103}{6} \pm \frac{6(7{,}103)(0.98)}{6^2} = 1{,}183.8 \pm 1{,}160.2 \text{ psf}$$

= 2,344 psf maximum

= 24 psf minimum

Allowable soil bearing = 3000 psf OK

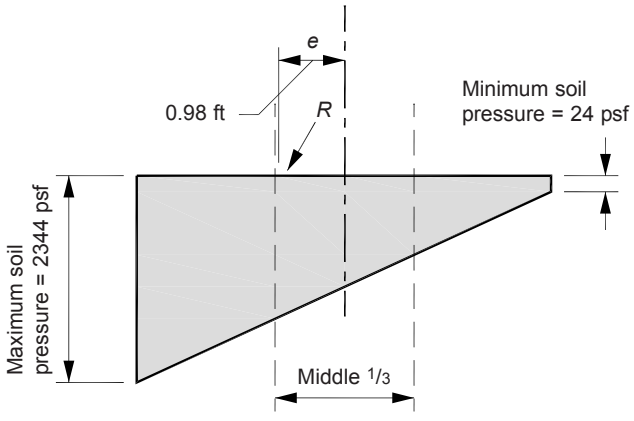


FIGURE 13.19 Soil pressure under footing.

13.4.3.2 SLIDING

Retaining wall stability analysis must consider sliding of the retaining wall. In determining the overturning pressures, consider movement of the footing through sliding of the concrete footing on the gravel sub-base. Sliding of the foundation and the soil in front of the shear key is possible if there is not sufficient base friction across the soil boundary indicated by the friction Component 11 in Figure 13.18.

Base friction within the soil is based on the angle of internal friction ϕ:

Coulomb soil shearing strength = $N \tan \phi$

For $\phi = 30°$; tan 30° = 0.5774

$N = \sum F_{Vertical}$ = Total Vertical Force (from Table 13.1) + Weight of soil in front of key (Component 10)

N = 7,103 + 120(5)(1) = 7,703 lbs/ft

Due to low bearing pressure at the heel end of the footing almost all of the normal pressure occurs within the soil-soil sliding Component 11 developing the F_{11} resistance force.

Coulomb soil shearing strength = 7,703(0.5774) = 4,448 lbs/ft > 2,619 lbs/ft OK (greater resistance than required)

Sliding safety factor

$$= \frac{F_9 + F_{11}}{F_8 + F_{12}} = \frac{999 + 4{,}448}{3{,}598} = \frac{5{,}477}{3{,}598} = 1.514 \text{ OK}$$

Checking the bearing pressures at the base of the shear key requires summation of forces at that level and summation of moments about the heel of the shear key.

$$\sum F_{Vertical} = 7{,}703 \text{ lbs/ft}$$

$$\sum F_{Resist} = 30{,}411 \text{ ft lbs/ft} \qquad \text{(Table 13.3)}$$

The resulting toe and heel pressures are:

$$= \frac{7{,}703}{6} \pm \frac{6(7{,}703)(0.9566)}{6^2} = 1{,}283.8 \pm 1{,}228.1 \text{ psf}$$

= 2,512 psf maximum < 3,000 psf OK

= 56 psf minimum

TABLE 13.3 Bearing Moment (Σ Moments at Heel of Key)

Item	Factor		Height ft		Width ft		Unit Pressure psf		Weight lbs/ft		Moment Arm ft		Resisting Moment ft lbs/ft
(1) Footing			1.333	x	6	x	150	=	1,200	x	3.00	=	3,600
(2) Key			1.0	x	1.0	x	150	=	150	x	0.50	=	75
(3) Wall			6.667	x	0.636	x	120	=	508	x	3.68	=	1,872
(4) Wall			3.333	x	0.969	x	120	=	388	x	3.52	=	1,362
(5) Earth			3.333	x	3.031	x	110	=	1,111	x	1.52	=	1,685
(6) Earth			6.667	x	3.365	x	110	=	2,467	x	1.68	=	4,151
(7) Earth	1/2	x	1.083	x	3.365	x	110	=	200	x	1.12	=	224
(10) Undisturbed Soil			1	x	5	x	120	=	600	x	3.5	=	2,100
					Total weight = 6,624								
(8)+(12) Vertical component of lateral earth pressure	1/2	x	13.412	x	13.412	x	k_v = 12	=	1,079	x	0	=	0
Total vertical force = 7,703													
(8)+(12) Horizontal component of lateral earth pressure	1/2	x	13.412	x	13.412	x	k_h = 40	=	3,598	x	4.48	=	16,119
(9) Horizontal resistance above toe	1/2	x	2.333	x	2.333	x	k_p = -367*	=	-999	x	0.78	=	-777
(11) Friction								=	2,599	x	0	=	0
Total Bearing Moment Σ*M*												=	**30,411**

*Passive earth pressure

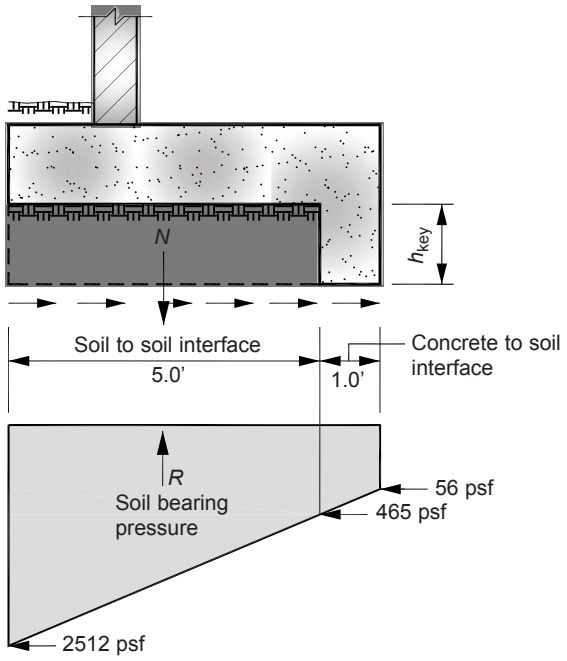


FIGURE 13.20 Pressures at base of shear key for computing friction.

The friction forces resisting sliding:

$$F_{11} = (\mu N)_{soil-soil} + (\mu N)_{concrete-sub-base}$$

$$= 0.5774(5)\left(\frac{2{,}512 + 465}{2}\right) + 0.35(1)\left(\frac{465 + 56}{2}\right)$$

$$= 4{,}297 + 91 = 4{,}388 \text{ lbs/ft}$$

Sliding safety factor

$$= \frac{F_9 + F_{11}}{F_8 + F_{12}} = \frac{999 + 4{,}388}{3{,}599} = \frac{5{,}387}{3{,}599} = 1.49 < 1.5$$

This is probably close enough, depending upon the confidence in the design parameters and soil properties. If the design is to be modified to improve upon this safety factor the best change would be to increase the footing width. With the current design, simply increasing the key depth causes eccentricity to move outside the middle third of the footing and results in uplift at the heel of the footing.

13.4.3.3 ANALYSIS FOR ULTIMATE STRENGTH DESIGN OF FOOTING

Repeating the analysis to compute the overturning moments combined with ACI Load Factors (*LF*), the wall design and the sliding analysis, and then shear and moments can be determined for the footing.

TABLE 13.4 Factored Moments (Σ Moments at Toe of Footing)

Item	Factor		Height ft		Width ft		*LF* x Unit Pressure psf		Weight lbs/ft		Moment Arm ft		Resisting Moment ft lbs/ft
(1) Footing			1.333	x	6	x	1.2 x 150	=	1,440	x	3.00	=	4,320
(2) Key			1.0	x	1.0	x	1.2 x 150	=	180	x	5.50	=	990
(3) Wall			6.667	x	0.636	x	1.2 x 120	=	610	x	2.32	=	1,414
(4) Wall			3.333	x	0.969	x	1.2 x 120	=	465	x	2.48	=	1,155
(5) Earth			3.333	x	3.031	x	1.6 x 110	=	1,778	x	4.48	=	7,975
(6) Earth			6.667	x	3.365	x	1.6 x 110	=	3,948	x	4.32	=	17,045
(7) Earth	1/2	x	1.083	x	3.365	x	1.6 x 110	=	321	x	4.88	=	1,566
Total weight = 8,742													
(8) Vertical component of lateral earth pressure	1/2	x	13.412	x	13.412	x	1.6 x 12	=	1,727	x	6.00	=	10,362
Total vertical force = 10,469													
(9) Horizontal resistance above toe	1/2	x	1.333	x	1.333	x	1.6 x 367	=	522	x	0.444	=	232
(12) Horizontal component of lateral earth pressure on key below toe			12.412	x	1	x	1.6 x 40	=	794	x	0.5	=	397
	1/2	x	1	x	1	x	1.6 x 40	=	32	x	0.667	=	21
Total Resisting Moment ΣM												=	**45,477**

Lateral earth pressures:

$$F = F_8 = \frac{(LF)k_h h^2}{2} = \frac{1.6(40)(12.412)^2}{2} = 4{,}930 \text{ lbs/ft}$$

F_{12} = 794 + 32 = 826 lbs/ft (Table 13.4 Component 12)

F_L = 4,930 + 826 = 5,756 lbs/ft

Friction between footing and sub-base:

$$F_{\text{footing friction}} = 0.35(10{,}469) = 3{,}664 \text{ lbs/ft}$$

Passive resistance at the toe:

$$F_9 = LF(k_p)\left[\frac{h_{ftg}^2}{2} + h_{ftg}h_{key} + \frac{h_{key}^2}{2}\right]$$

$$= 1.6(367)\left[\frac{(1.333)^2}{2} + 1.333(1) + \frac{(1)^2}{2}\right]$$

= 1,598 lbs/ft

Friction force required at the base of the key, where $F_R = F_L$:

$F_{11} = F_R - F_9 - F_{\text{footing friction}}$ = 5,756 - 1,598 - 3,664 = 494 lbs/ft

Estimating friction on the bottom of the key:

$$F_{\text{key friction}} = LF\mu P_{avg} = 1.6(0.35)\left(\frac{465 + 56}{2}\right)$$

= 146 lbs/ft

Overturning moment

$$= LF\left(\frac{k_h h^3}{6} + \frac{k_p h_{ftg} h_{key}^2}{2} + \frac{k_p h_{key}^3}{3}\right)$$

$$+ (F_{11} + F_{\text{key friction}})h_{key}$$

$$= 1.6\left(\frac{40(12.412)^3}{6} + \frac{1.333(367)(1)^2}{2} + \frac{(367)(1)^3}{3}\right)$$

+ (494 + 146) (1) = 21,430 ft lbs/ft

$$\text{Eccentricity} = \frac{6}{2} - \left[\frac{45{,}477 - 21{,}430}{10{,}469}\right] = 0.7030 \text{ ft}$$

Soil pressure under footing

$$= \frac{10{,}469}{6} \pm \frac{6(10{,}469)(0.7030)}{6^2} = 1{,}744.8 \pm 1{,}226.6 \text{ psf}$$

= 2,971 psf maximum

= 518 psf minimum

Using these foundation reaction pressures, a corrected key friction force can be obtained:

$$F_{\text{key friction}} = 0.35(1)\left[\frac{518 + (1)(2{,}971 - 518)/6 + 518}{2}\right]$$

= 253 lbs/ft

Using the ultimate bearing pressures calculated above, with the applied loads, ultimate shear and moment diagrams can be developed for the footing. The shear and moment diagram in Figure 13.21 illustrates, in one graph, the effects of factored

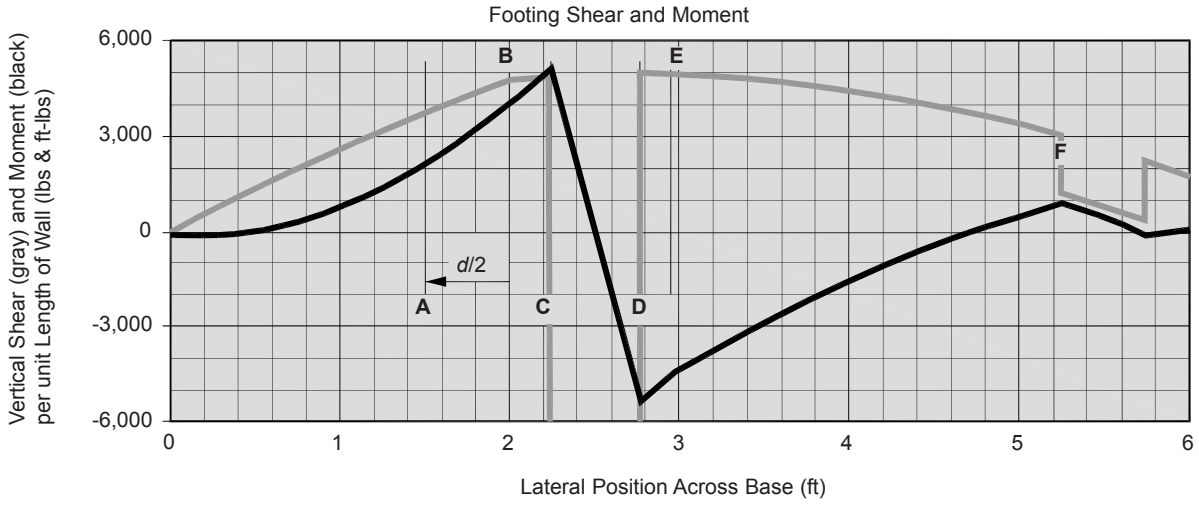


FIGURE 13.21 Footing shear and moment.

bearing pressures, wall and shear key moments (applied as force couples), and dead loads. Points B and E represent the outer and inner faces of the wall at its base. Points C and D represent points of maximum moment, which occur at the location of the wall force couple, traditionally taken to be at points 1/4 and 3/4 of the wall thickness. Point D is close enough to the vertical reinforcement for this concrete masonry wall, so that '*d*' for wall reinforcement will be used instead, that is, Point D is located at d = 9.25 in.

13.4.3.4 DESIGN OF FOOTING THICKNESS FOR SHEAR

The critical locations for shear are Point A for the shear in front of the wall and the point of maximum shear near Point D and Point E, in Figure 13.21. The location of Point A is at a distance $d/2$ out from the face of the wall (ACI 318-05, Section 11.12.1.2). Both of the ultimate shear loads at these locations are upward on the left, with shear resistance downward on the right from the weight of the soil behind the wall. The wall couple only produces local shear forces, which are transferred through a compression strut and tension in the reinforcement directly from the wall bearing and reinforcement to the footing concrete and reinforcement. The maximum net shear across the footing, in this case, is located at a point beneath the wall.

An estimate for d, given the estimated footing thickness of 16 in. is:

d_{est} = 16 in. - 3 in. (cover for concrete exposed to soil) - $d_{bar}/2 \approx$ 12.5 in.

$d/2 \approx$ 6.25 in.

Pressure at Point A:

$$= \frac{(2{,}971 - 518)(72 - 24 + 6.25)}{72} + 518 = 2{,}366 \text{ psf}$$

Pressure at Point D, a distance from the toe = 24 + 9.25 = 33.25 in. (2.771 ft) or a distance from the heel = 3.229 ft:

$$= \frac{(2{,}971 - 518)(72 - 24 - 9.25)}{72} + 518 = 1{,}838 \text{ psf}$$

Pressure at Point E:

$$= \frac{(2{,}971 - 518)(72 - 24 - 11.63)}{72} + 518 = 1{,}757 \text{ psf}$$

The ultimate strength shear load at Point A is:

$$V_{uA} = \left[\frac{(2{,}971 + 2{,}366)}{2} - 1.2(1.333)(150)\right]\frac{(24 - 6.25)}{12}$$

= 3,593 lbs/ft

The ultimate strength shear load at Point D is:

$$V_{uD} = \left[\frac{(2{,}971 + 1{,}838)}{2} - 1.2(1.333)(150)\right]\frac{(24 + 9.25)}{12}$$

$$- 1.2\left[10(120)\left(\frac{7.63}{12}\right) + 3.333(120)\left(\frac{9.25 - 7.63}{12}\right)\right]$$

$$- 1.6\left[6.667(110)\left(\frac{9.25 - 7.63}{12}\right) + \frac{110}{6}\left(\frac{9.25 - 7.63}{12}\right)^2\right]$$

= 4,861 lbs/ft

The ultimate strength shear load at Point E is:

$$V_{uE} = \left[\frac{(2{,}971 + 1{,}757)}{2} - 1.2(1.333)(150)\right]\frac{(24 + 11.63)}{12}$$

$$- 1.2\left[10(120)\left(\frac{7.63}{12}\right) + 3.333(120)\left(\frac{4}{12}\right)\right]$$

$$- 1.6\left[6.667(110)\left(\frac{4}{12}\right) + \frac{110}{6}\left(\frac{4}{12}\right)^2\right] = 4{,}838 \text{ lbs/ft}$$

Clearly, the shear is decreasing as the distance increases from the toe beyond Point D. Further inspection will indicate that Point D is approximately the location of the maximum ultimate shear load per foot of wall, which is 4,861 lbs.

The shear strength provided by the concrete, V_c, is:

$$V_c = 2\sqrt{f'_c}\, b_w d \qquad \text{(ACI 318 Eq 11-3)}$$

Evaluating this expression with b_w = 12 in., d = 12.5 in, f'_c = 2,500 psi and $\sqrt{f'_c} = \sqrt{2{,}500} = 50 < 100$ (from ACI 318, Section 11.1.2):

V_c = 2(50)(12)(12.5) = 15,000 lbs/ft

$\phi V_n = \phi V_c$ = 0.75(15,000) = 11,250 lbs/ft >> V_u = 4,861 lbs/ft

Thus shear is not a problem for the footing with 16 in. thickness and 12.5 in. depth of reinforcement. In this case the depth is upward from the bottom of the footing, since the shear force is acting upward from the left. Based on shear criteria, the thickness of the footing could be reduced to:

$$t = d + \frac{d_b est}{2} + \text{cover} = \frac{V_u}{2\phi\sqrt{f'_c}\, b_w} + \frac{d_b}{2} + \text{cover}$$

$$= \frac{4{,}861}{2(0.75)\sqrt{2{,}500}(12)} + 0.5 + 3 \approx 9 \text{ in.}$$

Stability, safety factors and development of the wall reinforcement into the footing also affect the footing thickness.

13.4.3.5 DESIGN OF FOOTING THICKNESS FOR DEVELOPMENT OF WALL REINFORCEMENT

A thinner footing would be possible if the reinforcement development length could be reduced. Check development of #6 bars being embedded in the footing from wall:

A_s required = 0.0043(8)(9.25) = 0.3182 < 0.44 sq in. A_s provided

The basic development length for a #6 vertical uncoated bar and normal weight concrete from ACI 318, Section 12.2.2 is:

$$l_d = \left(\frac{A_{s(\text{required})}}{A_{s(\text{provided})}}\right)\left(\frac{f_y \psi_t \psi_e \lambda}{25\sqrt{f'_c}}\right) d_b$$

$$= \left(\frac{0.3182}{0.44}\right)\left(\frac{60{,}000(1.0)(1.0)(1.0)}{25\sqrt{2{,}500}}\right)0.75 = 26 \text{ in.}$$

This development length is much greater than the thickness of the footing. A more precise value can be found taking into account bar size, the 8 in. spacing, transverse reinforcement, presuming at least two layers of the smallest bar size, and ACI 318, Section 12.2.3 to compute development length:

$$K_{tr} = \frac{A_{tr} f_{yt}}{1500sn} = \frac{2(0.11)(60{,}000)}{1500(8)(1)} = 1.1$$

(ACI 318 Eq 12-2)

$$\left(\frac{c_b + K_{tr}}{d_b}\right) = \left(\frac{\frac{8}{2} + 1.1}{0.75}\right) = 6.8 > 2.5$$

so use 2.5 in ACI 318 Equation 12-1

Using ACI 318 Equation 12-1 combined with the factors from ACI 318, Sections 12.2.4 and 12.2.5:

$$l_d = \left(\frac{A_{s(\text{required})}}{A_{s(\text{provided})}}\right)\left(\frac{3 f_y \psi_t \psi_e \lambda}{40\sqrt{f'_c}\left(\frac{c_b + K_{tr}}{d_b}\right)}\right) d_b$$

$$= \left(\frac{0.3182}{0.44}\right)\left[\frac{3(60{,}000)(1.0)(1.0)(0.8)(1.0)}{40\sqrt{2{,}500}(2.5)}\right]0.75$$

= 15.6 in.

Thus a hook is still required to develop the vertical reinforcement in a 16 in. thick footing. For a 90° hook with side and extension cover and cover of more than 2 in. beyond hook:

$$l_{dh} = 0.7\left(\frac{A_{s(\text{required})}}{A_{s(\text{provided})}}\right)\left(\frac{0.02\psi_e \lambda f_y}{\sqrt{f'_c}}\right) d_b$$

$$= 0.7\left(\frac{0.3182}{0.44}\right)\left(\frac{0.02(1.0)(1.0)(60{,}000)}{\sqrt{2{,}500}}\right)0.75$$

= 9.11 in. > 6 in.

Adding 3 in. of protective cover for concrete cast against soil, the total required thickness of the footing will be greater than 12 inches. Foundation footings are generally constructed in increments of 2 or 3 in., so a footing thickness of 14 in. is possible. The original 16 in. is also reasonable, however, reducing the footing thickness below 16 in. results in a lower factor of safety for sliding, so the 16 in. footing thickness will be used.

13.4.3.6 DESIGN OF FOOTING BOTTOM STEEL

The moment for the footing under the masonry wall is taken at the critical section, Point C, which can be taken as a point halfway between the middle and edge of the wall (ACI 318, Section 15.4.2b), a distance from the toe = 24 +11.63/4 = 26.9 in. (2.24 ft) or a distance from the heel = 3.76 ft.

Soil pressure at critical section, Point C:

$$= [2{,}971 - 518]\left(\frac{3.76}{6}\right) + 518 = 2{,}055 \text{ psf}$$

Moment at critical section:

$$M_{uC} = \frac{(2{,}971 - 2{,}055)(2.24)^2}{3} + \frac{(2{,}055)(2.24)^2}{2}$$

$$- \frac{1.6(367)(1.333)^3}{2(6)} - \frac{1.2(1.333)(150)(2.24)^2}{2}$$

$$- \frac{1.2(10)(120)(0.24)^2}{2} - 0.35\left(\frac{2{,}971 + 2{,}055}{2}\right)$$

$$(2.24)\left(\frac{1.333}{2}\right) = 4{,}619 \text{ ft lbs/ft}$$

$$\frac{M_u}{\phi f'_c\, bd^2} = \frac{4{,}619(12)}{0.9(2{,}500)(12)(12.5)^2} = 0.0131$$

$$\omega = \frac{1}{2(0.59)} - \frac{1}{2}\sqrt{\left(\frac{1}{0.59}\right)^2 - \left(\frac{4}{0.59}\right)\frac{M_u}{\phi f'_c\, bd^2}}$$

$$= 0.8475 - 0.5\sqrt{2.8727 - 6.7797(0.0131)}$$

$$= 0.0132$$

$$\rho = \omega\frac{f'_c}{f_y} = 0.0132\,\frac{2{,}500}{60{,}000} = 0.0006$$

$$A_s = \rho b_w d = 0.0006(12)(12.5) = 0.090 \text{ in.}^2/\text{ft}$$

Use #4 bars at 16 in. o. c. to match the vertical reinforcement in the wall (A_s provided = 0.15 in.^{2} > 0.090 in.^{2})

13.4.3.7 DESIGN OF FOOTING TOP STEEL

The top steel moment for the footing under a masonry wall is taken at the maximum negative moment, which is approximately the critical section halfway between the middle and the edge of the (ACI 318, Section 15.4.2b). Moment at the critical section, Point D:

$$M_{uD} = \frac{(2{,}971 - 1{,}838)(2.771)^2}{3} + \frac{(1{,}838)(2.771)^2}{2}$$

$$- \frac{1.6(367)(1.333)^3}{2(6)} - \frac{1.2(1.333)(150)(2.771)^2}{2}$$

$$- \frac{1.2(10)(120)(0.6354)^2}{2} - \frac{1.2(3.333)(120)(0.1354)^2}{2}$$

$$- \frac{1.6(6.667)(110)(0.1354)^2}{2} - \left[0.35\left(\frac{2{,}971 + 1{,}838}{2}\right)\right.$$

$$\left.(2.771)\left(\frac{1.333}{2}\right)\right] - 1.6(6{,}667) - 1.6(2{,}000)\left(\frac{1.333}{2}\right)$$

= -5,737 ft lbs/ft

$$\frac{M_u}{\phi f'_c\, bd^2} = \frac{5{,}737(12)}{0.9(2{,}500)(12)(12.5)^2} = 0.0163$$

$$\omega = 0.8475 - 0.5\sqrt{2.8727 - 6.7797(0.0163)}$$

= 0.0165

$$\rho = \omega\frac{f'_c}{f_y} = 0.0165\,\frac{2{,}500}{60{,}000} = 0.0007$$

$A_s = \rho b_w d$ = 0.0007(12)(12.5) = 0.1031 in.^{2}/ft

Minimum A_s for Grade 60 deformed reinforcement from ACI 318, Section 7.12.2.1

A_s = 0.0018(12)(16) = 0.3456 sq in./ft

Maximum spacing is 18 in. from ACI 318, Section 7.6.5

Minimum flexural reinforcement from ACI 318, Section 10.5

$$A_{s,\min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{2{,}500}}{60{,}000}(12)(12.5) = 0.375 \text{ sq in.}$$

(ACI 318 Eq 10-3)

$$\frac{200 b_w d}{f_y} = \frac{200(12)(12.5)}{60{,}000} = 0.50 > 0.375$$

so 0.5 sq in. governs

From ACI 318, Section 10.5.3, the minimum flexural reinforcement need not be applied if at every section A_s provided is at least one-third greater than that required by analysis. The A_s provided for the top steel:

$$\frac{A_{s(\text{provided})}}{A_{s(\text{analysis})}} = \frac{0.2(12/8)}{0.1031} = 2.910 > \frac{4}{3} = 1.333 \text{ OK}$$

The A_s provided for the bottom steel:

$$\frac{A_{s(\text{provided})}}{A_{s(\text{analysis})}} = \frac{0.2(12/16)}{0.090} = 1.667 > \frac{4}{3} = 1.333 \text{ OK}$$

Use #4 bars at 8 in. o. c. top bars to match vertical reinforcement:

A_s = 0.2(12/8) + 0.2(12/16) = 0.45 sq in./ft > 0.3456 sq in./ft

13.4.3.8 DESIGN OF FOOTING KEY

Shear and moment on a key are small because most of the lateral forces are resisted by friction on the bottom of the toe of the footing, where the

pressures are greatest. The shear key must be strong enough to force the development of passive pressures and resist the remaining unbalanced forces.

Soil pressure at front of key:

$$= [2{,}971 - 518]\left(\frac{1}{6}\right) + 518 = 927 \text{ psf}$$

Footing friction in front of the key:

$$= 0.35\left(\frac{2{,}971 + 927}{2}\right)(5) = 3{,}410 \text{ lbs/ft}$$

Passive resistance:

$$F_9 toe = LF(k_p)\left(\frac{h_{ftg}^2}{2}\right) = 1.6(367)\frac{(1.333)^2}{2} = 522 \text{ lbs/ft}$$

$$F_9 key = LF(k_p)\left(h_{ftg}h_{key} + \frac{h_{key}^2}{2}\right)$$

$$= 1.6(367)\left[1.333(1) + \frac{(1)^2}{2}\right] = 1.76 \text{ lbs/ft}$$

Friction force required at the base of the key, where $F_R = F_L$:

$F_{11} = F_R - F_9 - F_{\text{footing friction}}$ = 5,756 - 1,598 - 3,410 = 748 lbs/ft

$V_u$key = 1,076 + 748 = 1,824 lbs/ft

$$M_u key = (1)(748) + 1.6(367)\left[\frac{1.333(1)^2}{2} + \frac{(1)^3}{3}\right]$$

= 1,335 ft lbs/ft

Computing depth of reinforcement as d = 12 - 3 - 0.5 = 8.5 in. shear capacity is:

$$V_c = 2\sqrt{2{,}500}\,(12)(8.5) = 10{,}200 \text{ lbs/ft}$$

$\phi V_n = \phi V_c$ = 0.75(10,200) = 7,650 lbs/ft >> V_u = 1,831 lbs/ft

Thus no shear reinforcement is required.

$$\frac{M_u}{\phi f'_c bd^2} = \frac{1{,}335\,(12)}{0.9(2{,}500)(12)(8.5)^2} = 0.0082$$

$$\omega = 0.8475 - 0.5\sqrt{2.8727 - 6.7797(0.0082)}$$

= 0.0083

$$\rho = \omega\frac{f'_c}{f_y} = 0.0083\,\frac{2{,}500}{60{,}000} = 0.0003$$

$A_s = \rho b_w d$ = 0.0003(12)(8.5) = 0.0353 sq in./ft

Check minimum ACI steel requirements for temperature and shrinkage (ACI 318, Section 7.12.2.1):

A_s = 0.0018(12)(12) = 0.259 sq in./ft > 0.0353 sq in./ft

Use 2 #4 bars at 16 in. o. c. to match the vertical reinforcement spacing in the wall (A_s provided = 0.3 sq in./ft > 0.259 sq in./ft).

13.4.3.9 DESIGN OF LONGITUDINAL REINFORCEMENT

Check minimum ACI steel requirements for temperature and shrinkage (ACI 318, Section 7.12.2.1):

A_s = 0.0018(72)(16) = 2.074 sq in.

Use 10 #4 bars or #4 bars 16 in. o. c. top and bottom along footing (A_s provided = 2.0 sq in.).

Use 2 additional #4 bars in the shear key.

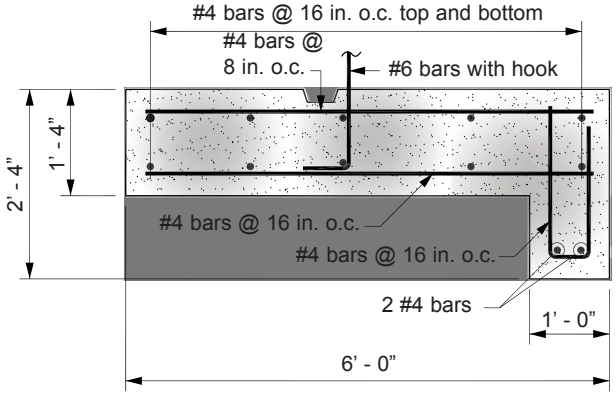


FIGURE 13.22 Footing reinforcement detail.

13.5 QUESTIONS AND PROBLEMS

13-1 Design a brick property line retaining wall and footing without a toe. Assume h = 8 ft, f'_m = 2500 psi, F_s = 24,000 psi, lateral earth pressure, E.F.P. = 30 pcf, f'_c = 2500 psi for footing, allowable soil bearing = 3000 psf, and weight of earth = 125 psf.

13-2 Design an 8 ft high supported concrete block wall for a subterranean garage to support an equivalent fluid pressure of 45 pounds per cubic foot. The wall is shown below and will be constructed with f'_m = 2000 psi, no special inspection, F_s = 24,000 psi, and an allowable soil bearing = 2500 psf.

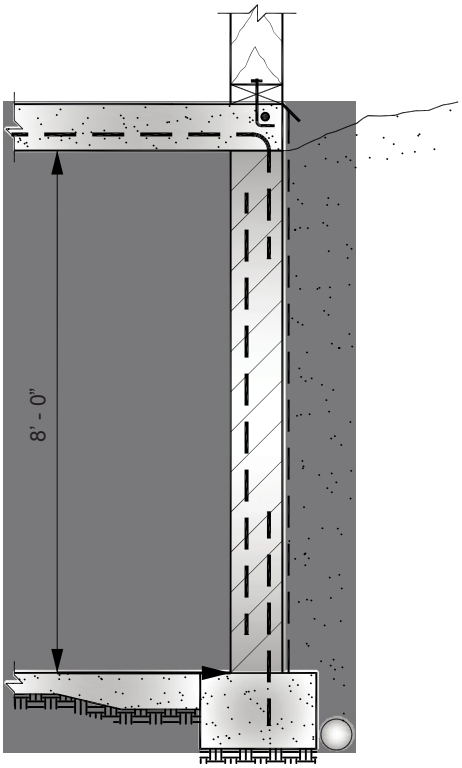


FIGURE 13.27 Problem 13-2 diagram.

13-3 Design a 6 ft high retaining wall using (a) concrete block masonry solid grouted, f'_m = 1500 psi and (b) solid grouted brick masonry, f'_m = 2000 psi . For both parts (a) and (b) assume f'_c = 2500 psi, F_s = 24,000 psi and allowable soil bearing = 2000 psf. Backfill is level, E.F.P. = 40 pcf with no surcharge. Footing extends under backfill.

13-4 Design a 16 ft high buttress retaining wall with the buttresses located 12 ft on centers. The backfill against the wall is on a slope of 2 to 1. Design the wall using grouted brick, f'_m = 2500 psi, F_s = 24,000 psi, and f'_c = 2500 psi. Weight of soil is 110 pounds per cubic foot and the allowable soil bearing is 4000 psf.

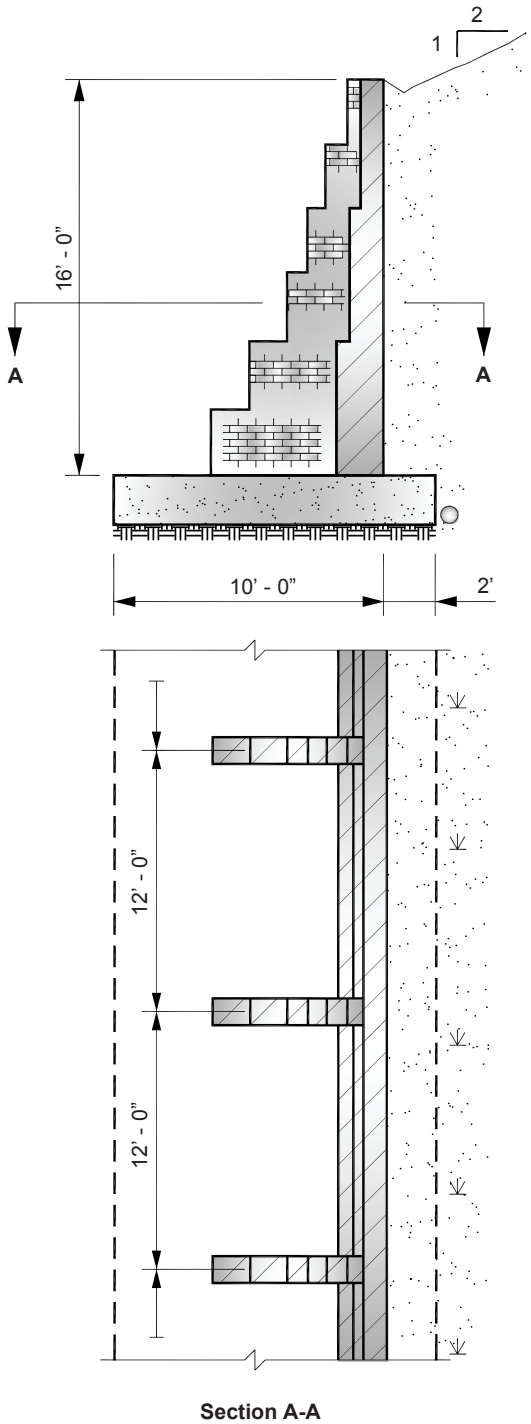


FIGURE 13.28 Problem 13-4 diagram.

CHAPTER

ASD

TABLES AND DIAGRAMS

ALLOWABLE STRESS DESIGN TABLES AND DIAGRAMS

Based on the
International Building Code
Requirements

Use judgment when using tables to the 4th decimal when the initial data is based on an estimate.

Don’t be so precise that you forget to be accurate.

Table ASD-1a Compressive Strength of Clay Masonry[1]

Net Area Compressive Strength of Clay Masonry Units (psi)		Net Area Compressive Strength of Masonry (psi)
Type M or S Mortar	Type N Mortar	
1,700	2,100	1,000
3,350	4,150	1,500
4,950	6,200	2,000
6,600	8,250	2,500
8,250	10,300	3,000
9,900	–	3,500
13,200	–	4,000

1. Based on MSJC Specification for Masonry Structures Table 1

Table ASD-1b Compressive Strength of Concrete Masonry[1]

Net Area Compresive Strength of Concrete Masonry Units (psi)		Net Area Compressive Strength of Masonry[2] (psi)
Type M or S Mortar	Type N Mortar	
1,250	1,300	1,000
1,900	2,150	1,500
2,800	3,050	2,000
3,750	4,050	2,500
4,800	5,250	3,000

1. Based on MSJC Specification for Masonry Structures Table 2
2. For units of less than 4 in. height, 85 percent of the values listed

Table ASD-2a Clay Masonry f'_m, E_m, n and E_v Values Based on the Clay Masonry Unit Strength and the Mortar Type

Type M or S Mortar				
Net Area Compressive Strength of Clay Masonry Units[1] (psi)	**Net Area Compressive Strength of Clay Masonry[2] (psi)**	**Modulus of Elasticity[3] $E_m = 700f'_m$ (psi)**	**Modular Ratio $n = E_s/E_m$ Where E_s = 29,000,000 psi[4]**	**Modulus of Rigidity[5] $E_v = 0.4E_m = 280f'_m$ (psi)**
1,700	1,000	700,000	41.4	280,000
3,350	1,500	1,050,000	27.6	420,000
4,950	2,000	1,400,000	20.7	560,000
6,600	2,500	1,750,000	16.6	700,000
8,250	3,000	2,100,000	13.8	840,000
9,900	3,500	2,450,000	11.8	980,000
13,200	4,000	2,800,000	10.4	1,120,000
Type N Mortar				
Net Area Compressive Strength of Clay Masonry Units[1] (psi)	**Net Area Compressive Strength of Clay Masonry[2] (psi)**	**Modulus of Elasticity[3] $E_m = 700f'_m$ (psi)**	**Modular Ratio $n = E_s/E_m$ Where E_s = 29,000,000 psi[4]**	**Modulus of Rigidity[5] $E_v = 0.4E_m = 280f'_m$ (psi)**
2,100	1,000	700,000	41.4	280,000
4,150	1,500	1,050,000	27.6	420,000
6,200	2,000	1,400,000	20.7	560,000
8,250	2,500	1,750,000	16.6	700,000
10,300	3,000	2,100,000	13.8	840,000

1. Compressive strength of solid masonry units is based on the gross area. Compressive strength of hollow clay masonry units is based on the minimum net area. Values may be interpolated.
2. Based on MSJC Specification Table 1.
3. Based on MSJC Code Section 1.8.2.2.1.
4. Based on MSJC Code Section 1.8.2.1.
5. Based on MSJC Code Section 1.8.2.2.2.

Table ASD-2b Concrete Masonry f'_m, E_m, n and E_v Values Based on the Concrete Masonry Unit Strength and the Mortar Type

Type M or S Mortar				
Net Area Compressive Strength of Concrete Masonry Units[1] (psi)	**Net Area Compressive Strength of Concrete Masonry[2] (psi)**	**Modulus of Elasticity[3] $E_m = 900f'_m$ (psi)**	**Modular Ratio $n = E_s/E_m$ Where E_s = 29,000,000 psi[4]**	**Modulus of Rigidity[5] $E_v = 0.4E_m = 280f'_m$ (psi)**
1,250	1,000	900,000	32.2	360,000
1,900	1,500	1,350,000	21.5	540,000
2,800	2,000	1,800,000	16.1	720,000
3,750	2,500	2,250,000	12.9	900,000
4,800	3,000	2,700,000	10.7	1,080,000
Type N Mortar				
Net Area Compressive Strength of Concrete Masonry Units[1] (psi)	**Net Area Compressive Strength of Concrete Masonry[2] (psi)**	**Modulus of Elasticity[3] $E_m = 900f'_m$ (psi)**	**Modular Ratio $n = E_s/E_m$ Where E_s = 29,000,000 psi[4]**	**Modulus of Rigidity[5] $E_v = 0.4E_m = 280f'_m$ (psi)**
1,300	1,000	900,000	32.2	360,000
2,150	1,500	1,350,000	21.5	540,000
3,050	2,000	1,800,000	16.1	720,000
4,050	2,500	2,250,000	12.9	900,000
5,250	3,000	2,700,000	10.7	1,080,000

1. Compressive strength of solid masonry units is based on the gross area. Compressive strength of hollow concrete masonry units is based on the minimum net area. Values may be interpolated.
2. Based on MSJC Specification Table 2.
3. Based on MSJC Code Section 1.8.2.2.1.
4. Based on MSJC Code Section 1.8.2.1.
5. Based on MSJC Code Section 1.8.2.2.2.

Table ASD-3 Maximum Allowable Working Stresses (psi), for Reinforced Solid and Hollow Unit Masonry[1,2]

Type of Stress		Specified Strength of Masonry, f'_m (psi)					
Specified Compressive Stress	f'_m	1500		2000		2500	
Masonry Type		Clay	CMU[3]	Clay	CMU[3]	Clay	CMU[3]
Masonry Unit Strength	Type M or S mortar	3350	1900	4950	2800	6600	3750
	Type N mortar	4150	2150	6200	3050	8250	4050
		Modulus Values					
Modulus of Elasticity	E_m (x10^6)	1.05	1.35	1.4	1.8	1.75	2.25
Modular Ratio	n (E_s/E_m)	27.6	21.5	20.7	16.1	16.6	12.9
Modulus of Rigidity	G (0.4*E_m) (x10^5)	4.2	5.4	5.6	7.2	7.0	9.0
	Allowable Stress of Stress Coefficient	Specified Values for Masonry (psi)					
Compression –							
Axial Column[12,13]	$F_a = 0.25\ f'_m$	375		500		625	
Axial Wall[10,12,13]	$F_a = 0.25\ f'_m$	375		500		625	
Flexural[9,13]	$F_b = \frac{1}{3} f'_m \le 2000$	500		667		833	
Shear for Unreinforced Masonry –							
Flexural[9,10,11,13] $f_v = VQ/I_nb$	$F_v = 1.5\sqrt{f'_m} \le 120$	58.09		67.08		75	
Shear Wall[11,13]	$F_v = 1.5\sqrt{f'_m} \le 120$	58.09		67.08		75	
Shear with No Shear Reinforcement for Reinforced Masonry –							
Flexural (No Flexural Tension)[4,6,13] –							
$f_v = VQ/I_nb$	$F_v = 3\sqrt{f'_m} \le 150$	116.2		134.2		150	
Flexural (Flexural Tension)[4,6,13,14] –							
$f_v = V/bd$	$F_v = \sqrt{f'_m} \le 50$	38.73		44.72		50	
Shear Wall –							
$M/Vd \ge 1$[4,7,13]	$F_v = \sqrt{f'_m} \le 35$	35		35		35	
$M/Vd < 1$[4,7,13] (values for $M/Vd = 0$)	$F_v = \frac{1}{3}(4 - M/Vd)\sqrt{f'_m} \le 80 - 45\ M/Vd$	51.64		59.63		66.67	
Shear Reinforcement Taking All Shear for Reinforced Masonry –							
Flexural[5,6,13,14] $f_v = V/bd$	$F_v = 3\sqrt{f'_m} \le 150$	116.2		134.2		150	
Shear Wall –							
$M/Vd \ge 1$[5,6,7,13]	$F_v = 1.5\sqrt{f'_m} \le 75$	58.09		67.08		75	
$M/Vd < 1$[5,6,7,13] (values for $M/Vd = 0$)	$F_v = \frac{1}{2}(4 - M/Vd)\sqrt{f'_m} \le 120 - 45\ M/Vd$	77.46		89.44		100	
Bearing –							
on full area[8,13]	$F_a = 0.25\ f'_m$	375		500		625	
on less than full area[8,13]	$F_a = \frac{1}{4}\sqrt{\frac{A_2}{A_1}} f'_m \le 0.5 f'_m$	≤ 750		≤ 1000		≤ 1250	

Table ASD-3 Maximum Allowable Working Stresses (psi), for Reinforced Solid and Hollow Unit Masonry[1,2] - continued

Type of Stress		Specified Strength of Masonry, f'_m (psi)					
Specified Compressive Stress	f'_m	3000		3500		4000	
Masonry Type		Clay	CMU[3]	Clay	CMU[3]	Clay	CMU[3]
Masonry Unit Strength	Type M or S mortar	8250	4800	9900	–	13200	–
	Type N mortar	10300	5250	–	–	–	–
		Modulus Values					
Modulus of Elasticity	E_m (x10^6)	2.1	2.7	2.45	3.15	2.8	3.6
Modular Ratio	n (E_s/E_m)	13.8	10.7	11.8	9.2	10.4	8.1
Modulus of Rigidity	G (0.4*E_m) (x10^5)	8.4	10.8	9.8	12.6	11.2	14.4
	Allowable Stress of Stress Coefficient	Specified Values for Masonry (psi)					
Compression –							
Axial Column[12,13]	$F_a = 0.25\ f'_m$	750		875		1000	
Axial Wall[10,12,13]	$F_a = 0.25\ f'_m$	750		875		1000	
Flexural[9,13]	$F_b = \frac{1}{3} f'_m \le 2000$	1000		1167		1333	
Shear for Unreinforced Masonry –							
Flexural[9,10,11,13] $f_v = VQ/I_nb$	$F_v = 1.5\sqrt{f'_m} \le 120$	82.16		88.74		94.87	
Shear Wall[11,13]	$F_v = 1.5\sqrt{f'_m} \le 120$	82.16		88.74		94.87	
Shear with No Shear Reinforcement for Reinforced Masonry –							
Flexural (No Flexural Tension)[4,6,13] –							
$f_v = VQ/I_nb$	$F_v = 3\sqrt{f'_m} \le 150$	150		150		150	
Flexural (Flexural Tension)[4,6,13,14] –							
$f_v = V/bd$	$F_v = \sqrt{f'_m} \le 50$	50		50		50	
Shear Wall –							
$M/Vd \ge 1$[4,7,13]	$F_v = \sqrt{f'_m} \le 35$	35		35		35	
$M/Vd < 1$[4,7,13] (values for $M/Vd = 0$)	$F_v = \frac{1}{3}(4 - M/Vd)\sqrt{f'_m} \le 80 - 45\ M/Vd$	73.03		78.88		80	
Shear Reinforcement Taking All Shear for Reinforced Masonry –							
Flexural[5,6,13,14] $f_v = V/bd$	$F_v = 3\sqrt{f'_m} \le 150$	150		150		150	
Shear Wall –							
$M/Vd \ge 1$[5,6,7,13]	$F_v = 1.5\sqrt{f'_m} \le 75$	75		75		75	
$M/Vd < 1$[5,6,7,13] (values for $M/Vd = 0$)	$F_v = \frac{1}{2}(4 - M/Vd)\sqrt{f'_m} \le 120 - 45\ M/Vd$	109.54		118.32		120	
Bearing –							
on full area[8,13]	$F_a = 0.25\ f'_m$	750		875		1000	
on less than full area[8,13]	$F_a = \frac{1}{4}\sqrt{\frac{A_2}{A_1}} f'_m \le 0.5 f'_m$	≤ 1500		≤ 1750		≤ 2000	

1. Equations and values are based on MSJC Code.
2. Stresses for hollow unit masonry are based on the net section.
3. CMU = Concrete Masonry Unit.
4. Shear reinforcement must be provided to carry the entire shear if shear stresses are in excess of the limits for Shear with No Shear Reinforcement for Reinforced Masonry. (MSJC Code Section 2.3.5.2.2)
5. When shear exceeds the limits for the case of Shear with No Shear Reinforcement for Reinforced Masonry, then all shear must be carried by the shear reinforcement, and the limits are provided by the case of Shear Reinforcement taking all Shear for Reinforced Masonry. (MSJC Code Section 2.3.5.2.3)
6. Where masonry is reinforced and shear reinforcement is required, then the minimum area of shear reinforcement provided shall be determined by $A_v = Vs/F_s d$. Shear reinforcement shall be provided parallel to the direction of applied shear force. Spacing of the shear reinforcement shall not exceed $d/2$ or 48 inches. Reinforcement with an area at least equal to one-third A_v shall also be provided perpendicular to the shear reinforcement and shall be uniformly distributed with a maximum spacing of 8 feet. (MSJC Code Section 2.3.5.3)
7. M is the bending moment occurring simultaneously with the shear load V at the section under consideration. M/Vd shall always be taken as a positive number. For allowable shear stresses when $0 < M/Vd < 1$, see Tables ASD-5 and ASD-6.
8. Allowable bearing stresses may be increased by up to a maximum factor of 2 based on the formula $\sqrt{A_2 / A_1}$, where A_1 is the bearing area and A_2 is the supporting surface wider than A_1 on all sides, or A_2 is the area of the lower base of the largest frustum of a right pyramid or cone having A_1 as the upper base sloping 45 degrees from the horizontal and wholly contained within the support. For walls other than running bond, area A_2 shall terminate at head joints. (MSJC Code Section 2.1.9.2b)
9. Allowable tensile stresses for masonry elements subject to flexure shall be in accordance with Table ASD-10.
10. The axial stress in unreinforced masonry may not exceed the value determined by the following formulas: $f_a/F_a + f_b/F_b \leq 1$ and $P \leq 0.25 P_e$, where $F_a = 0.25f'_m [1 - (h/140r)^2]$ when $h/r \leq 99$ and $F_a = 0.25 f'_m [1 - (70 r/h)^2]$ when $h/r > 99$, $F_b = f'_m / 3$, and $P_e = (\pi^2 E_m I_n/h^2) (1 - 0.577 e/r)^3$.
11. For running bond masonry the shear stresses shall not exceed (37 psi + 0.45 N_v/A_n) for masonry not grouted solid or (60 psi + 0.45 N_v/A_n) for masonry grouted solid. For stack bond masonry the shear stresses shall not exceed (37 psi + 0.45 N_v/A_n) for masonry with open end units and grouted solid or (15 psi) for masonry other than open end units grouted solid. (MSJC Code Section 2.2.5.2)
12. The compressive force in reinforced masonry due to axial load only shall not exceed: $P_a = (0.25 f'_m A_n + 0.65 A_{st} F_s) [1 - (h/140r)^2]$ when $h/r \leq 99$ and $P_a = (0.25f'_m A_n + 0.65 A_{st} F_s) (70 r/h)^2$ when $h/r > 99$.
13. MSJC Code Section 2.1.2.3 permits an increase in the allowable stress values when considering wind or seismic forces.
14. Members subjected to flexural tension shall be reinforced to resist the tension.

Table ASD-4 Allowable Steel Working Stresses, psi

Tensile Strength, F_s:
For deformed bars with a yield strength of 40,000 psi (Grade 40).....20,000[1]
For deformed bars with a yield strength of 60,000 psi or more and in sizes No. 11 and smaller.....24,000[1]
Joint reinforcement, 50 percent of the minimum yield point specified in ASTM A951 for the particular kind and grade of steel used, but in no case to exceed.....30,000[1]
Compressive Stress in Column Verticals, F'_s:
40 percent of the minimum yield strength, but not to exceed.....24,000[1]
Compressive Stress in Flexural Members:
For compression reinforcement in flexural members, the allowable stress shall not be taken as greater than the allowable tensile stress shown above.
The modulus of elasticity of steel reinforcement, E_s.....29,000,000[2]

1. Based on MSJC Code Section 2.3.2.2.2 and 2.3.2.1
2. Based on MSJC Code Section 1.8.2.1

Diagram ASD-5 Allowable Shear Wall Stresses with the Masonry Designed to Carry the Entire Shear Load

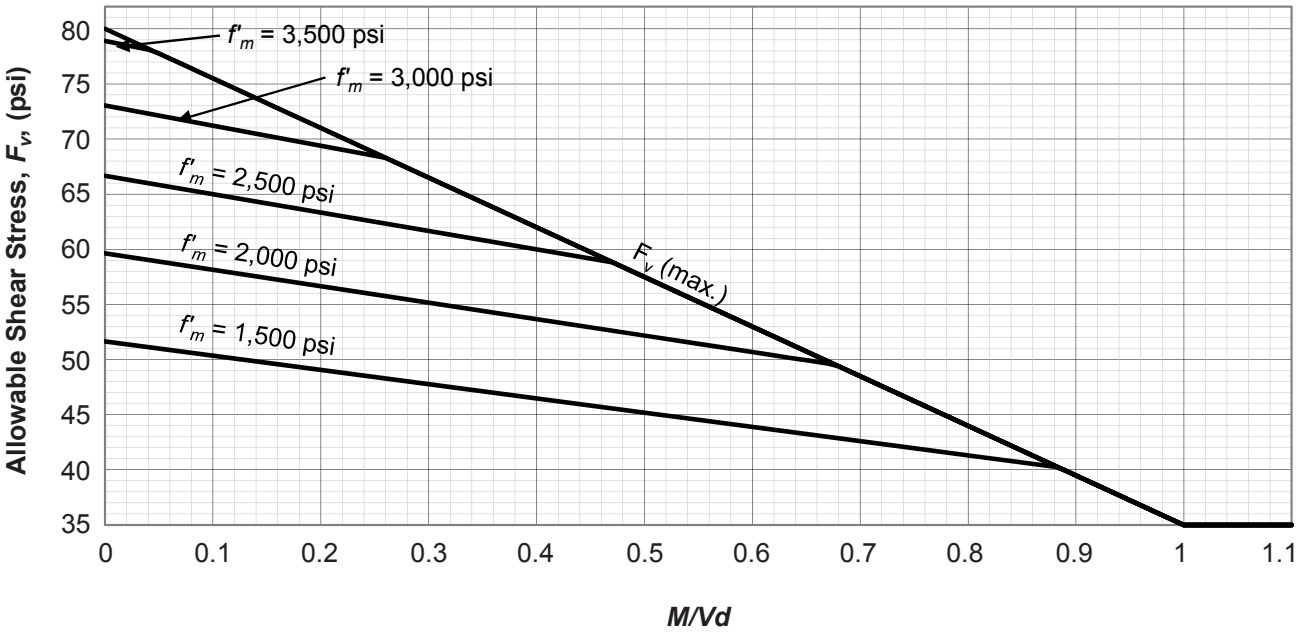


Table ASD-5 Allowable Shear Wall Stresses, psi, Where Masonry is Designed to Carry the Entire Shear Load[1]

f'_m (psi)	*M/Vd*										
	0.0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0+**
1500	52	50	49	48	46	45	44	43	41	40	35
2000	60	58	57	55	54	52	51	48	44	40	35
2500	67	65	63	62	60	58	53	48	44	40	35
3000	73	71	69	66	62	58	53	48	44	40	35
3500	79	76	71	66	62	58	53	48	44	40	35
4000+	80	76	71	66	62	58	53	48	44	40	35

1. Based on MSJC Code Section 2.3.5.2.2 Eqs. 2-21 and 2-22.

Diagram ASD-6 Allowable Shear Wall Stresses with the Steel Designed to Carry the Entire Shear Load

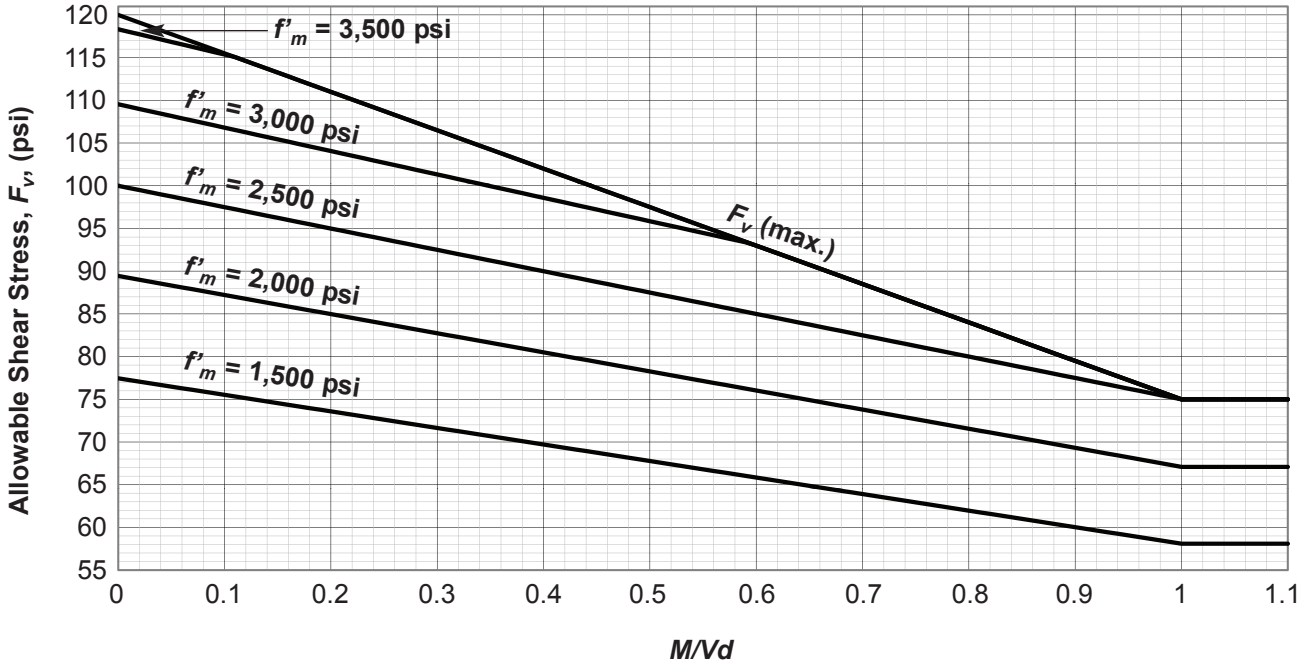


Table ASD-6 Allowable Shear Wall Stresses, psi, Where Reinforcement is Designed to Carry the Entire Shear Load[1]

f'_m	*M/Vd*										
(psi)	**0.0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0+**
1500	77	76	74	72	70	68	66	64	62	60	58
2000	89	87	85	83	80	78	76	74	72	69	67
2500	100	98	95	92	90	88	85	82	80	78	75
3000	110	107	104	101	99	96	93	88	84	80	75
3500	118	115	111	106	102	98	93	88	84	80	75
4000+	120	116	111	106	102	98	93	88	84	80	75

[1] Based on MSJC Code Section 2.3.5.2.3 Eqs. 2-24 and 2-25.

Table ASD-7a Allowable Tension B_a (pounds) for Embedded Anchor Bolts in Clay and Concrete Masonry Based on the Masonry Strength[1,2,3]

f'_m	Embedment Length l_b or Edge Distance l_{be} (inches)						
(psi)	**2**	**3**	**4**	**5**	**6**	**8**	**10**
1000	200	450	790	1240	1790	3180	4970
1500	240	550	970	1520	2190	3890	6080
2000	280	630	1120	1760	2530	4500	7020
2500	310	710	1260	1960	2830	5030	7850
3000	340	770	1380	2150	3100	5510	8600
3500	370	840	1490	2320	3350	5950	9290
4000	400	890	1590	2480	3580	6360	9930
4500	420	950	1690	2630	3790	6740	10540
5000	440	1000	1780	2780	4000	7110	11110
5500	470	1050	1860	2910	4190	7460	11650
6000	490	1100	1950	3040	4380	7790	12170

1. The allowable tension values in Table ASD-7a are based on the compressive strength of masonry assemblages. Where yield strength of anchor bolt steel governs, the allowable tension in pounds is given in Table ASD-7b.
2. Values based on MSJC Code Section 2.1.4.2.2 Eq. 2-1.
3. Values may be increased by one-third when considering load combinations including earthquake per MSJC Code Section 2.1.2.3.

Table ASD-7b Allowable Tension B_a (pounds) for Embedded Anchor Bolts in Clay and Concrete Masonry Based on A307 Anchor Bolts[1,2,3]

Anchor Bolt Diameter (inches)							
1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8
350	800	1410	2210	3180	4330	5650	7160

1. Values are for bolts conforming to ASTM A307. Bolts shall be those specified in MSJC Code Section 2.1.4.2.
2. Values based on MSJC Code Section 2.1.4.2.2 Eq. 2-2.
3. Values may be increased by one-third when considering load combinations including earthquake per MSJC Code Section 2.1.2.3.

Table ASD-7c Percent Tension Capacity of Anchor Bolts Based on Bolt Spacing[1,2,3]

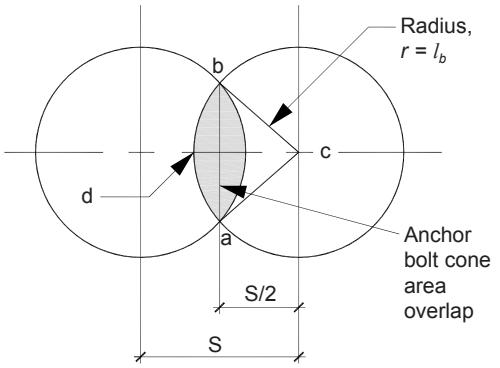
Per MSJC Code Section 2.1.4.2.2.1, the tension capacity of anchor bolts must be reduced if the areas of their tension (pullout) cones, A_p, overlap. The tensile capacity of such bolts must be determined by reducing, A_p, of the bolts by one half the overlapping area. The values in this table show the appropriate percent capacity reduction based on the spacing of the anchor bolts (see figure below).

Area of Segment, adb = Area of Sector, adbc - Area of Triangle, abc

$$= \pi l_b^2 \cos^{-1}\left(\frac{s}{2l_b}\right) - \left(\frac{s}{2}\right)\sqrt{l_b^2 - \left(\frac{s}{2}\right)^2}$$

Tension Cone Area, $A_p = \pi l_b^2$
Reduction % = Area of Segment, adb x $100/A_p$
Note to find the percent reduction, set $l_b = 1.0$

The anchor bolt must be embedded in a solid or grouted cell



Spacing of Bolts, *s*	0.0	$0.1l_b$	$0.2l_b$	$0.3l_b$	$0.4l_b$	$0.5l_b$	$0.6l_b$	$0.7l_b$	$0.8l_b$	$0.9l_b$	$1.0l_b$
% Capacity	50	53	56	60	63	66	69	72	75	78	80
% Reduction	50	47	44	40	37	34	31	28	25	22	20
Spacing of Bolts, *s*		$1.1l_b$	$1.2l_b$	$1.3l_b$	$1.4l_b$	$1.5l_b$	$1.6l_b$	$1.7l_b$	$1.8l_b$	$1.9l_b$	$2.0l_b$
% Capacity		83	86	88	91	93	95	97	98	99	100
% Reduction		17	14	12	9	7	5	3	2	1	0

1. l_b = Embedment depth of anchor bolts, inches.
2. Embedment length shall be measured perpendicular from the masonry surface to the bearing head of the anchor head for headed anchor bolts (to the bearing surface of the bent ends, minus one anchor bolt diameter, for bent bar anchor bolts).
3. The minimum effective embedment length required for placement of headed and bent bar anchor bolts shall be the greater of 2 in. or 4 bolt diameters.
4. The projected area, A_p, shall be reduced by half the overlapping area, between adjacent bolts, and all of any area outside the contiguous solid masonry assembly in which the anchor bolt is placed.

Table ASD-8a Allowable Shear B_v (pounds) for Embedded Anchor Bolts in Clay and Concrete Masonry Based on the Masonry Strength and ASTM A307 Anchor Bolts[1,2,3,4]

f'_m (psi)	Anchor Bolt Diameter (inches)							
	1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8
1000	210	480	850	1330	1600	1730	1850	1970
1500	210	480	850	1330	1780	1920	2050	2170
2000	210	480	850	1330	1910	2060	2200	2340
2500	210	480	850	1330	1910	2180	2330	2470
3000	210	480	850	1330	1910	2280	2440	2590
3500	210	480	850	1330	1910	2370	2530	2690
4000	210	480	850	1330	1910	2450	2620	2780
4500	210	480	850	1330	1910	2520	2700	2860
5000	210	480	850	1330	1910	2590	2770	2940
5500	210	480	850	1330	1910	2600	2840	3010
6000	210	480	850	1330	1910	2600	2900	3080

1. Values are for bolts conforming to ASTM A307. Bolts shall be those specified in MSJC Code Section 2.1.4.2.
2. The allowable shear values in Table ASD-8a are based on where the compressive strength of masonry assemblages or yield strength of anchor bolt steel governs. Refer to Table ASD-8b for the percent capacity of anchor bolts based on edge distance.
3. Values based on MSJC Code Section 2.1.4.2.3 and MSJC Code Eqs. 2-5 and 2-6. Shaded values are controlled by the capacity of the bolt as given by MSJC Code Eq. 2-6.
4. Values may be increased by one-third when considering load combinations including earthquake per MSJC Code Section 2.1.2.3.

Table ASD-8b Percentage of Shear Capacity of Anchor Bolts Based on Edge Distance l_{be}[1,2,3,4]

Edge Distance	Anchor Bolt Diameter (inches)															
	1/4		3/8		1/2		5/8		3/4		7/8		1		1 1/8	
	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%
$12d_b$	3.0	100	4.5	100	6.0	100	7.5	100	9.0	100	10.5	100	12.0	100	13.5	100
$11d_b$	2.8	87.5	4.1	89.3	5.5	90.0	6.9	90.4	8.3	90.6	9.6	90.8	11.0	90.9	12.4	91.0
$10d_b$	2.5	75.0	3.8	78.6	5.0	80.0	6.3	80.8	7.5	81.3	8.8	81.6	10.0	81.8	11.3	82.0
$9d_b$	2.3	62.5	3.4	67.9	4.5	70.0	5.6	71.2	6.8	71.9	7.9	72.4	9.0	72.7	10.1	73.0
$8d_b$	2.0	50.0	3.0	57.1	4.0	60.0	5.0	61.5	6.0	62.5	7.0	63.2	8.0	63.6	9.0	64.0
$7d_b$	1.8	37.5	2.6	46.4	3.5	50.0	4.4	51.9	5.3	53.1	6.1	53.9	7.0	54.5	7.9	55.0
$6d_b$	1.5	25.0	2.3	35.7	3.0	40.0	3.8	42.3	4.5	43.8	5.3	44.7	6.0	45.5	6.8	46.0
$5d_b$	1.3	12.5	1.9	25.0	2.5	30.0	3.1	32.7	3.8	34.4	4.4	35.5	5.0	36.4	5.6	37.0
$4d_b$	1.0	0	1.5	14.3	2.0	20.0	2.5	23.1	3.0	25.0	3.5	26.3	4.0	27.3	4.5	28.0
$3d_b$			1.1	3.6	1.5	10.0	1.9	13.5	2.3	15.6	2.6	17.1	3.0	18.2	3.4	19.0
$2d_b$					1.0	0	1.3	3.8	1.5	6.3	1.8	7.9	2.0	9.1	2.3	10.0
$1d_b$													1.0	0	1.1	1.0
1 in.	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0

1. MSJC Code Section 2.1.4.2.3 requires that the capacity of anchor bolts determined by MSJC Code Eq. 2-5 be reduced when the edge distance is less than $12d_b$.
2. d_b = Bar diameters
3. l_{be} = Edge distance in inches
4. % = Percentage capacity of Anchor Bolts

Table ASD-9a Allowable Axial Wall Compressive Stresses[1] $F_a = 0.25\ f'_m R$ (psi) and $R = [1 - (h/140r)^2]$

		Specified Strength of Masonry, f'_m, (psi)											
		1000	1500	2000	2500	3000	3500	4000	4500	5000	5500	6000	
h/r	*R*	Allowable Axial Wall Stress, F_a (psi)											*h/r*
5	**0.999**	250	375	499	624	749	874	999	1124	1248	1373	1498	**5**
10	**0.995**	249	373	497	622	746	871	995	1119	1244	1368	1492	**10**
15	**0.989**	247	371	494	618	741	865	989	1112	1236	1359	1483	**15**
20	**0.980**	245	367	490	612	735	857	980	1102	1224	1347	1469	**20**
25	**0.968**	242	363	484	605	726	847	968	1089	1210	1331	1452	**25**
30	**0.954**	239	358	477	596	716	835	954	1073	1193	1312	1431	**30**
31	**0.951**	238	357	475	594	713	832	951	1070	1189	1308	1426	**31**
32	**0.948**	237	355	474	592	711	829	948	1066	1185	1303	1422	**32**
33	**0.944**	236	354	472	590	708	826	944	1062	1181	1299	1417	**33**
34	**0.941**	235	353	471	588	706	823	941	1059	1176	1294	1412	**34**
35	**0.938**	234	352	469	586	703	820	938	1055	1172	1289	1406	**35**
36	**0.934**	233	350	467	584	700	817	934	1051	1167	1284	1401	**36**
37	**0.930**	233	349	465	581	698	814	930	1046	1163	1279	1395	**37**
38	**0.926**	232	347	463	579	695	811	926	1042	1158	1274	1389	**38**
39	**0.922**	231	346	461	576	692	807	922	1038	1153	1268	1384	**39**
40	**0.918**	230	344	459	574	689	804	918	1033	1148	1263	1378	**40**
41	**0.914**	229	343	457	571	686	800	914	1029	1143	1257	1371	**41**
42	**0.910**	228	341	455	569	683	796	910	1024	1138	1251	1365	**42**
43	**0.906**	226	340	453	566	679	792	906	1019	1132	1245	1358	**43**
44	**0.901**	225	338	451	563	676	789	901	1014	1127	1239	1352	**44**
45	**0.897**	224	336	448	560	673	785	897	1009	1121	1233	1345	**45**
46	**0.892**	223	335	446	558	669	781	892	1004	1115	1227	1338	**46**
47	**0.887**	222	333	444	555	665	776	887	998	1109	1220	1331	**47**
48	**0.882**	221	331	441	552	662	772	882	993	1103	1213	1324	**48**
49	**0.878**	219	329	439	548	658	768	878	987	1097	1207	1316	**49**
50	**0.872**	218	327	436	545	654	763	872	982	1091	1200	1309	**50**
51	**0.867**	217	325	434	542	650	759	867	976	1084	1193	1301	**51**
52	**0.862**	216	323	431	539	647	754	862	970	1078	1185	1293	**52**
53	**0.857**	214	321	428	535	643	750	857	964	1071	1178	1285	**53**
54	**0.851**	213	319	426	532	638	745	851	958	1064	1170	1277	**54**
55	**0.846**	211	317	423	529	634	740	846	951	1057	1163	1268	**55**
56	**0.840**	210	315	420	525	630	735	840	945	1050	1155	1260	**56**
57	**0.834**	209	313	417	521	626	730	834	939	1043	1147	1251	**57**
58	**0.828**	207	311	414	518	621	725	828	932	1035	1139	1243	**58**
59	**0.822**	206	308	411	514	617	720	822	925	1028	1131	1234	**59**
60	**0.816**	204	306	408	510	612	714	816	918	1020	1122	1224	**60**

1. Based on MSJC Code Section 2.2.3.1a Eq. 2-12.

Table ASD-9b Allowable Axial Wall Compressive Stresses[1] $F_a = 0.25\ f'_m R$ (psi) and $R = [1 - (h/140r)^2]$

		Specified Strength of Masonry, f'_m, (psi)											
		1000	1500	2000	2500	3000	3500	4000	4500	5000	5500	6000	
h/r	*R*	Allowable Axial Wall Stress, F_a (psi)											*h/r*
61	**0.810**	203	304	405	506	608	709	810	911	1013	1114	1215	**61**
62	**0.804**	201	301	402	502	603	703	804	904	1005	1105	1206	**62**
63	**0.798**	199	299	399	498	598	698	798	897	997	1097	1196	**63**
64	**0.791**	198	297	396	494	593	692	791	890	989	1088	1187	**64**
65	**0.784**	196	294	392	490	588	686	784	882	981	1079	1177	**65**
66	**0.778**	194	292	389	486	583	681	778	875	972	1069	1167	**66**
67	**0.771**	193	289	385	482	578	675	771	867	964	1060	1156	**67**
68	**0.764**	191	287	382	478	573	669	764	860	955	1051	1146	**68**
69	**0.757**	189	284	379	473	568	662	757	852	946	1041	1136	**69**
70	**0.750**	188	281	375	469	563	656	750	844	938	1031	1125	**70**
71	**0.743**	186	279	371	464	557	650	743	836	929	1021	1114	**71**
72	**0.736**	184	276	368	460	552	644	736	827	919	1011	1103	**72**
73	**0.728**	182	273	364	455	546	637	728	819	910	1001	1092	**73**
74	**0.721**	180	270	360	450	540	631	721	811	901	991	1081	**74**
75	**0.713**	178	267	357	446	535	624	713	802	891	980	1070	**75**
76	**0.705**	176	264	353	441	529	617	705	793	882	970	1058	**76**
77	**0.698**	174	262	349	436	523	610	698	785	872	959	1046	**77**
78	**0.690**	172	259	345	431	517	603	690	776	862	948	1034	**78**
79	**0.682**	170	256	341	426	511	596	682	767	852	937	1022	**79**
80	**0.673**	168	253	337	421	505	589	673	758	842	926	1010	**80**
81	**0.665**	166	249	333	416	499	582	665	748	832	915	998	**81**
82	**0.657**	164	246	328	411	493	575	657	739	821	903	985	**82**
83	**0.649**	162	243	324	405	486	567	649	730	811	892	973	**83**
84	**0.640**	160	240	320	400	480	560	640	720	800	880	960	**84**
85	**0.631**	158	237	316	395	474	552	631	710	789	868	947	**85**
86	**0.623**	156	233	311	389	467	545	623	700	778	856	934	**86**
87	**0.614**	153	230	307	384	460	537	614	691	767	844	921	**87**
88	**0.605**	151	227	302	378	454	529	605	681	756	832	907	**88**
89	**0.596**	149	223	298	372	447	521	596	670	745	819	894	**89**
90	**0.587**	147	220	293	367	440	513	587	660	733	807	880	**90**
91	**0.578**	144	217	289	361	433	505	578	650	722	794	866	**91**
92	**0.568**	142	213	284	355	426	497	568	639	710	781	852	**92**
93	**0.559**	140	210	279	349	419	489	559	629	698	768	838	**93**
94	**0.549**	137	206	275	343	412	481	549	618	686	755	824	**94**
95	**0.540**	135	202	270	337	405	472	540	607	674	742	809	**95**
96	**0.530**	132	199	265	331	397	464	530	596	662	728	795	**96**
97	**0.520**	130	195	260	325	390	455	520	585	650	715	780	**97**
98	**0.510**	128	191	255	319	383	446	510	574	638	701	765	**98**
99	**0.500**	125	187	250	312	375	437	500	562	625	687	750	**99**

1. Based on MSJC Code Section 2.2.3.1a Eq. 2-12.

Table ASD-9c Allowable Axial Wall Compressive Stresses[1] $F_a = 0.25\ f'_m R$ (psi) and $R = (70r/h)^2$]

		Specified Strength of Masonry, f'_m, (psi)											
		1000	1500	2000	2500	3000	3500	4000	4500	5000	5500	6000	
h/r	R	Allowable Axial Wall Stress, F_a (psi)											h/r
100	0.490	123	184	245	306	368	429	490	551	613	674	735	100
101	0.480	120	180	240	300	360	420	480	540	600	660	721	101
102	0.471	118	177	235	294	353	412	471	530	589	648	706	102
103	0.462	115	173	231	289	346	404	462	520	577	635	693	103
104	0.453	113	170	227	283	340	396	453	510	566	623	680	104
105	0.444	111	167	222	278	333	389	444	500	556	611	667	105
106	0.436	109	164	218	273	327	382	436	491	545	600	654	106
107	0.428	107	160	214	267	321	374	428	481	535	588	642	107
108	0.420	105	158	210	263	315	368	420	473	525	578	630	108
109	0.412	103	155	206	258	309	361	412	464	516	567	619	109
110	0.405	101	152	202	253	304	354	405	456	506	557	607	110
111	0.398	99	149	199	249	298	348	398	447	497	547	597	111
112	0.391	98	146	195	244	293	342	391	439	488	537	586	112
113	0.384	96	144	192	240	288	336	384	432	480	528	576	113
114	0.377	94	141	189	236	283	330	377	424	471	518	566	114
115	0.371	93	139	185	232	278	324	371	417	463	509	556	115
116	0.364	91	137	182	228	273	319	364	410	455	501	546	116
117	0.358	89	134	179	224	268	313	358	403	447	492	537	117
118	0.352	88	132	176	220	264	308	352	396	440	484	528	118
119	0.346	87	130	173	216	260	303	346	389	433	476	519	119
120	0.340	85	128	170	213	255	298	340	383	425	468	510	120
121	0.335	84	126	167	209	251	293	335	377	418	460	502	121
122	0.329	82	123	165	206	247	288	329	370	412	453	494	122
123	0.324	81	121	162	202	243	283	324	364	405	445	486	123
124	0.319	80	120	159	199	239	279	319	359	398	438	478	124
125	0.314	78	118	157	196	235	274	314	353	392	431	470	125
126	0.309	77	116	154	193	231	270	309	347	386	424	463	126
127	0.304	76	114	152	190	228	266	304	342	380	418	456	127
128	0.299	75	112	150	187	224	262	299	336	374	411	449	128
129	0.294	74	110	147	184	221	258	294	331	368	405	442	129
130	0.290	72	109	145	181	217	254	290	326	362	399	435	130
131	0.286	71	107	143	178	214	250	286	321	357	393	428	131
132	0.281	70	105	141	176	211	246	281	316	352	387	422	132
133	0.277	69	104	139	173	208	242	277	312	346	381	416	133
134	0.273	68	102	136	171	205	239	273	307	341	375	409	134
135	0.269	67	101	134	168	202	235	269	302	336	370	403	135
136	0.265	66	99	132	166	199	232	265	298	331	364	397	136
137	0.261	65	98	131	163	196	228	261	294	326	359	392	137
138	0.257	64	96	129	161	193	225	257	289	322	354	386	138
139	0.254	63	95	127	159	190	222	254	285	317	349	380	139
140	0.250	63	94	125	156	188	219	250	281	313	344	375	140

1. Based on MSJC Code Section 2.2.3.1b Eq. 2-13.

Table ASD-10 Allowable Flexural Tension of Clay and Concrete Masonry[2] (psi)

Direction of Flexural Tensile Stress and Masonry Type	Mortar Types			
	Portland Cement/Lime or Mortar Cement		Masonry Cement or Air Entrained Portland Cement/Lime	
	M or S	N	M or S	N
Normal to bed joints				
Solid units	40	30	24	15
Hollow units[1]				
Ungrouted	25	19	15	9
Fully grouted	65	63	61	58
Parallel to bed joints in running bond				
Solid units	80	60	48	30
Hollow units				
Ungrouted and partially grouted	50	38	30	19
Fully grouted	80	60	48	30
Parallel to bed joints in stack bond	0	0	0	0

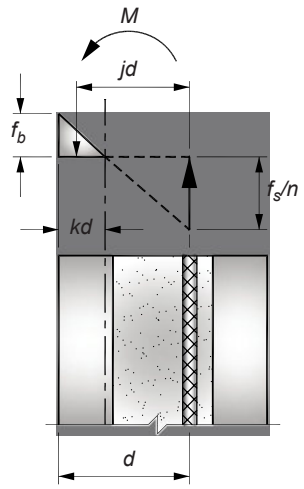
1. For partially grouted masonry, allowable stresses shall be determined on the basis of linear interpolation between fully grouted hollow units and ungrouted hollow units based on amount (percentage) of grouting.
2. MSJC Code Section Table 2.2.3.2.

Table ASD-22 Standard Bends and Hooks and Basic Development Length Provided

Bar Size[4]	Bar Diameter (in.)	f_y (ksi)	Minimum Diameters of Bend[2] (in.)	Minimum Extension Beyond Bend[3] (in.)		Development Length Provided[1] (in.)
				180-degree Hook	90-degree Bend	
#3 (#10)	0.375	40	1.875	2.5	4.5	4.22
#3 (#10)	0.375	50, 60	2.25	2.5	4.5	4.22
#4 (#13)	0.500	40	2.5	2.5	6	5.62
#4 (#13)	0.500	50, 60	3	2.5	6	5.62
#5 (#16)	0.625	40	3.125	2.5	7.5	7.03
#5 (#16)	0.625	50, 60	3.75	2.5	7.5	7.03
#6 (#19)	0.750	40	3.75	3	9	8.44
#6 (#19)	0.750	50, 60	4.5	3	9	8.44
#7 (#22)	0.875	40	4.375	3.5	10.5	9.84
#7 (#22)	0.875	50, 60	5.25	3.5	10.5	9.84
#8 (#25)	1.000	50, 60	6	4	12	11.25
#9 (#29)	1.128	50, 60	9.0	4.5	13.5	12.69
#10[5] (#32)	1.270	50, 60	10.2	5.1	15.2	14.29
#11[5] (#36)	1.410	50, 60	11.3	5.6	16.9	15.86

1. Development length provided by the hook or bend - based on MSJC Code Section 2.1.10.5.1
2. MSJC Code Section 1.13.6 Table 1.13.6 for Hooks and Bends other than for stirrups and ties
3. MSJC Code Section 1.13.5 for Hooks and Bends other than for stirrups and ties
4. Size in parentheses is Soft Metric Equivalent Size
5. Strength Design does not allow use of Bar Sizes greater than #9

Table ASD-24a Flexural Design Coefficients for Allowable Stresses (Clay Masonry) for f'_m = 1500 psi, f_y = 60,000 psi and n = 27.6



DESIGN DATA

f'_m = 1500 psi — f_y = 60,000 psi

$F_b = f'_m/3$ = 500 psi — F_s = 24,000 psi

$E_m = 700\ f'_m$ = 1,050,000 psi — E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 27.6 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	Increase for wind or earthquake $4F_b/3$ = 667 psi, $4F_s/3$ = 32,000 psi: f_b	f_s	K_f
50	24000	1.3	0.00006	0.002	0.054	0.982	37.44	67	32000	1.8
75	24000	2.9	0.00012	0.003	0.079	0.974	25.86	100	32000	3.9
100	24000	5.0	0.00022	0.006	0.103	0.966	20.07	133	32000	6.6
125	24000	7.5	0.00033	0.009	0.126	0.958	16.60	167	32000	10.0
150	24000	10.5	0.00046	0.013	0.147	0.951	14.29	200	32000	14.0
175	24000	13.8	0.00061	0.017	0.168	0.944	12.64	233	32000	18.5
200	24000	17.5	0.00078	0.022	0.187	0.938	11.40	267	32000	23.4
225	24000	21.6	0.00096	0.027	0.206	0.931	10.44	300	32000	28.7
250	24000	25.8	0.00116	0.032	0.223	0.926	9.67	333	32000	34.5
275	24000	30.4	0.00138	0.038	0.240	0.920	9.04	367	32000	40.5
300	24000	35.2	0.00160	0.044	0.257	0.914	8.52	400	32000	46.9
325	24000	40.2	0.00184	0.051	0.272	0.909	8.08	433	32000	53.6
350	24000	45.4	0.00209	0.058	0.287	0.904	7.70	467	32000	60.6
375	24000	50.8	0.00236	0.065	0.301	0.900	7.38	500	32000	67.8
400	24000	56.4	0.00263	0.073	0.315	0.895	7.09	533	32000	75.2
425	24000	62.2	0.00291	0.080	0.328	0.891	6.84	567	32000	82.9
450	24000	68.0	0.00320	0.088	0.341	0.886	6.61	600	32000	90.7
475	24000	74.1	0.00350	0.097	0.353	0.882	6.41	633	32000	98.7
500	24000	80.2	0.00380	0.105	0.365	0.878	6.23	667	32000	106.9
500	23000	82.1	0.00408	0.113	0.375	0.875	6.09	667	30667	109.4
500	22000	84.0	0.00438	0.121	0.386	0.871	5.95	667	29333	112.0
500	21000	86.1	0.00472	0.130	0.397	0.868	5.81	667	28000	114.8
500	20000	88.2	0.00511	0.141	0.408	0.864	5.67	667	26667	117.6
500	19000	90.5	0.00554	0.153	0.421	0.860	5.53	667	25333	120.6
500	18000	92.8	0.00603	0.167	0.434	0.855	5.39	667	24000	123.8
500	17000	95.3	0.00659	0.182	0.448	0.851	5.25	667	22667	127.1
500	16000	97.9	0.00724	0.200	0.463	0.846	5.11	667	21333	130.6
500	15000	100.7	0.00799	0.221	0.479	0.840	4.97	667	20000	134.3
500	14000	103.6	0.00887	0.245	0.497	0.834	4.83	667	18667	138.1
500	13000	106.7	0.00991	0.274	0.515	0.828	4.69	667	17333	142.2
500	12000	109.9	0.01115	0.308	0.535	0.822	4.55	667	16000	146.5
500	11000	113.3	0.01265	0.349	0.557	0.814	4.41	667	14667	151.1
500	10000	117.0	0.01450	0.400	0.580	0.807	4.27	667	13333	156.0
500	9000	120.8	0.01682	0.464	0.605	0.798	4.14	667	12000	161.1
500	8000	124.9	0.01979	0.547	0.633	0.789	4.00	667	10667	166.5
500	7000	129.2	0.02370	0.655	0.664	0.779	3.87	667	9333	172.3

Diagram ASD-24a K_f Versus ρ for Various Masonry and Steel Stresses, Clay Masonry, f'_m = 1500 psi, n = 27.6

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

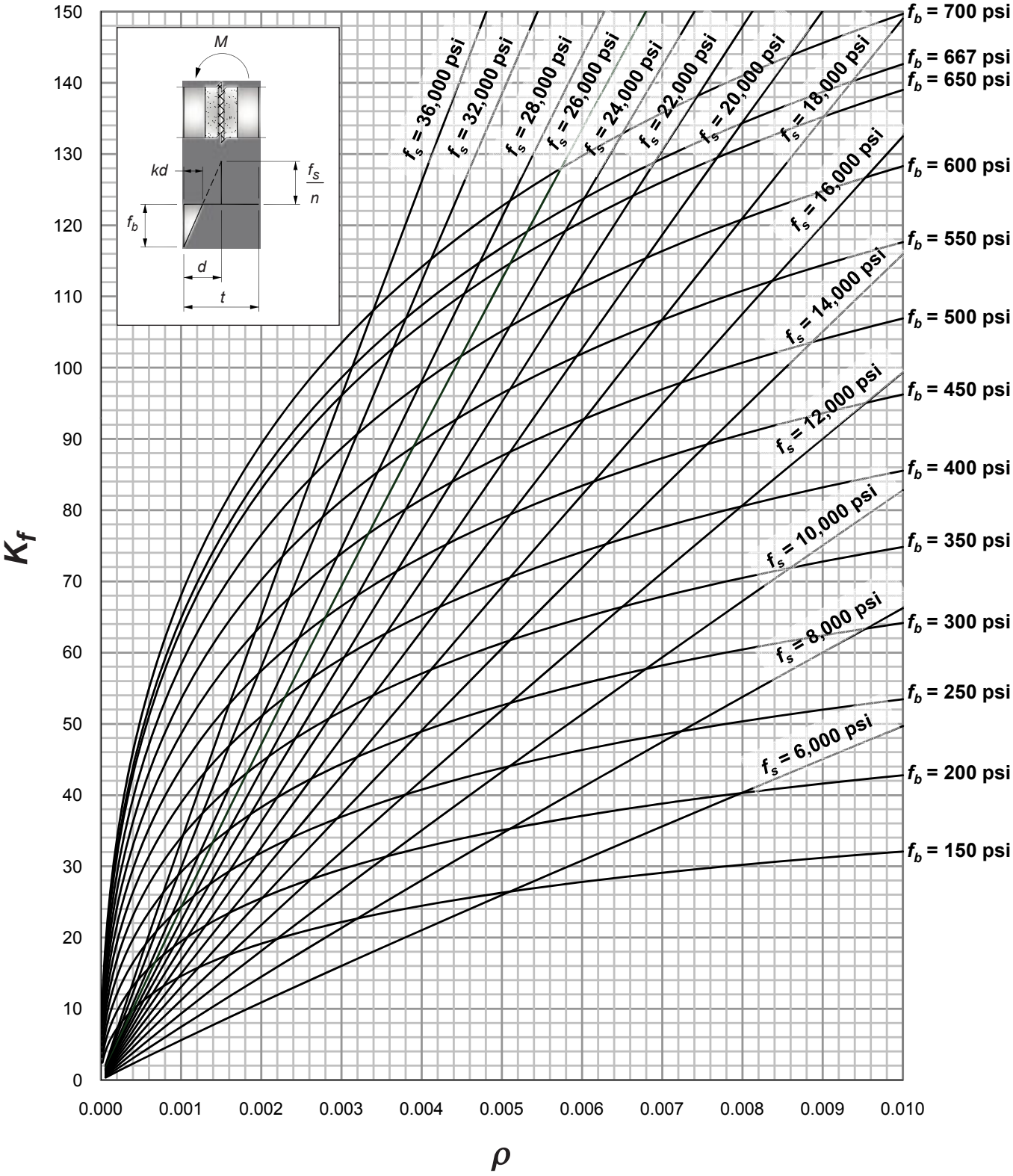
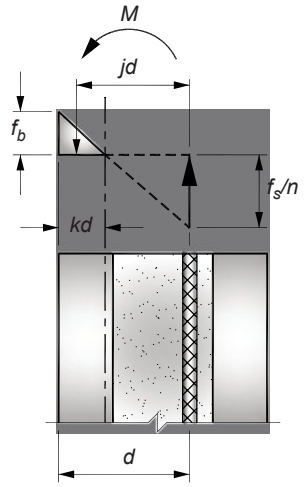


Table ASD-24b Flexural Design Coefficients for Allowable Stresses (Concrete Masonry) for f'_m = 1500 psi, f_y = 60,000 psi and n = 21.5



DESIGN DATA

f'_m = 1500 psi — f_y = 60,000 psi

$F_b = f'_m/3$ = 500 psi — F_s = 24,000 psi

$E_m = 900\ f'_m$ = 1,350,000 psi — E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 21.5 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	Increase for wind or earthquake $4F_b/3$ = 667 psi, $4F_s/3$ = 32,000 psi: f_b	f_s	K_f
50	24000	1.1	0.00004	0.001	0.043	0.986	47.37	67	32000	1.4
75	24000	2.3	0.00010	0.002	0.063	0.979	32.47	100	32000	3.1
100	24000	4.0	0.00017	0.004	0.082	0.973	25.03	133	32000	5.3
125	24000	6.1	0.00026	0.006	0.101	0.966	20.57	167	32000	8.1
150	24000	8.5	0.00037	0.008	0.118	0.961	17.59	200	32000	11.4
175	24000	11.3	0.00049	0.011	0.135	0.955	15.47	233	32000	15.1
200	24000	14.4	0.00063	0.014	0.152	0.945	13.87	267	32000	19.2
225	24000	17.8	0.00079	0.017	0.168	0.944	12.64	300	32000	23.7
250	24000	21.5	0.00095	0.020	0.183	0.939	11.65	333	32000	28.6
275	24000	25.4	0.00113	0.024	0.198	0.934	10.84	367	32000	33.8
300	24000	29.5	0.00132	0.028	0.212	0.929	10.17	400	32000	39.3
325	24000	33.9	0.00153	0.033	0.225	0.925	9.60	433	32000	45.2
350	24000	38.4	0.00174	0.037	0.239	0.920	9.11	467	32000	51.2
375	24000	43.2	0.00196	0.042	0.251	0.916	8.69	500	32000	57.6
400	24000	48.1	0.00220	0.047	0.264	0.912	8.32	533	32000	64.1
425	24000	53.2	0.00244	0.052	0.276	0.908	7.99	567	32000	70.9
450	24000	58.4	0.00269	0.058	0.287	0.904	7.70	600	32000	77.9
475	24000	63.8	0.00295	0.063	0.298	0.901	7.44	633	32000	85.1
500	24000	69.3	0.00322	0.069	0.309	0.897	7.21	667	32000	92.4
500	23000	71.1	0.00346	0.074	0.318	0.894	7.03	667	30667	94.9
500	22000	73.0	0.00373	0.080	0.328	0.891	6.85	667	29333	97.4
500	21000	75.1	0.00403	0.087	0.338	0.887	6.66	667	28000	100.1
500	20000	77.2	0.00437	0.094	0.349	0.884	6.48	667	26667	102.9
500	19000	79.4	0.00475	0.102	0.361	0.880	6.30	667	25333	105.9
500	18000	81.8	0.00519	0.111	0.374	0.875	6.11	667	24000	109.1
500	17000	84.3	0.00569	0.122	0.387	0.871	5.93	667	22667	112.4
500	16000	87.0	0.00628	0.135	0.402	0.866	5.75	667	21333	116.0
500	15000	89.8	0.00695	0.149	0.417	0.861	5.57	667	20000	119.7
500	14000	92.8	0.00775	0.167	0.434	0.855	5.39	667	18667	123.8
500	13000	96.0	0.00870	0.187	0.452	0.849	5.21	667	17333	128.1
500	12000	99.5	0.00984	0.211	0.472	0.843	5.03	667	16000	132.7
500	11000	103.2	0.01123	0.241	0.494	0.835	4.85	667	14667	137.6
500	10000	107.1	0.01295	0.278	0.518	0.827	4.67	667	13333	142.8
500	9000	111.4	0.01511	0.325	0.544	0.819	4.49	667	12000	148.5
500	8000	115.9	0.01791	0.385	0.573	0.809	4.31	667	10667	154.5
500	7000	120.8	0.02162	0.464	0.605	0.798	4.14	667	9333	161.1

Diagram ASD-24b K_f Versus ρ for Various Masonry and Steel Stresses, Concrete Masonry, f'_m = 1500 psi, n = 21.5

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

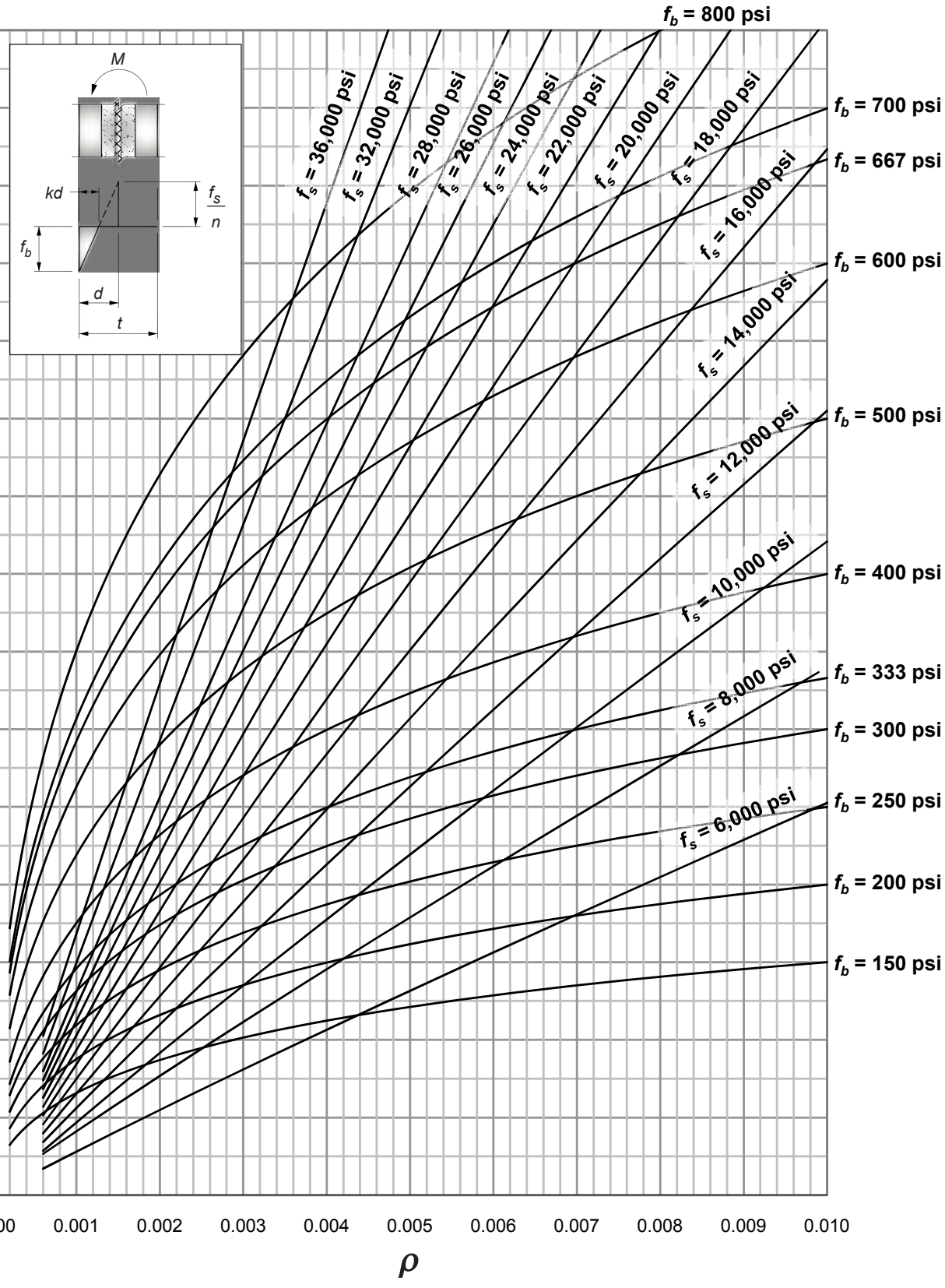
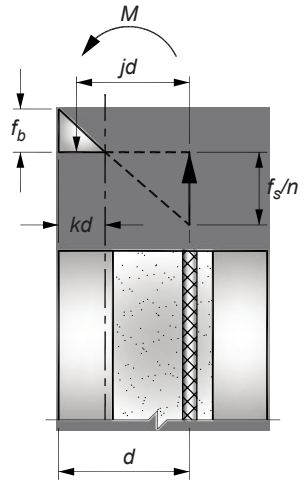


Table ASD-25a Flexural Design Coefficients for Allowable Stresses (Clay Masonry) for f'_m = 2000 psi, f_y = 60,000 psi and n = 20.7



DESIGN DATA

f'_m = 2000 psi
$F_b = f'_m/3$ = 667 psi
$E_m = 700\ f'_m$ = 1,400,000 psi
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 20.7 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 889 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	f_b	f_s	K_f
50	24000	1.0	0.00004	0.001	0.041	0.986	49.02	67	32000	1.4
100	24000	3.9	0.00017	0.003	0.079	0.974	25.86	133	32000	5.2
150	24000	8.3	0.00036	0.007	0.115	0.962	18.14	200	32000	11.0
200	24000	14.0	0.00061	0.013	0.147	0.951	14.29	267	32000	18.7
250	24000	20.9	0.00092	0.019	0.177	0.941	11.98	333	32000	27.8
300	24000	28.7	0.00129	0.027	0.206	0.931	10.44	400	32000	38.3
350	24000	37.5	0.00169	0.035	0.232	0.923	9.34	467	32000	49.9
400	24000	46.9	0.00214	0.044	0.257	0.914	8.52	533	32000	62.6
450	24000	57.1	0.00262	0.054	0.280	0.907	7.88	600	32000	76.1
500	24000	67.8	0.00314	0.065	0.301	0.900	7.38	667	32000	90.4
550	24000	79.0	0.00369	0.076	0.322	0.893	6.96	733	32000	105.4
600	24000	90.7	0.00426	0.088	0.341	0.886	6.61	800	32000	121.0
650	24000	102.8	0.00487	0.101	0.359	0.880	6.32	867	32000	137.1
667	24000	106.9	0.00507	0.105	0.365	0.878	6.23	889	32000	142.6
667	23000	109.4	0.00544	0.113	0.375	0.875	6.09	889	30667	145.9
667	22000	112.0	0.00584	0.121	0.386	0.871	5.95	889	29333	149.4
667	21000	114.8	0.00630	0.130	0.397	0.868	5.81	889	28000	153.0
667	20000	117.6	0.00681	0.141	0.408	0.864	5.67	889	26667	156.8
667	19000	120.6	0.00738	0.153	0.421	0.860	5.53	889	25333	160.8
667	18000	123.8	0.00804	0.167	0.434	0.855	5.39	889	24000	165.0
667	17000	127.1	0.00879	0.182	0.448	0.851	5.25	889	22667	169.4
667	16000	130.6	0.00965	0.200	0.463	0.846	5.11	889	21333	174.1
667	15000	134.3	0.01065	0.221	0.479	0.840	4.97	889	20000	179.0
667	14000	138.1	0.01182	0.245	0.497	0.834	4.83	889	18667	184.2
667	13000	142.2	0.01321	0.274	0.515	0.828	4.69	889	17333	189.6
667	12000	146.5	0.01486	0.308	0.535	0.822	4.55	889	16000	195.4
667	11000	151.1	0.01687	0.349	0.557	0.814	4.41	889	14667	201.5
667	10000	156.0	0.01933	0.400	0.580	0.807	4.27	889	13333	207.9
667	9000	161.1	0.02242	0.464	0.605	0.798	4.14	889	12000	214.8
667	8000	166.5	0.02638	0.547	0.633	0.789	4.00	889	10667	222.0
667	7000	172.3	0.03160	0.655	0.664	0.779	3.87	889	9333	229.7
667	6000	178.4	0.03873	0.802	0.697	0.768	3.74	889	8000	237.8
667	5000	184.8	0.04895	1.014	0.734	0.755	3.61	889	6667	246.4
667	4000	191.7	0.06462	1.338	0.775	0.742	3.48	889	5333	255.5
667	3000	198.9	0.09128	1.891	0.822	0.726	3.35	889	4000	265.1

Diagram ASD-25a K_f Versus ρ for Various Masonry and Steel Stresses, Clay Masonry, f'_m = 2000 psi, n = 20.7

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

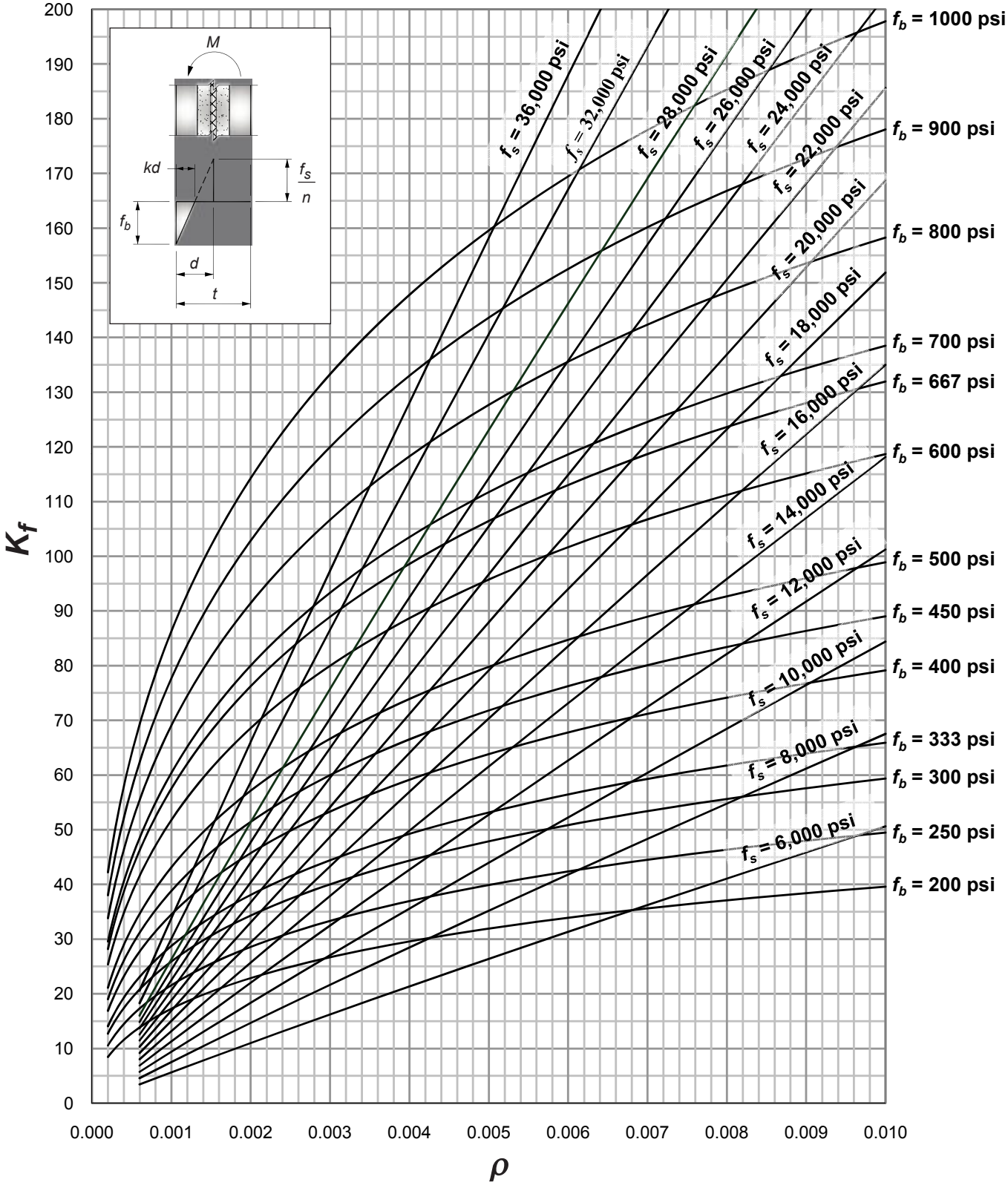
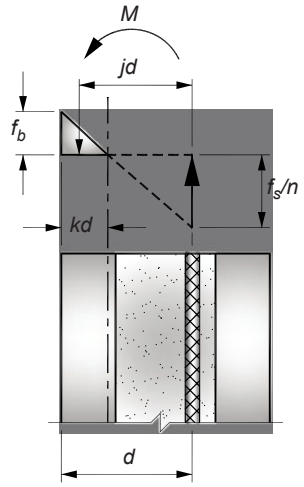


Table ASD-25b Flexural Design Coefficients for Allowable Stresses (Concrete Masonry) for f'_m = 2000 psi, f_y = 60,000 psi and n = 16.1



DESIGN DATA

f'_m = 2000 psi
$F_b = f'_m/3$ = 667 psi
$E_m = 900\ f'_m$ = 1,800,000 psi
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 16.1 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 889 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	f_b	f_s	K_f
50	24000	0.8	0.00003	0.001	0.032	0.989	62.26	67	32000	1.1
100	24000	3.1	0.00013	0.002	0.063	0.979	32.47	133	32000	4.1
150	24000	6.7	0.00029	0.005	0.091	0.970	22.55	200	32000	8.9
200	24000	11.4	0.00049	0.008	0.118	0.961	17.59	267	32000	15.2
250	24000	17.1	0.00075	0.012	0.144	0.952	14.62	333	32000	22.8
300	24000	23.7	0.00105	0.017	0.168	0.944	12.64	400	32000	31.7
350	24000	31.2	0.00139	0.022	0.190	0.937	11.22	467	32000	41.6
400	24000	39.3	0.00176	0.028	0.212	0.929	10.17	533	32000	52.5
450	24000	48.2	0.00218	0.035	0.232	0.923	9.34	600	32000	64.2
500	24000	57.6	0.00262	0.042	0.251	0.916	8.69	667	32000	76.7
550	24000	67.5	0.00309	0.050	0.270	0.910	8.15	733	32000	90.0
600	24000	77.9	0.00359	0.058	0.287	0.904	7.70	800	32000	103.9
650	24000	88.7	0.00411	0.066	0.304	0.899	7.33	867	32000	118.3
667	24000	92.4	0.00429	0.069	0.309	0.897	7.21	889	32000	123.2
667	23000	94.9	0.00461	0.074	0.318	0.894	7.03	889	30667	126.5
667	22000	97.4	0.00497	0.080	0.328	0.891	6.85	889	29333	129.9
667	21000	100.1	0.00537	0.087	0.338	0.887	6.66	889	28000	133.4
667	20000	102.9	0.00582	0.094	0.349	0.884	6.48	889	26667	137.2
667	19000	105.9	0.00634	0.102	0.361	0.880	6.30	889	25333	141.2
667	18000	109.1	0.00692	0.111	0.374	0.875	6.11	889	24000	145.4
667	17000	112.4	0.00759	0.122	0.387	0.871	5.93	889	22667	149.9
667	16000	116.0	0.00837	0.135	0.402	0.866	5.75	889	21333	154.6
667	15000	119.7	0.00927	0.149	0.417	0.861	5.57	889	20000	159.7
667	14000	123.8	0.01034	0.167	0.434	0.855	5.39	889	18667	165.0
667	13000	128.1	0.01160	0.187	0.452	0.849	5.21	889	17333	170.8
667	12000	132.7	0.01312	0.211	0.472	0.843	5.03	889	16000	176.9
667	11000	137.6	0.01497	0.241	0.494	0.835	4.85	889	14667	183.4
667	10000	142.8	0.01726	0.278	0.518	0.827	4.67	889	13333	190.4
667	9000	148.5	0.02015	0.325	0.544	0.819	4.49	889	12000	198.0
667	8000	154.5	0.02388	0.385	0.573	0.809	4.31	889	10667	206.1
667	7000	161.1	0.02883	0.464	0.605	0.798	4.14	889	9333	214.8
667	6000	168.1	0.03564	0.574	0.642	0.786	3.97	889	8000	224.2
667	5000	175.7	0.04549	0.733	0.682	0.773	3.79	889	6667	234.3
667	4000	183.9	0.06072	0.978	0.729	0.757	3.63	889	5333	245.2
667	3000	192.7	0.08685	1.399	0.782	0.739	3.46	889	4000	256.9

Diagram ASD-25b K_f Versus ρ for Various Masonry and Steel Stresses, Concrete Masonry, f'_m = 2000 psi, n = 16.1

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

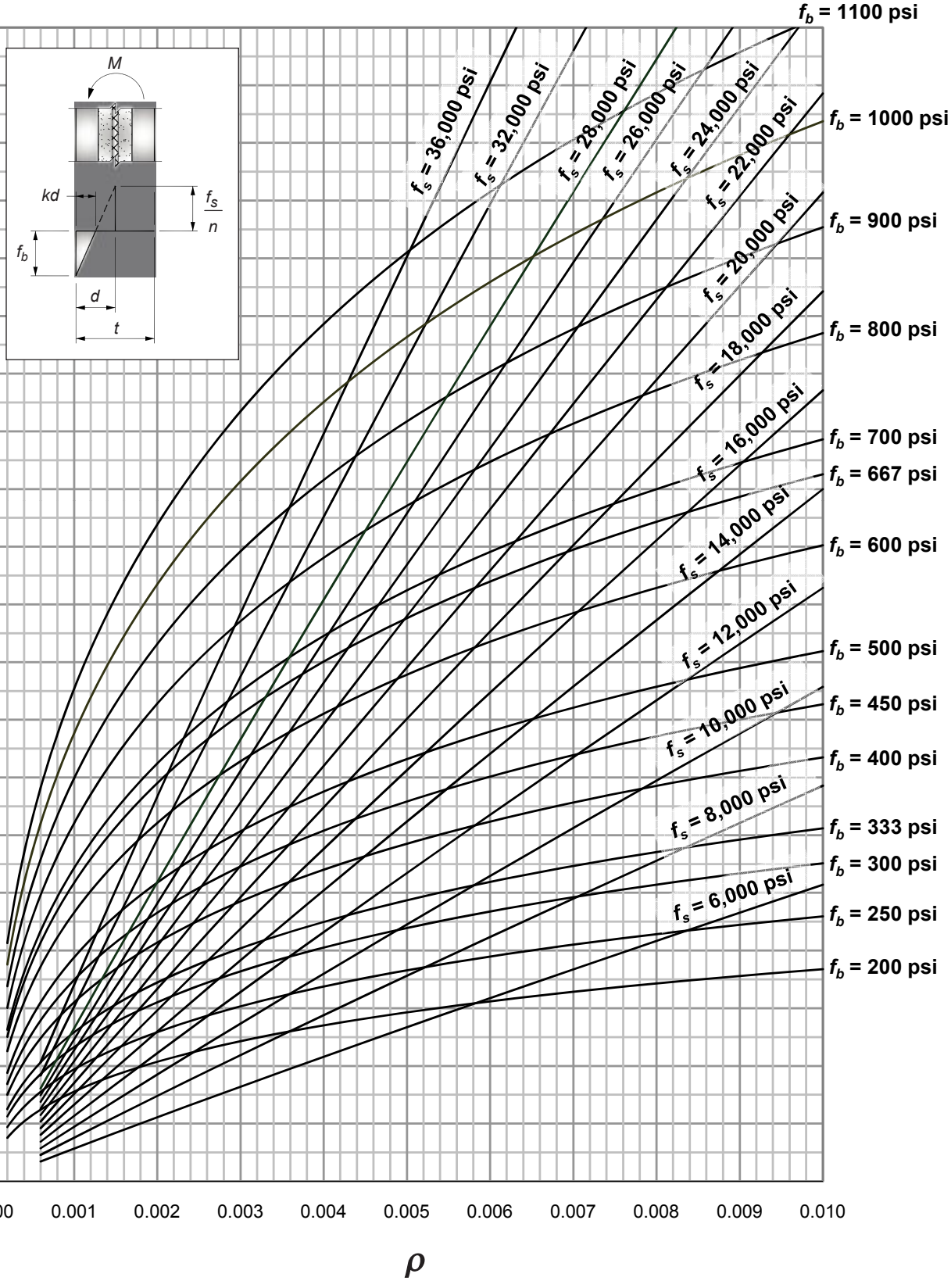
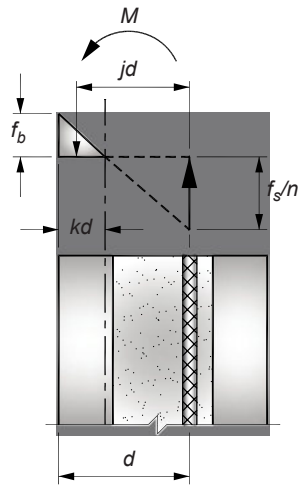


Table ASD-26a Flexural Design Coefficients for Allowable Stresses (Clay Masonry) for f'_m = 2500 psi, f_y = 60,000 psi and n = 16.6



DESIGN DATA

f'_m = 2500 psi — f_y = 60,000 psi

$F_b = f'_m/3$ = 833 psi — F_s = 24,000 psi

$E_m = 900\ f'_m$ = 1,750,000 psi — E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 16.6 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 1111 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	f_b	f_s	K_f
50	24000	0.8	0.00003	0.001	0.033	0.989	60.61	67	32000	1.1
100	24000	3.2	0.00013	0.002	0.065	0.978	31.65	133	32000	4.2
150	24000	6.8	0.00029	0.005	0.094	0.969	22.00	200	32000	9.1
200	24000	11.6	0.00051	0.008	0.121	0.960	17.18	267	32000	15.5
250	24000	17.5	0.00077	0.013	0.147	0.951	14.29	333	32000	23.3
300	24000	24.3	0.00107	0.018	0.172	0.943	12.36	400	32000	32.4
350	24000	31.9	0.00142	0.024	0.195	0.935	10.99	467	32000	42.5
400	24000	40.2	0.00180	0.030	0.216	0.928	9.96	533	32000	53.5
450	24000	49.1	0.00222	0.037	0.237	0.921	9.16	600	32000	65.5
500	24000	58.7	0.00267	0.044	0.257	0.914	8.52	667	32000	78.2
550	24000	68.7	0.00315	0.052	0.275	0.908	8.00	733	32000	91.7
600	24000	79.3	0.00366	0.061	0.293	0.902	7.57	800	32000	105.7
650	24000	90.3	0.00419	0.070	0.310	0.897	7.20	867	32000	120.4
700	24000	101.7	0.00475	0.079	0.326	0.891	6.89	933	32000	135.5
750	24000	113.4	0.00533	0.088	0.341	0.886	6.61	1000	32000	151.2
800	24000	125.4	0.00593	0.098	0.356	0.881	6.38	1067	32000	167.3
833	24000	133.7	0.00634	0.105	0.365	0.878	6.23	1111	32000	178.2
833	23000	136.8	0.00680	0.113	0.375	0.875	6.09	1111	30667	182.4
833	22000	140.0	0.00730	0.121	0.386	0.871	5.95	1111	29333	186.7
833	21000	143.4	0.00787	0.130	0.397	0.868	5.81	1111	28000	191.3
833	20000	147.0	0.00851	0.141	0.408	0.864	5.67	1111	26667	196.0
833	19000	150.8	0.00923	0.153	0.421	0.860	5.53	1111	25333	201.0
833	18000	154.7	0.01005	0.167	0.434	0.855	5.39	1111	24000	206.3
833	17000	158.9	0.01099	0.182	0.448	0.851	5.25	1111	22667	211.8
833	16000	163.2	0.01206	0.200	0.463	0.846	5.11	1111	21333	217.6
833	15000	167.8	0.01331	0.221	0.479	0.840	4.97	1111	20000	223.8
833	14000	172.7	0.01478	0.245	0.497	0.834	4.83	1111	18667	230.2
833	13000	177.8	0.01651	0.274	0.515	0.828	4.69	1111	17333	237.0
833	12000	183.2	0.01858	0.308	0.535	0.822	4.55	1111	16000	244.2
833	11000	188.9	0.02108	0.349	0.557	0.814	4.41	1111	14667	251.9
833	10000	194.9	0.02417	0.400	0.580	0.807	4.27	1111	13333	259.9
833	9000	201.4	0.02803	0.464	0.605	0.798	4.14	1111	12000	268.5
833	8000	208.1	0.03298	0.547	0.633	0.789	4.00	1111	10667	277.5
833	7000	215.3	0.03950	0.655	0.664	0.779	3.87	1111	9333	287.1
833	6000	223.0	0.04841	0.802	0.697	0.768	3.74	1111	8000	297.3

Diagram ASD-26a K_f Versus ρ for Various Masonry and Steel Stresses, Clay Masonry, f'_m = 2500 psi, n = 16.6

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

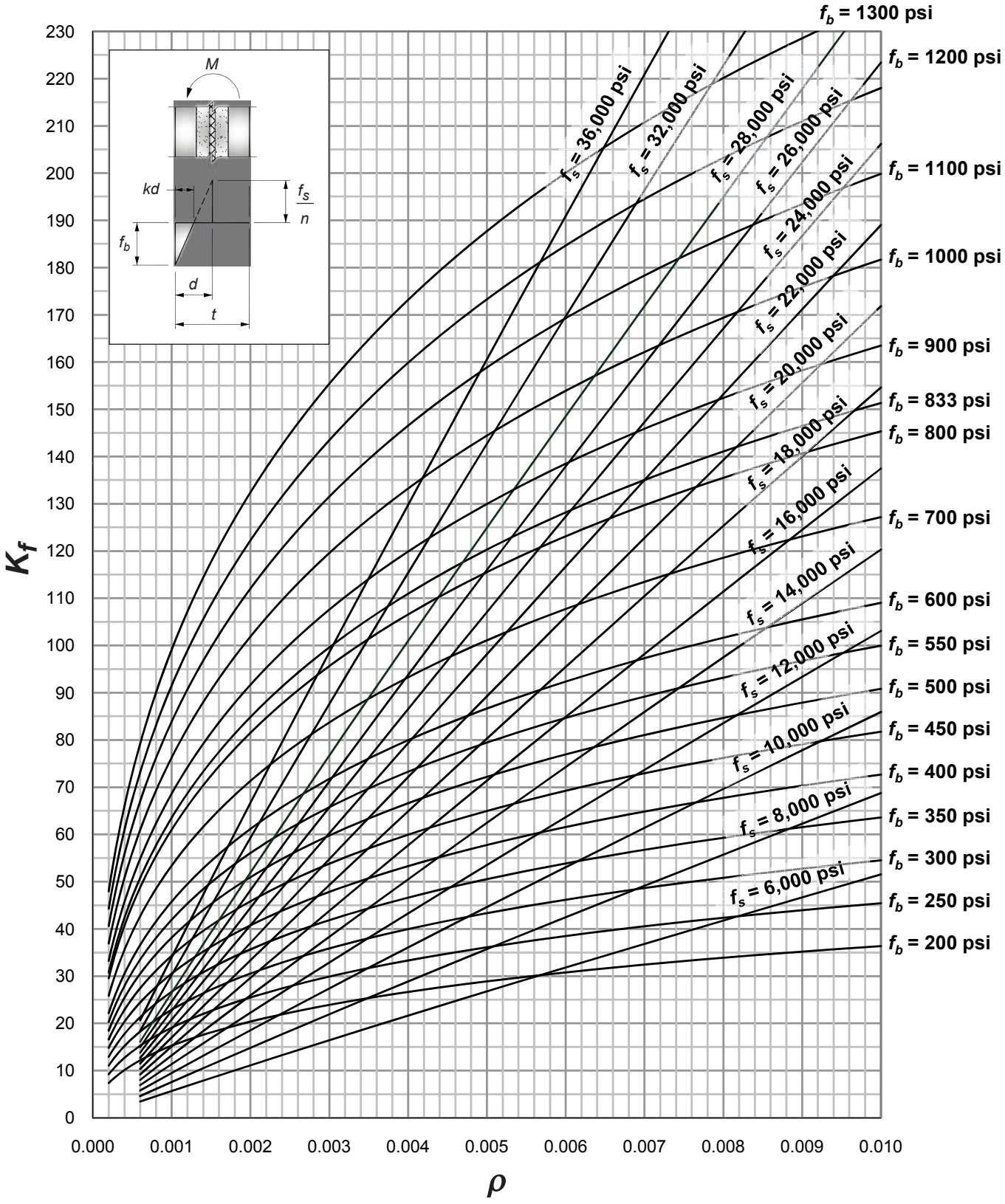
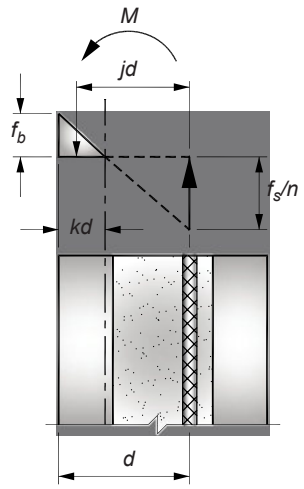


Table ASD-26b Flexural Design Coefficients for Allowable Stresses (Concrete Masonry) for f'_m = 2500 psi, f_y = 60,000 psi and n = 12.9



DESIGN DATA

f'_m = 2500 psi — f_y = 60,000 psi

$F_b = f'_m/3$ = 833 psi — F_s = 24,000 psi

$E_m = 900\ f'_m$ = 2,250,000 psi — E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 12.9 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 1111 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	f_b	f_s	K_f
50	24000	0.6	0.00003	0.000	0.026	0.991	77.16	67	32000	0.9
100	24000	2.5	0.00011	0.001	0.051	0.983	39.92	133	32000	3.3
150	24000	5.5	0.00023	0.003	0.075	0.975	27.51	200	32000	7.3
200	24000	9.4	0.00040	0.005	0.097	0.968	21.31	267	32000	12.5
250	24000	14.2	0.00062	0.008	0.118	0.961	17.59	333	32000	18.9
300	24000	19.9	0.00087	0.011	0.139	0.954	15.11	400	32000	26.5
350	24000	26.2	0.00115	0.015	0.158	0.947	13.34	467	32000	35.0
400	24000	33.3	0.00147	0.019	0.177	0.941	12.02	533	32000	44.4
450	24000	41.0	0.00182	0.024	0.195	0.935	10.99	600	32000	54.6
500	24000	49.2	0.00220	0.028	0.212	0.929	10.17	667	32000	65.6
550	24000	57.9	0.00261	0.034	0.228	0.924	9.49	733	32000	77.3
600	24000	67.2	0.00305	0.039	0.244	0.919	8.93	800	32000	89.6
650	24000	76.8	0.00350	0.045	0.259	0.914	8.46	867	32000	102.5
700	24000	86.9	0.00398	0.051	0.273	0.909	8.05	933	32000	115.9
750	24000	97.4	0.00449	0.058	0.287	0.904	7.70	1000	32000	129.8
800	24000	108.2	0.00501	0.065	0.301	0.900	7.40	1067	32000	144.2
833	24000	115.5	0.00537	0.069	0.309	0.897	7.21	1111	32000	154.1
833	23000	118.6	0.00577	0.074	0.318	0.894	7.03	1111	30667	158.1
833	22000	121.7	0.00621	0.080	0.328	0.891	6.85	1111	29333	162.3
833	21000	125.1	0.00671	0.087	0.338	0.887	6.66	1111	28000	166.8
833	20000	128.6	0.00728	0.094	0.349	0.884	6.48	1111	26667	171.5
833	19000	132.4	0.00792	0.102	0.361	0.880	6.30	1111	25333	176.5
833	18000	136.3	0.00865	0.111	0.374	0.875	6.11	1111	24000	181.8
833	17000	140.5	0.00949	0.122	0.387	0.871	5.93	1111	22667	187.3
833	16000	145.0	0.01046	0.135	0.402	0.866	5.75	1111	21333	193.3
833	15000	149.7	0.01159	0.149	0.417	0.861	5.57	1111	20000	199.6
833	14000	154.7	0.01292	0.167	0.434	0.855	5.39	1111	18667	206.3
833	13000	160.1	0.01450	0.187	0.452	0.849	5.21	1111	17333	213.4
833	12000	165.8	0.01640	0.211	0.472	0.843	5.03	1111	16000	221.1
833	11000	171.9	0.01871	0.241	0.494	0.835	4.85	1111	14667	229.3
833	10000	178.5	0.02158	0.278	0.518	0.827	4.67	1111	13333	238.0
833	9000	185.6	0.02519	0.325	0.544	0.819	4.49	1111	12000	247.5
833	8000	193.2	0.02985	0.385	0.573	0.809	4.31	1111	10667	257.6
833	7000	201.4	0.03604	0.464	0.605	0.798	4.14	1111	9333	268.5
833	6000	210.2	0.04456	0.574	0.642	0.786	3.97	1111	8000	280.2
833	5000	219.6	0.05686	0.733	0.682	0.773	3.79	1111	6667	292.9

Diagram ASD-26b K_f Versus ρ for Various Masonry and Steel Stresses, Concrete Masonry, f'_m = 2500 psi, n = 12.9

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

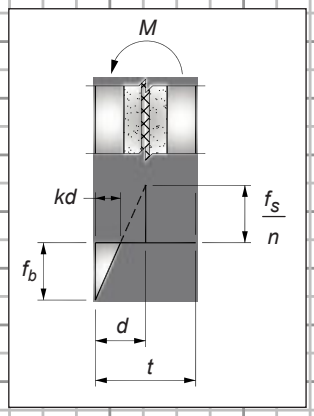
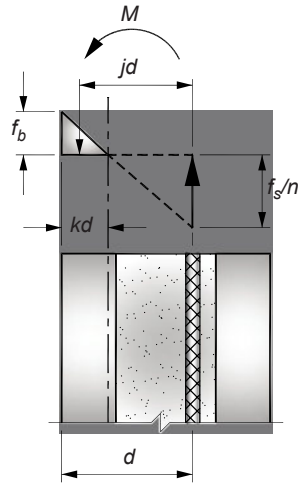


Table ASD-27a Flexural Design Coefficients for Allowable Stresses (Clay Masonry) for f'_m = 3000 psi, f_y = 60,000 psi and n = 13.8



DESIGN DATA

f'_m = 3000 psi
$F_b = f'_m/3$ = 1000 psi
$E_m = 700\ f'_m$ = 2,100,000 psi
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 13.8 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 1333 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	f_b	f_s	K_f
100	24000	2.7	0.00011	0.002	0.054	0.982	37.4	133	32000	3.6
150	24000	5.8	0.00025	0.003	0.079	0.974	25.86	200	32000	7.7
200	24000	10.0	0.00043	0.006	0.103	0.966	20.07	267	32000	13.3
250	24000	15.1	0.00065	0.009	0.126	0.958	16.60	333	32000	20.1
300	24000	21.0	0.00092	0.013	0.147	0.951	14.29	400	32000	28.0
350	24000	27.7	0.00122	0.017	0.168	0.944	12.64	467	32000	36.9
400	24000	35.1	0.00156	0.022	0.187	0.938	11.40	533	32000	46.8
450	24000	43.1	0.00193	0.027	0.206	0.931	10.44	600	32000	57.5
500	24000	51.7	0.00233	0.032	0.223	0.926	9.67	667	32000	68.9
550	24000	60.8	0.00275	0.038	0.240	0.920	9.04	733	32000	81.1
600	24000	70.4	0.00321	0.044	0.257	0.914	8.52	800	32000	93.9
650	24000	80.4	0.00369	0.051	0.272	0.909	8.08	867	32000	107.3
700	24000	90.9	0.00419	0.058	0.287	0.904	7.70	933	32000	121.2
750	24000	101.7	0.00471	0.065	0.301	0.900	7.38	1000	32000	135.6
800	24000	112.8	0.00525	0.073	0.315	0.895	7.09	1067	32000	150.5
850	24000	124.3	0.00582	0.080	0.328	0.891	6.84	1133	32000	165.7
900	24000	136.1	0.00640	0.088	0.341	0.886	6.61	1200	32000	181.4
950	24000	148.1	0.00700	0.097	0.353	0.882	6.41	1267	32000	197.5
1000	24000	160.4	0.00761	0.105	0.365	0.878	6.23	1333	32000	213.8
1000	23000	164.1	0.00816	0.113	0.375	0.875	6.09	1333	30667	218.8
1000	22000	168.0	0.00876	0.121	0.386	0.871	5.95	1333	29333	224.0
1000	21000	172.1	0.00945	0.130	0.397	0.868	5.81	1333	28000	229.5
1000	20000	176.4	0.01021	0.141	0.408	0.864	5.67	1333	26667	235.2
1000	19000	180.9	0.01108	0.153	0.421	0.860	5.53	1333	25333	241.2
1000	18000	185.7	0.01206	0.167	0.434	0.855	5.39	1333	24000	247.5
1000	17000	190.6	0.01318	0.182	0.448	0.851	5.25	1333	22667	254.2
1000	16000	195.9	0.01448	0.200	0.463	0.846	5.11	1333	21333	261.1
1000	15000	201.4	0.01598	0.221	0.479	0.840	4.97	1333	20000	268.5
1000	14000	207.2	0.01773	0.245	0.497	0.834	4.83	1333	18667	276.3
1000	13000	213.3	0.01981	0.274	0.515	0.828	4.69	1333	17333	284.4
1000	12000	219.8	0.02229	0.308	0.535	0.822	4.55	1333	16000	293.1
1000	11000	226.7	0.02530	0.349	0.557	0.814	4.41	1333	14667	302.2
1000	10000	233.9	0.02900	0.400	0.580	0.807	4.27	1333	13333	311.9
1000	9000	241.6	0.03363	0.464	0.605	0.798	4.14	1333	12000	322.2
1000	8000	249.8	0.03957	0.547	0.633	0.789	4.00	1333	10667	333.0
1000	7000	258.4	0.04740	0.655	0.664	0.779	3.87	1333	9333	344.5

Diagram ASD-27a K_f Versus ρ for Various Masonry and Steel Stresses, Clay Masonry, f'_m = 3000 psi, n = 13.8

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

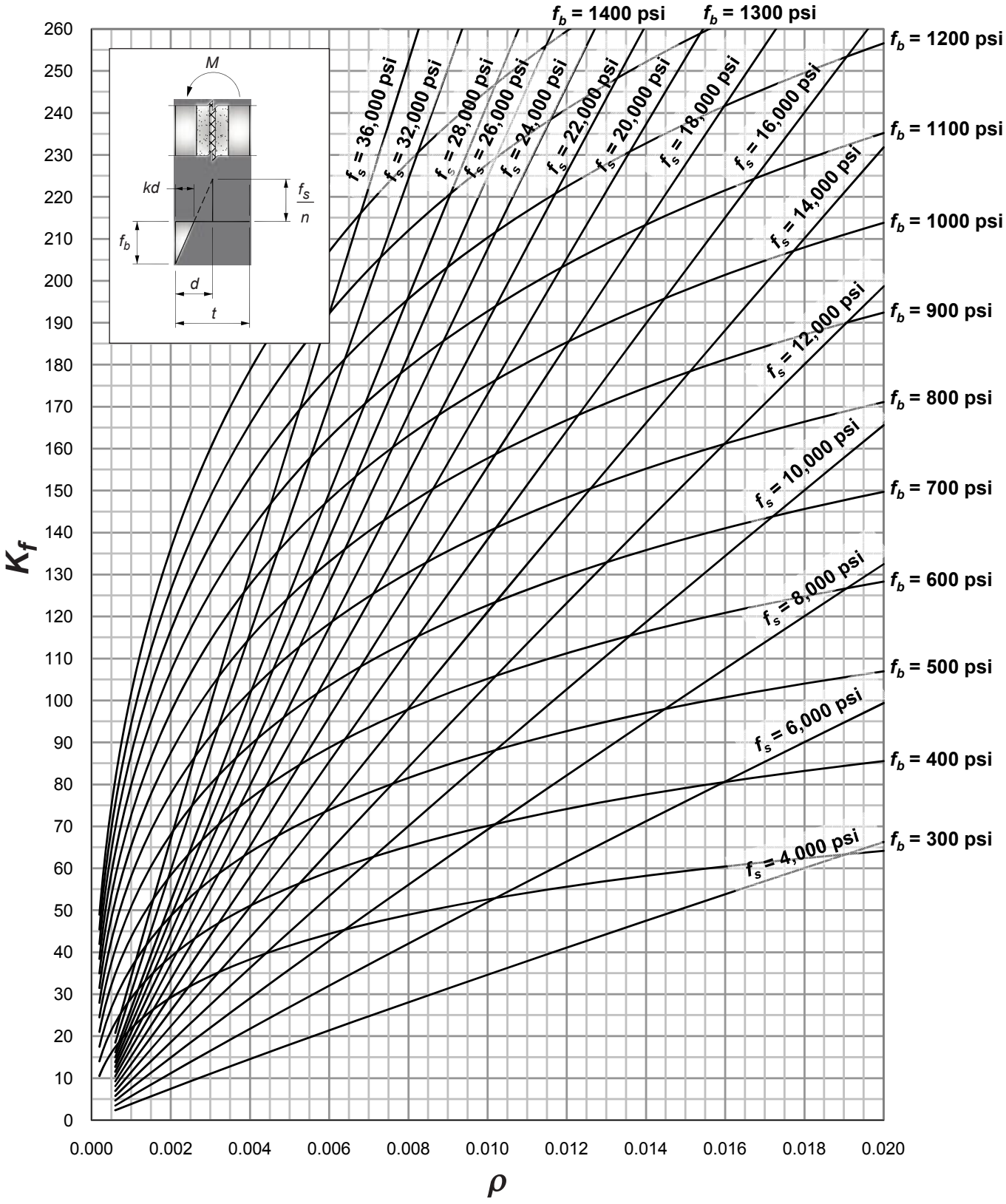
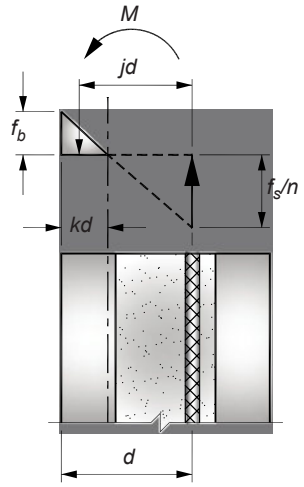


Table ASD-27b Flexural Design Coefficients for Allowable Stresses (Concrete Masonry) for f'_m = 3000 psi, f_y = 60,000 psi and n = 10.7



DESIGN DATA

f'_m = 3000 psi — f_y = 60,000 psi

$F_b = f'_m/3$ = 1000 psi — F_s = 24,000 psi

$E_m = 900\ f'_m$ = 2,700,000 psi — E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 10.7 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

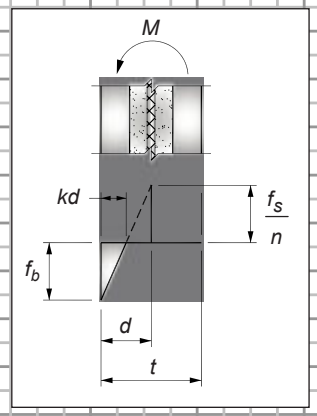
$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 1333 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	f_b	f_s	K_f
100	24000	2.1	0.00009	0.001	0.043	0.986	47.37	133	32000	2.8
150	24000	4.6	0.00020	0.002	0.063	0.979	32.47	200	32000	6.2
200	24000	8.0	0.00034	0.004	0.082	0.973	25.03	267	32000	10.7
250	24000	12.2	0.00052	0.006	0.101	0.966	20.57	333	32000	16.2
300	24000	17.1	0.00074	0.008	0.118	0.961	17.59	400	32000	22.7
350	24000	22.6	0.00099	0.011	0.135	0.955	15.47	467	32000	30.2
400	24000	28.8	0.00127	0.014	0.152	0.949	13.87	533	32000	38.4
450	24000	35.6	0.00157	0.017	0.168	0.944	12.64	600	32000	47.5
500	24000	42.9	0.00190	0.020	0.183	0.939	11.65	667	32000	57.2
550	24000	50.7	0.00226	0.024	0.198	0.934	10.84	733	32000	67.7
600	24000	59.0	0.00265	0.028	0.212	0.929	10.17	800	32000	78.7
650	24000	67.7	0.00305	0.033	0.225	0.925	9.60	867	32000	90.3
700	24000	76.9	0.00348	0.037	0.239	0.920	9.11	933	32000	102.5
750	24000	86.3	0.00393	0.042	0.251	0.916	8.69	1000	32000	115.1
800	24000	96.2	0.00439	0.047	0.264	0.912	8.32	1067	32000	128.2
850	24000	106.4	0.00488	0.052	0.276	0.908	7.99	1133	32000	141.8
900	24000	116.8	0.00538	0.058	0.287	0.904	7.70	1200	32000	155.8
950	24000	127.6	0.00590	0.063	0.298	0.901	7.44	1267	32000	170.1
1000	24000	138.7	0.00644	0.069	0.309	0.897	7.21	1333	32000	184.9
1000	23000	142.3	0.00692	0.074	0.318	0.894	7.03	1333	30667	189.7
1000	22000	146.1	0.00476	0.080	0.328	0.891	6.85	1333	29333	194.8
1000	21000	150.1	0.00806	0.087	0.338	0.887	6.66	1333	28000	200.1
1000	20000	154.4	0.00873	0.094	0.349	0.884	6.48	1333	26667	205.8
1000	19000	158.8	0.00950	0.102	0.361	0.880	6.30	1333	25333	211.8
1000	18000	163.6	0.01038	0.111	0.374	0.875	6.11	1333	24000	218.1
1000	17000	168.6	0.01139	0.122	0.387	0.871	5.93	1333	22667	224.8
1000	16000	173.9	0.01255	0.135	0.402	0.866	5.75	1333	21333	231.9
1000	15000	179.6	0.01391	0.149	0.417	0.861	5.57	1333	20000	239.5
1000	14000	185.7	0.01550	0.167	0.434	0.855	5.39	1333	18667	247.5
1000	13000	192.1	0.01740	0.187	0.452	0.849	5.21	1333	17333	256.1
1000	12000	199.0	0.01968	0.211	0.472	0.843	5.03	1333	16000	265.3
1000	11000	206.3	0.02246	0.241	0.494	0.835	4.85	1333	14667	275.1
1000	10000	214.2	0.02589	0.278	0.518	0.827	4.67	1333	13333	285.6
1000	9000	222.7	0.03023	0.325	0.544	0.819	4.49	1333	12000	296.9
1000	8000	231.8	0.03582	0.385	0.573	0.809	4.31	1333	10667	309.1
1000	7000	241.6	0.04324	0.464	0.605	0.798	4.14	1333	9333	322.2

Diagram ASD-27b K_f Versus ρ for Various Masonry and Steel Stresses, Concrete Masonry, f'_m = 3000 psi, n = 10.7

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$



K_f

320 310 300 290 280 270 260 250 240 230 220 210 200 190 180 170 160 150 140 130 120 110 100 90 80 70 60 50 40 30 20 10 0

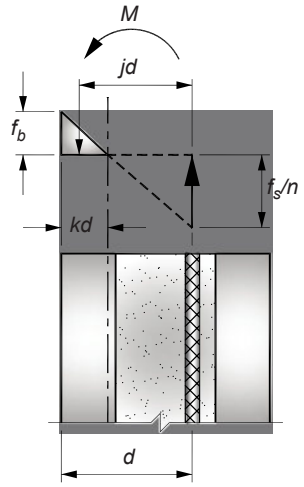
f_s = 36,000 psi
f_s = 32,000 psi
f_s = 28,000 psi
f_s = 26,000 psi
f_s = 24,000 psi
f_s = 22,000 psi
f_s = 20,000 psi
f_s = 18,000 psi
f_s = 16,000 psi
f_s = 14,000 psi
f_s = 12,000 psi
f_s = 10,000 psi
f_s = 8,000 psi
f_s = 6,000 psi
f_s = 4,000 psi

f_b = 1600 psi
f_b = 1500 psi
f_b = 1400 psi
f_b = 1300 psi
f_b = 1200 psi
f_b = 1100 psi
f_b = 1000 psi
f_b = 900 psi
f_b = 800 psi
f_b = 700 psi
f_b = 600 psi
f_b = 500 psi
f_b = 400 psi
f_b = 300 psi

0.000 0.002 0.004 0.006 0.008 0.010 0.012 0.014 0.016 0.018 0.020

ρ

Table ASD-28a Flexural Design Coefficients for Allowable Stresses (Clay Masonry) for f'_m = 3500 psi, f_y = 60,000 psi and n = 11.8



DESIGN DATA

f'_m = 3500 psi — f_y = 60,000 psi

$F_b = f'_m/3$ = 1167 psi — F_s = 24,000 psi

$E_m = 900\ f'_m$ = 2,450,000 psi — E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 11.8 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 1556 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	f_b	f_s	K_f
300	24000	18.5	0.00081	0.010	0.129	0.957	16.21	400	32000	24.7
350	24000	24.5	0.00107	0.013	0.147	0.951	14.29	467	32000	32.7
400	24000	31.1	0.00137	0.016	0.165	0.945	12.84	533	32000	41.5
450	24000	38.4	0.00170	0.020	0.182	0.939	11.72	600	32000	51.2
500	24000	46.2	0.00206	0.024	0.198	0.934	10.82	667	32000	61.6
550	24000	54.5	0.00244	0.029	0.213	0.929	10.09	733	32000	72.7
600	24000	63.3	0.00285	0.034	0.228	0.924	9.48	800	32000	84.4
650	24000	72.5	0.00329	0.039	0.243	0.919	8.96	867	32000	96.7
700	24000	82.1	0.00374	0.044	0.257	0.914	8.52	933	32000	109.5
750	24000	92.1	0.00422	0.050	0.270	0.910	8.14	1000	32000	122.9
800	24000	102.5	0.00472	0.056	0.283	0.906	7.81	1067	32000	136.7
850	24000	113.2	0.00523	0.062	0.295	0.902	7.51	1133	32000	150.9
900	24000	124.2	0.00576	0.068	0.307	0.898	7.25	1200	32000	165.6
950	24000	135.4	0.00631	0.075	0.319	0.894	7.01	1267	32000	180.6
1000	24000	147.0	0.00688	0.081	0.330	0.890	6.80	1333	32000	196.0
1050	24000	158.7	0.00746	0.088	0.341	0.886	6.61	1400	32000	211.7
1100	24000	170.8	0.00806	0.095	0.352	0.883	6.44	1467	32000	227.7
1150	24000	183.0	0.00867	0.103	0.362	0.879	6.28	1533	32000	244.0
1167	24000	187.1	0.00888	0.105	0.365	0.878	6.23	1556	32000	249.5
1167	23000	191.5	0.00951	0.113	0.375	0.875	6.09	1556	30667	255.3
1167	22000	196.0	0.01023	0.121	0.386	0.871	5.95	1556	29333	261.4
1167	21000	200.8	0.01102	0.130	0.397	0.868	5.81	1556	28000	267.8
1167	20000	205.8	0.01191	0.141	0.408	0.864	5.67	1556	26667	274.4
1167	19000	211.1	0.01292	0.153	0.421	0.860	5.53	1556	25333	281.4
1167	18000	216.6	0.01407	0.167	0.434	0.855	5.39	1556	24000	288.8
1167	17000	222.4	0.01538	0.182	0.448	0.851	5.25	1556	22667	296.5
1167	16000	228.5	0.01689	0.200	0.463	0.846	5.11	1556	21333	304.7
1167	15000	234.9	0.01864	0.221	0.479	0.840	4.97	1556	20000	313.3
1167	14000	241.7	0.02069	0.245	0.497	0.834	4.83	1556	18667	322.3
1167	13000	248.9	0.02311	0.274	0.515	0.828	4.69	1556	17333	331.8
1167	12000	256.4	0.02601	0.308	0.535	0.822	4.55	1556	16000	341.9
1167	11000	264.5	0.02952	0.349	0.557	0.814	4.41	1556	14667	352.6
1167	10000	272.9	0.03383	0.400	0.580	0.807	4.27	1556	13333	363.9
1167	9000	281.9	0.03924	0.464	0.605	0.798	4.14	1556	12000	375.9
1167	8000	291.4	0.04617	0.547	0.633	0.789	4.00	1556	10667	388.5
1167	7000	301.5	0.05530	0.655	0.664	0.779	3.87	1556	9333	402.0

Diagram ASD-28a K_f Versus ρ for Various Masonry and Steel Stresses, Clay Masonry, f'_m = 3500 psi, n = 11.8

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

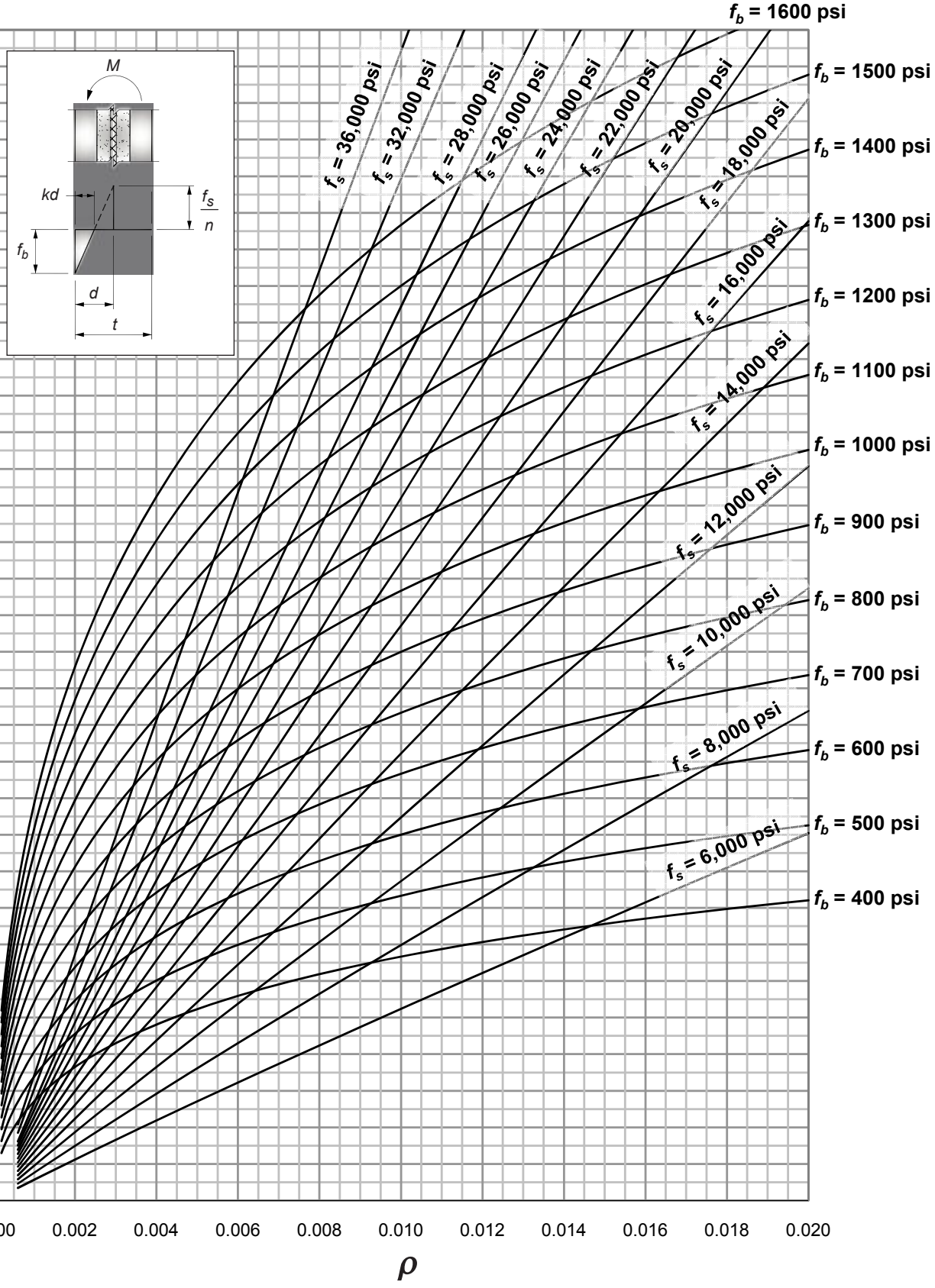
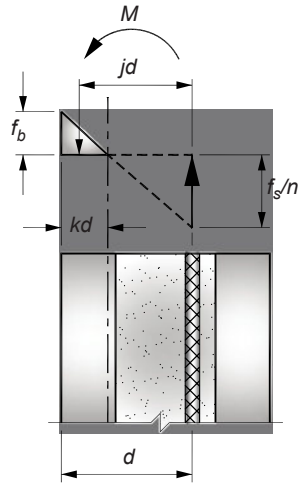


Table ASD-28b Flexural Design Coefficients for Allowable Stresses (Concrete Masonry) for f'_m = 3500 psi, f_y = 60,000 psi and n = 9.2



DESIGN DATA

f'_m = 3500 psi
$F_b = f'_m/3$ = 1167 psi
$E_m = 900\ f'_m$ = 3,150,000 psi
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 9.2 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 1556 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	f_b	f_s	K_f
300	24000	14.9	0.00065	0.006	0.103	0.966	20.07	400	32000	19.9
350	24000	19.9	0.00086	0.008	0.118	0.961	17.59	467	32000	26.5
400	24000	25.4	0.00111	0.010	0.133	0.956	15.73	533	32000	33.9
450	24000	31.5	0.00138	0.013	0.147	0.951	14.29	600	32000	42.0
500	24000	38.1	0.00168	0.015	0.161	0.946	13.13	667	32000	50.8
550	24000	45.1	0.00200	0.018	0.174	0.942	12.19	733	32000	60.2
600	24000	52.6	0.00234	0.022	0.187	0.938	11.40	800	32000	70.2
650	24000	60.5	0.00270	0.025	0.200	0.933	10.74	867	32000	80.7
700	24000	68.9	0.00309	0.028	0.212	0.929	10.17	933	32000	91.8
750	24000	77.5	0.00349	0.032	0.223	0.926	9.67	1000	32000	103.4
800	24000	86.6	0.00391	0.036	0.235	0.922	9.24	1067	32000	115.4
850	24000	95.9	0.00435	0.040	0.246	0.918	8.86	1133	32000	127.9
900	24000	105.6	0.00481	0.044	0.257	0.914	8.52	1200	32000	140.8
950	24000	115.6	0.00529	0.049	0.267	0.911	8.22	1267	32000	154.1
1000	24000	125.8	0.00578	0.053	0.277	0.908	7.95	1333	32000	167.7
1050	24000	136.3	0.00628	0.058	0.287	0.904	7.70	1400	32000	181.8
1100	24000	147.1	0.00680	0.063	0.297	0.901	7.48	1467	32000	196.1
1150	24000	158.1	0.00733	0.068	0.306	0.898	7.28	1533	32000	210.7
1167	24000	161.8	0.00751	0.069	0.309	0.897	7.21	1556	32000	215.7
1167	23000	166.0	0.00807	0.074	0.318	0.894	7.03	1556	30667	221.3
1167	22000	170.4	0.00870	0.080	0.328	0.891	6.85	1556	29333	227.3
1167	21000	175.1	0.00940	0.087	0.338	0.887	6.66	1556	28000	233.5
1167	20000	180.1	0.01019	0.094	0.349	0.884	6.48	1556	26667	240.1
1167	19000	185.3	0.01109	0.102	0.361	0.880	6.30	1556	25333	247.1
1167	18000	190.8	0.01211	0.111	0.374	0.875	6.11	1556	24000	254.5
1167	17000	196.7	0.01329	0.122	0.387	0.871	5.93	1556	22667	262.3
1167	16000	202.9	0.01464	0.135	0.402	0.866	5.75	1556	21333	270.6
1167	15000	209.6	0.01623	0.149	0.417	0.861	5.57	1556	20000	279.4
1167	14000	216.6	0.01809	0.167	0.434	0.855	5.39	1556	18667	288.8
1167	13000	224.1	0.02030	0.187	0.452	0.849	5.21	1556	17333	298.8
1167	12000	232.1	0.02296	0.211	0.472	0.843	5.03	1556	16000	309.5
1167	11000	240.7	0.02620	0.241	0.494	0.835	4.85	1556	14667	321.0
1167	10000	249.9	0.03021	0.278	0.518	0.827	4.67	1556	13333	333.3
1167	9000	259.8	0.03527	0.325	0.544	0.819	4.49	1556	12000	346.4
1167	8000	270.5	0.04179	0.385	0.573	0.809	4.31	1556	10667	360.6
1167	7000	281.9	0.05045	0.464	0.605	0.798	4.14	1556	9333	375.9

Diagram ASD-28b K_f Versus ρ for Various Masonry and Steel Stresses, Concrete Masonry, f'_m = 3500 psi, n = 9.2

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

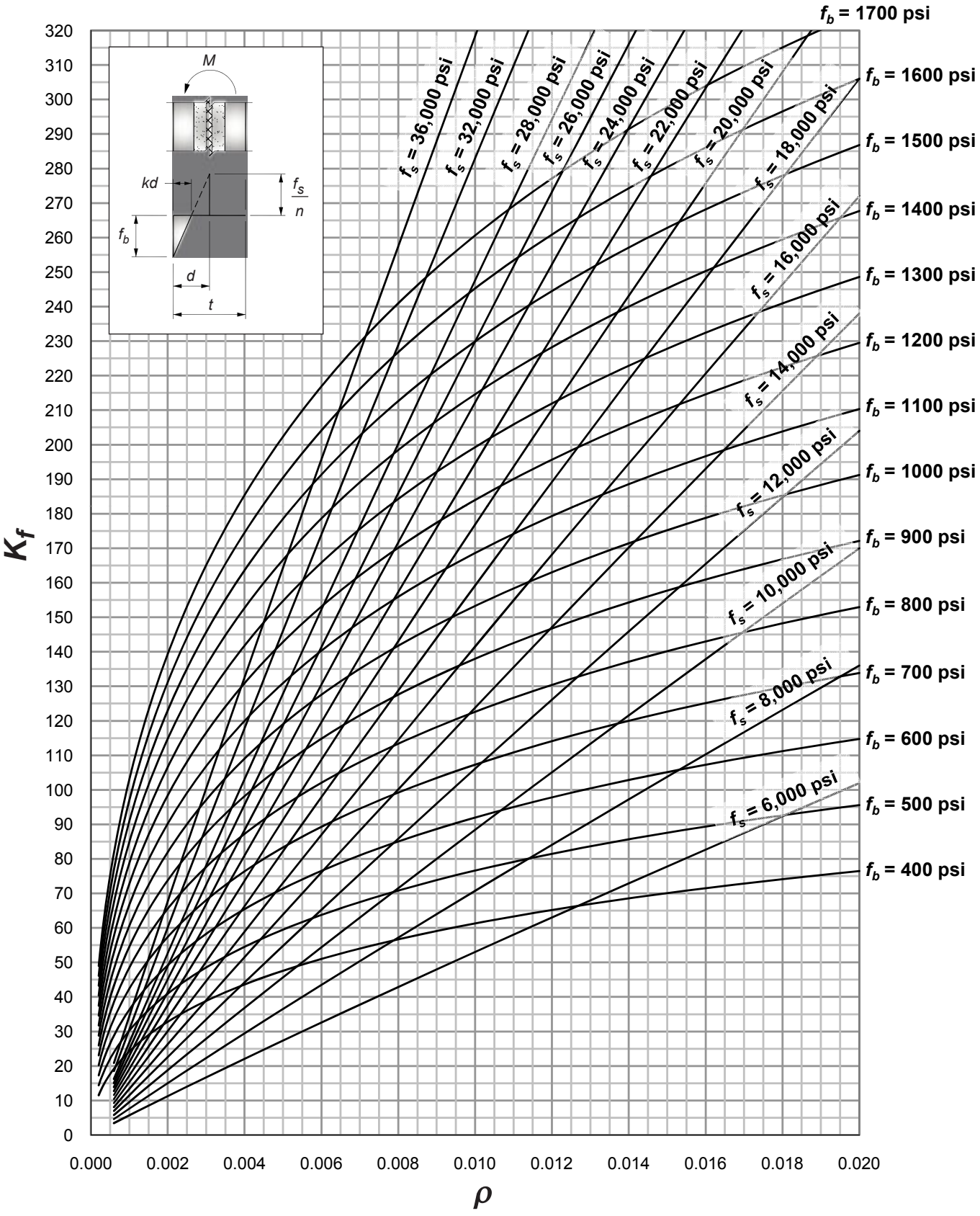
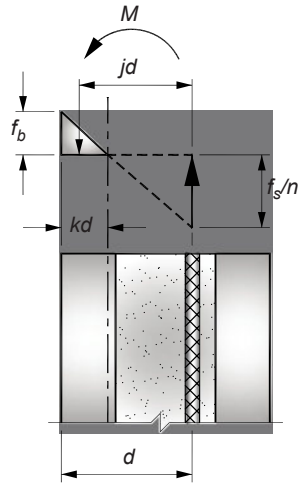


Table ASD-29a Flexural Design Coefficients for Allowable Stresses (Clay Masonry) for f'_m = 4000 psi, f_y = 60,000 psi and n = 10.4



DESIGN DATA

f'_m = 4000 psi
$F_b = f'_m/3$ = 1333 psi
$E_m = 700\ f'_m$ = 2,800,000 psi
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 10.4 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 1778 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	2 / jk	f_b	f_s	K_f
450	24000	34.6	0.00152	0.016	0.163	0.946	13.00	600	32000	46.1
500	24000	41.7	0.00185	0.019	0.177	0.941	11.98	667	32000	55.7
550	24000	49.4	0.00220	0.023	0.192	0.936	11.14	733	32000	65.8
600	24000	57.5	0.00257	0.027	0.206	0.931	10.44	800	32000	76.6
650	24000	66.0	0.00297	0.031	0.219	0.927	9.85	867	32000	88.0
700	24000	74.9	0.00338	0.035	0.232	0.923	9.34	933	32000	99.9
750	24000	84.2	0.00382	0.040	0.245	0.918	8.91	1000	32000	112.3
800	24000	93.9	0.00428	0.044	0.257	0.914	8.52	1067	32000	125.2
850	24000	103.9	0.00475	0.049	0.268	0.911	8.18	1133	32000	138.5
900	24000	114.1	0.00525	0.054	0.280	0.907	7.88	1200	32000	152.2
950	24000	124.7	0.00575	0.060	0.291	0.903	7.62	1267	32000	166.3
1000	24000	135.6	0.00628	0.065	0.301	0.900	7.38	1333	32000	180.8
1050	24000	146.7	0.00682	0.071	0.312	0.896	7.16	1400	32000	195.6
1100	24000	158.0	0.00738	0.076	0.322	0.893	6.96	1467	32000	210.7
1150	24000	169.6	0.00795	0.082	0.332	0.889	6.78	1533	32000	226.2
1200	24000	181.4	0.00853	0.088	0.341	0.886	6.61	1600	32000	241.9
1250	24000	193.4	0.00913	0.095	0.350	0.883	6.46	1667	32000	257.9
1300	24000	205.6	0.00973	0.101	0.359	0.880	6.32	1733	32000	274.2
1333	24000	213.8	0.01015	0.105	0.365	0.878	6.23	1778	32000	285.1
1333	23000	218.8	0.01087	0.113	0.375	0.875	6.09	1778	30667	291.8
1333	22000	224.0	0.01169	0.121	0.386	0.871	5.95	1778	29333	298.7
1333	21000	229.5	0.01259	0.130	0.397	0.868	5.81	1778	28000	306.0
1333	20000	235.2	0.01362	0.141	0.408	0.864	5.67	1778	26667	313.6
1333	19000	241.2	0.01477	0.153	0.421	0.860	5.53	1778	25333	321.6
1333	18000	247.5	0.01608	0.167	0.434	0.855	5.39	1778	24000	330.1
1333	17000	254.2	0.01758	0.182	0.448	0.851	5.25	1778	22667	338.9
1333	16000	261.1	0.01930	0.200	0.463	0.846	5.11	1778	21333	348.2
1333	15000	268.5	0.02130	0.221	0.479	0.840	4.97	1778	20000	358.0
1333	14000	276.3	0.02365	0.245	0.497	0.834	4.83	1778	18667	368.3
1333	13000	284.4	0.02642	0.274	0.515	0.828	4.69	1778	17333	379.2
1333	12000	293.1	0.02973	0.308	0.535	0.822	4.55	1778	16000	390.8
1333	11000	302.2	0.03373	0.349	0.557	0.814	4.41	1778	14667	403.0
1333	10000	311.9	0.03867	0.400	0.580	0.807	4.27	1778	13333	415.9
1333	9000	322.2	0.04485	0.464	0.605	0.798	4.14	1778	12000	429.6
1333	8000	333.0	0.05277	0.547	0.633	0.789	4.00	1778	10667	444.0
1333	7000	344.5	0.06320	0.655	0.664	0.779	3.87	1778	9333	459.4

Diagram ASD-29a K_f Versus ρ for Various Masonry and Steel Stresses, Clay Masonry, f'_m = 4000 psi, n = 10.4

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

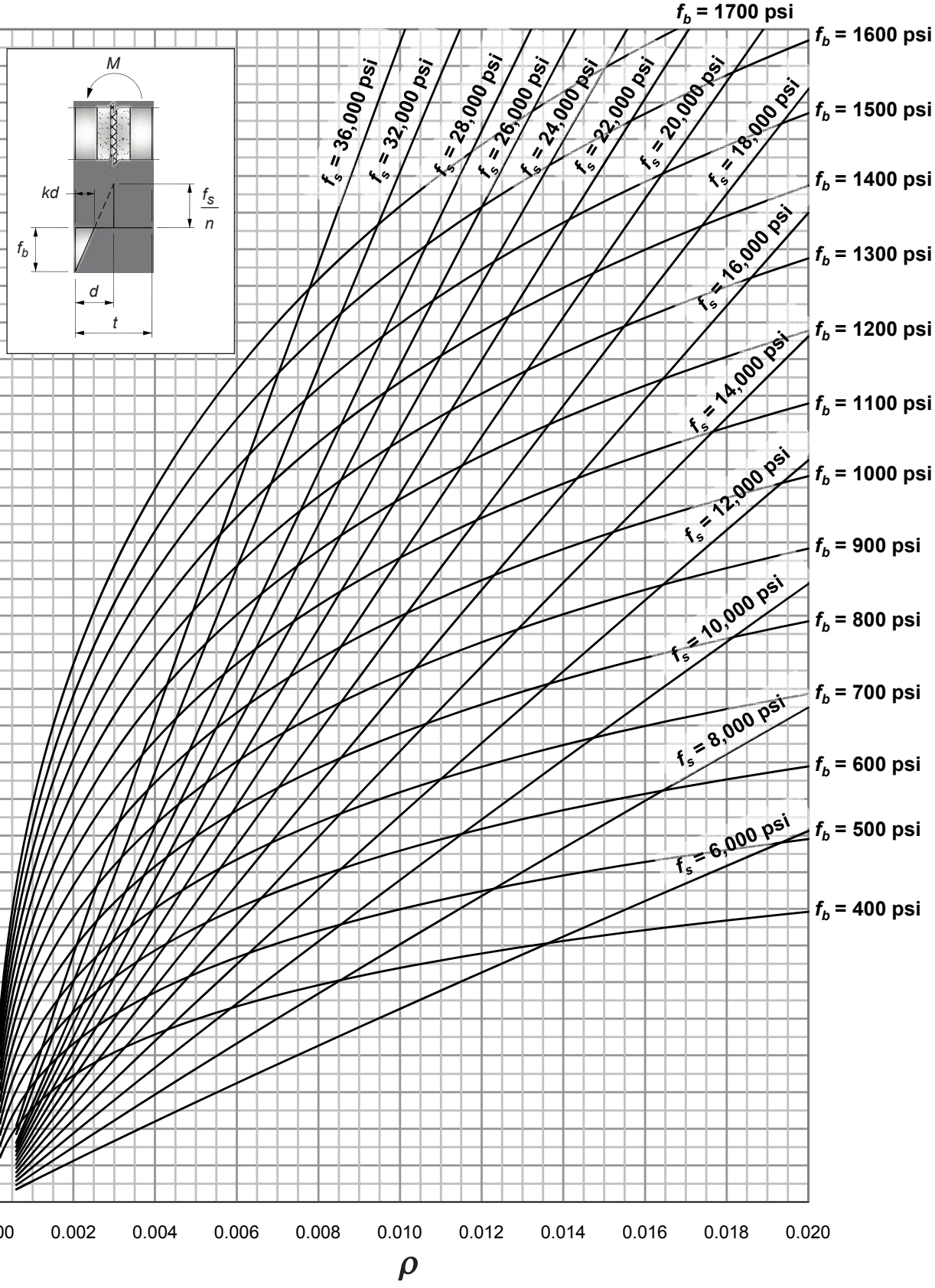
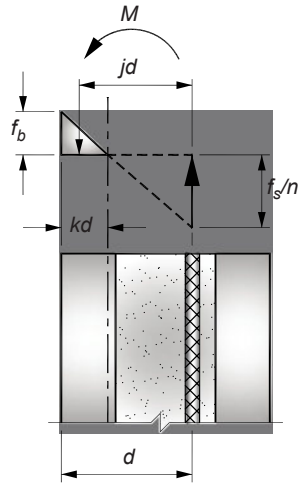


Table ASD-29b Flexural Design Coefficients for Allowable Stresses (Concrete Masonry) for f'_m = 4000 psi, f_y = 60,000 psi and n = 8.1



DESIGN DATA

f'_m = 4000 psi — f_y = 60,000 psi

$F_b = f'_m/3$ = 1333 psi — F_s = 24,000 psi

$E_m = 900\ f'_m$ = 3,600,000 psi — E_s = 29,000,000 psi

DESIGN EQUATIONS

$$n = \frac{E_s}{E_m} = 8.1 \qquad K_f = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$K_f = jkf_b/2$$

$$k = \frac{1}{1 + f_s/(nf_b)} \qquad j = 1 - \frac{k}{3}$$

$$A_s = \frac{M}{f_s jd} \qquad \rho = \frac{A_s}{bd} = \frac{K_f}{f_s j}$$

Increase for wind or earthquake
$4F_b/3$ = 1778 psi
$4F_s/3$ = 32,000 psi

f_b	f_s	K_f	ρ	$n\rho$	k	j	$2/jk$	f_b	f_s	K_f
450	24000	28.2	0.00123	0.010	0.131	0.956	15.94	600	32000	37.6
500	24000	34.2	0.00150	0.012	0.144	0.952	14.62	667	32000	45.6
550	24000	40.6	0.00179	0.014	0.156	0.948	13.54	733	32000	54.2
600	24000	47.5	0.00210	0.017	0.168	0.944	12.64	800	32000	63.3
650	24000	54.7	0.00243	0.020	0.179	0.940	11.88	867	32000	73.0
700	24000	62.4	0.00277	0.022	0.190	0.937	11.22	933	32000	83.2
750	24000	70.4	0.00314	0.025	0.201	0.933	10.66	1000	32000	93.8
800	24000	78.7	0.00353	0.028	0.212	0.929	10.17	1067	32000	104.9
850	24000	87.4	0.00393	0.032	0.222	0.926	9.73	1133	32000	116.5
900	24000	96.3	0.00435	0.035	0.232	0.923	9.34	1200	32000	128.4
950	24000	105.6	0.00479	0.039	0.242	0.919	9.00	1267	32000	140.8
1000	24000	115.1	0.00524	0.042	0.251	0.916	8.69	1333	32000	153.5
1050	24000	124.9	0.00570	0.046	0.261	0.913	8.40	1400	32000	166.6
1100	24000	135.0	0.00618	0.050	0.270	0.910	8.15	1467	32000	180.0
1150	24000	145.3	0.00667	0.054	0.278	0.907	7.92	1533	32000	193.7
1200	24000	155.8	0.00718	0.058	0.287	0.904	7.70	1600	32000	207.7
1250	24000	166.5	0.00770	0.062	0.296	0.901	7.51	1667	32000	222.0
1300	24000	177.5	0.00823	0.066	0.304	0.899	7.33	1733	32000	236.6
1333	24000	184.9	0.00859	0.069	0.309	0.897	7.21	1778	32000	246.5
1333	23000	189.7	0.00923	0.074	0.318	0.894	7.03	1778	30667	252.9
1333	22000	194.8	0.00994	0.080	0.328	0.891	6.85	1778	29333	259.7
1333	21000	200.1	0.01074	0.087	0.338	0.887	6.66	1778	28000	266.9
1333	20000	205.8	0.01165	0.094	0.349	0.884	6.48	1778	26667	274.4
1333	19000	211.8	0.01267	0.102	0.361	0.880	6.30	1778	25333	282.4
1333	18000	218.1	0.01384	0.111	0.374	0.875	6.11	1778	24000	290.8
1333	17000	224.8	0.01518	0.122	0.387	0.871	5.93	1778	22667	299.7
1333	16000	231.9	0.01674	0.135	0.402	0.866	5.75	1778	21333	309.2
1333	15000	239.5	0.01855	0.149	0.417	0.861	5.57	1778	20000	319.3
1333	14000	247.5	0.02067	0.167	0.434	0.855	5.39	1778	18667	330.1
1333	13000	256.1	0.02320	0.187	0.452	0.849	5.21	1778	17333	341.5
1333	12000	265.3	0.02624	0.211	0.472	0.843	5.03	1778	16000	353.7
1333	11000	275.1	0.02994	0.241	0.494	0.835	4.85	1778	14667	366.8
1333	10000	285.6	0.03452	0.278	0.518	0.827	4.67	1778	13333	380.9
1333	9000	296.9	0.04030	0.325	0.544	0.819	4.49	1778	12000	395.9
1333	8000	309.1	0.04776	0.385	0.573	0.809	4.31	1778	10667	412.1
1333	7000	322.2	0.05766	0.464	0.605	0.798	4.14	1778	9333	429.6

Diagram ASD-29b K_f Versus ρ for Various Masonry and Steel Stresses, Concrete Masonry, f'_m = 4000 psi, n = 8.1

$$K_f = \frac{M}{bd^2} \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = K_f\left(\frac{2}{jk}\right) \qquad \rho = \frac{A_s}{bd}$$

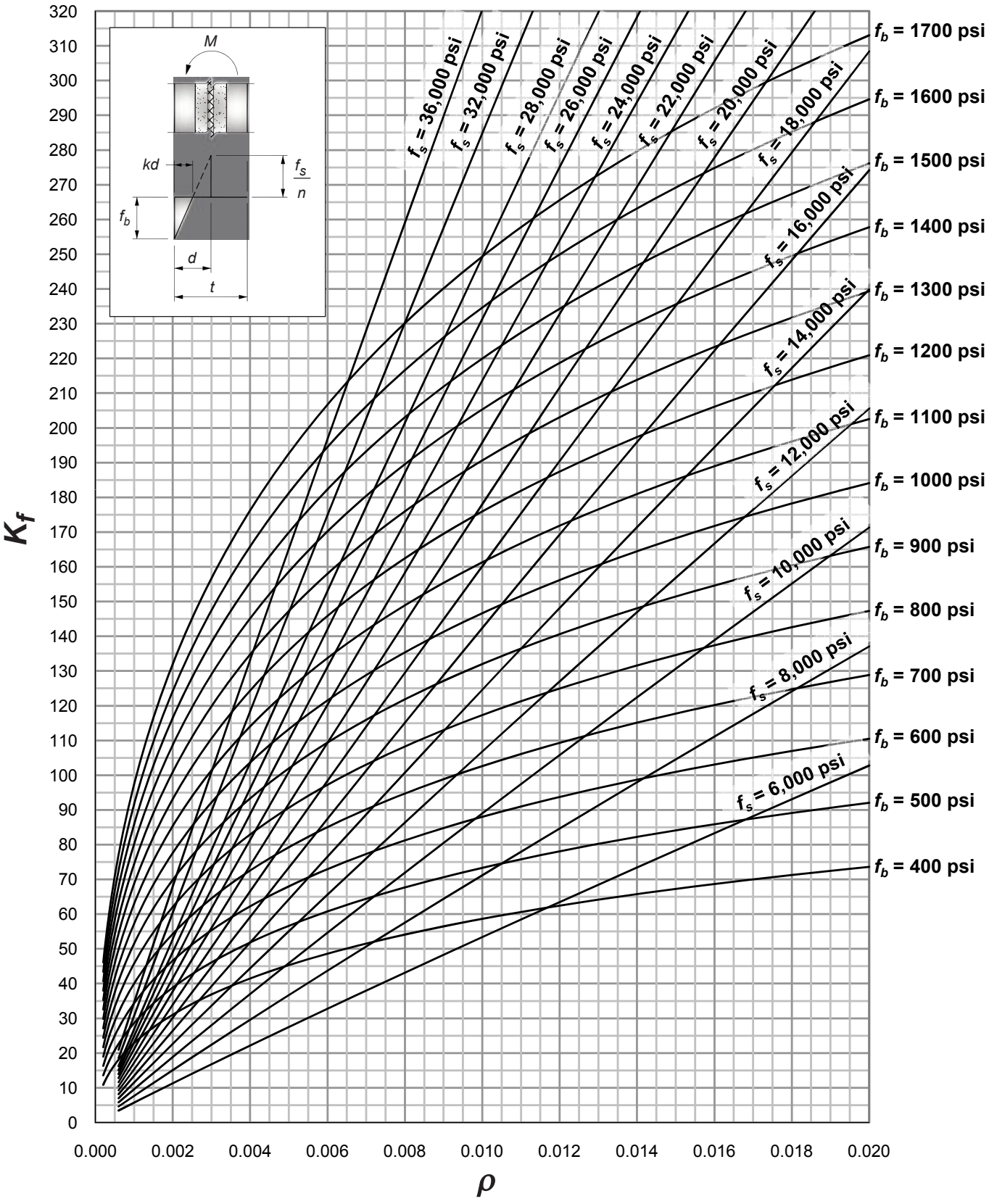


Diagram ASD-34 K_f Versus $n\rho$ for Various Masonry and Stresses f_b

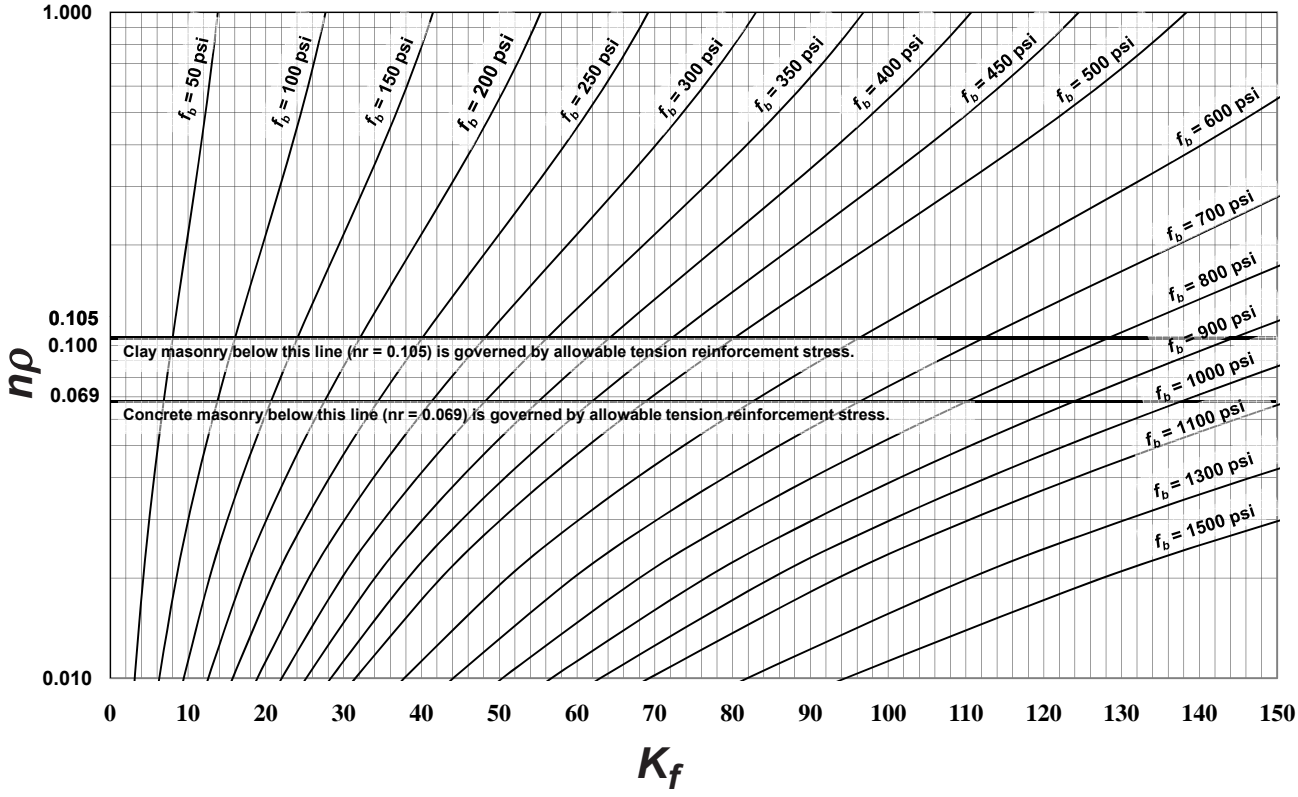


Table ASD-34a Flexural Coefficients Based on $n\rho$ Values

$$n = \frac{E_s}{E_m} \qquad \rho = \frac{A_s}{bd} \qquad k = \sqrt{2n\rho + (n\rho)^2} - n\rho \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right)$$

$$2/jk = \frac{bd^2 f_b}{M} \qquad n\rho\, j = \frac{nM}{bd^2 f_s} \qquad j = 1 - \frac{k}{3} \qquad f_b = \frac{M}{bd^2}\left(\frac{1}{\rho j}\right)$$

$$M_m = \frac{f_b jkbd^2}{2} = f_b bd^2\left(\frac{1}{2/jk}\right) \qquad M_s = f_s \rho\, jbd^2$$

$n\rho$	$2/jk$	j	k	$n\rho j$	$n\rho$	$2/jk$	j	k	$n\rho j$
0.001	46.409	0.985	0.044	0.0010	0.051	8.075	0.909	0.272	0.0464
0.002	33.319	0.980	0.061	0.0020	0.052	8.016	0.908	0.275	0.0472
0.003	27.523	0.975	0.075	0.0029	0.053	7.958	0.908	0.277	0.0481
0.004	24.069	0.971	0.086	0.0039	0.054	7.902	0.907	0.279	0.0490
0.005	21.713	0.968	0.095	0.0048	0.055	7.848	0.906	0.281	0.0498
0.006	19.975	0.965	0.104	0.0058	0.056	7.795	0.906	0.283	0.0507
0.007	18.625	0.963	0.112	0.0067	0.057	7.744	0.905	0.285	0.0516
0.008	17.537	0.960	0.119	0.0077	0.058	7.694	0.904	0.287	0.0524
0.009	16.636	0.958	0.125	0.0086	0.059	7.645	0.903	0.290	0.0533
0.010	15.875	0.956	0.132	0.0096	0.060	7.598	0.903	0.292	0.0542
0.011	15.220	0.954	0.138	0.0105	0.061	7.552	0.902	0.294	0.0550
0.012	14.649	0.952	0.143	0.0114	0.062	7.507	0.901	0.296	0.0559
0.013	14.145	0.950	0.149	0.0124	0.063	7.462	0.901	0.298	0.0568
0.014	13.697	0.949	0.154	0.0133	0.064	7.419	0.900	0.299	0.0576
0.015	13.294	0.947	0.159	0.0142	0.065	7.378	0.900	0.301	0.0585
0.016	12.930	0.945	0.164	0.0151	0.066	7.337	0.899	0.303	0.0593
0.017	12.599	0.944	0.168	0.0160	0.067	7.296	0.898	0.305	0.0602
0.018	12.296	0.942	0.173	0.0170	0.068	7.257	0.898	0.307	0.0610
0.019	12.017	0.941	0.177	0.0179	0.069	7.219	0.897	0.309	0.0619
0.020	11.759	0.940	0.181	0.0188	0.070	7.182	0.896	0.311	0.0628
0.021	11.521	0.938	0.185	0.0197	0.071	7.145	0.896	0.312	0.0636
0.022	11.298	0.937	0.189	0.0206	0.072	7.109	0.895	0.314	0.0645
0.023	11.091	0.936	0.193	0.0215	0.073	7.074	0.895	0.316	0.0653
0.024	10.897	0.935	0.196	0.0224	0.074	7.040	0.894	0.318	0.0662
0.025	10.714	0.933	0.200	0.0233	0.075	7.006	0.894	0.319	0.0670
0.026	10.543	0.932	0.204	0.0242	0.076	6.973	0.893	0.321	0.0679
0.027	10.381	0.931	0.207	0.0251	0.077	6.941	0.892	0.323	0.0687
0.028	10.227	0.930	0.210	0.0260	0.078	6.909	0.892	0.325	0.0696
0.029	10.082	0.929	0.214	0.0269	0.079	6.878	0.891	0.326	0.0704
0.030	9.945	0.928	0.217	0.0278	0.080	6.848	0.891	0.328	0.0713
0.031	9.814	0.927	0.220	0.0287	0.081	6.818	0.890	0.330	0.0721
0.032	9.689	0.926	0.223	0.0296	0.082	6.788	0.890	0.331	0.0729
0.033	9.570	0.925	0.226	0.0305	0.083	6.759	0.889	0.333	0.0738
0.034	9.456	0.924	0.229	0.0314	0.084	6.731	0.889	0.334	0.0746
0.035	9.348	0.923	0.232	0.0323	0.085	6.703	0.888	0.336	0.0755
0.036	9.244	0.922	0.235	0.0332	0.086	6.676	0.887	0.338	0.0763
0.037	9.144	0.921	0.238	0.0341	0.087	6.649	0.887	0.339	0.0772
0.038	9.048	0.920	0.240	0.0350	0.088	6.623	0.886	0.341	0.0780
0.039	8.956	0.919	0.243	0.0358	0.089	6.597	0.886	0.342	0.0788
0.040	8.868	0.918	0.246	0.0367	0.090	6.572	0.885	0.344	0.0797
0.041	8.782	0.917	0.248	0.0376	0.091	6.547	0.885	0.345	0.0805
0.042	8.700	0.916	0.251	0.0385	0.092	6.522	0.884	0.347	0.0814
0.043	8.621	0.916	0.253	0.0394	0.093	6.498	0.884	0.348	0.0822
0.044	8.545	0.915	0.256	0.0402	0.094	6.474	0.883	0.350	0.0830
0.045	8.471	0.914	0.258	0.0411	0.095	6.451	0.883	0.351	0.0839
0.046	8.399	0.913	0.261	0.0420	0.096	6.428	0.882	0.353	0.0847
0.047	8.330	0.912	0.263	0.0429	0.097	6.405	0.882	0.354	0.0856
0.048	8.263	0.911	0.266	0.0438	0.098	6.383	0.882	0.355	0.0864
0.049	8.199	0.911	0.268	0.0446	0.099	6.361	0.881	0.357	0.0872
0.050	8.136	0.910	0.270	0.0455	0.100	6.340	0.881	0.358	0.0881

Table ASD-34b Flexural Coefficients Based on $n\rho$ Values

$$n = \frac{E_s}{E_m} \qquad \rho = \frac{A_s}{bd} \qquad k = \sqrt{2n\rho + (n\rho)^2} - n\rho \qquad f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right)$$

$$2/jk = \frac{bd^2 f_b}{M} \qquad n\rho\, j = \frac{nM}{bd^2 f_s} \qquad j = 1 - \frac{k}{3} \qquad f_b = \frac{M}{bd^2}\left(\frac{1}{\rho j}\right)$$

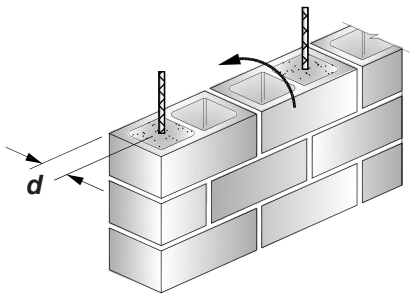
$$M_m = \frac{f_b jkbd^2}{2} = f_b bd^2\left(\frac{1}{2/jk}\right) \qquad M_s = f_s \rho\, jbd^2$$

$n\rho$	$2/jk$	j	k	$n\rho\, j$	$n\rho$	$2/jk$	j	k	$n\rho\, j$
0.101	6.318	0.880	0.360	0.0889	0.162	5.431	0.857	0.430	0.1388
0.102	6.297	0.880	0.361	0.0897	0.164	5.411	0.856	0.432	0.1404
0.103	6.277	0.879	0.362	0.0906	0.166	5.392	0.855	0.434	0.1420
0.104	6.257	0.879	0.364	0.0914	0.168	5.372	0.855	0.436	0.1436
0.105	6.237	0.878	0.365	0.0922	0.170	5.353	0.854	0.437	0.1452
0.106	6.217	0.878	0.366	0.0931	0.172	5.335	0.854	0.439	0.1468
0.107	6.197	0.877	0.368	0.0939	0.174	5.316	0.853	0.441	0.1484
0.108	6.178	0.877	0.369	0.0947	0.176	5.298	0.852	0.443	0.1500
0.109	6.159	0.877	0.370	0.0955	0.178	5.281	0.852	0.445	0.1516
0.110	6.141	0.876	0.372	0.0964	0.180	5.263	0.851	0.446	0.1532
0.111	6.122	0.876	0.373	0.0972	0.182	5.246	0.851	0.448	0.1548
0.112	6.104	0.875	0.374	0.0980	0.184	5.230	0.850	0.450	0.1564
0.113	6.086	0.875	0.376	0.0989	0.186	5.213	0.849	0.452	0.1580
0.114	6.069	0.874	0.377	0.0997	0.188	5.197	0.849	0.453	0.1596
0.115	6.051	0.874	0.378	0.1005	0.190	5.181	0.848	0.455	0.1612
0.116	6.034	0.874	0.379	0.1013	0.192	5.165	0.848	0.457	0.1628
0.117	6.017	0.873	0.381	0.1022	0.194	5.150	0.847	0.458	0.1644
0.118	6.001	0.873	0.382	0.1030	0.196	5.135	0.847	0.460	0.1659
0.119	5.984	0.872	0.383	0.1038	0.198	5.120	0.846	0.462	0.1675
0.120	5.968	0.872	0.384	0.1046	0.200	5.105	0.846	0.463	0.1691
0.121	5.952	0.871	0.386	0.1054	0.202	5.091	0.845	0.465	0.1707
0.122	5.936	0.871	0.387	0.1063	0.204	5.076	0.844	0.467	0.1723
0.123	5.920	0.871	0.388	0.1071	0.206	5.062	0.844	0.468	0.1739
0.124	5.905	0.870	0.389	0.1079	0.208	5.049	0.843	0.470	0.1754
0.125	5.890	0.870	0.390	0.1087	0.210	5.035	0.843	0.471	0.1770
0.126	5.874	0.869	0.392	0.1096	0.212	5.022	0.842	0.473	0.1786
0.127	5.860	0.869	0.393	0.1104	0.214	5.008	0.842	0.474	0.1802
0.128	5.845	0.869	0.394	0.1112	0.216	4.995	0.841	0.476	0.1817
0.129	5.830	0.868	0.395	0.1120	0.218	4.983	0.841	0.477	0.1833
0.130	5.816	0.868	0.396	0.1128	0.220	4.970	0.840	0.479	0.1849
0.131	5.802	0.868	0.397	0.1136	0.222	4.957	0.840	0.480	0.1865
0.132	5.788	0.867	0.398	0.1145	0.224	4.945	0.839	0.482	0.1880
0.133	5.774	0.867	0.400	0.1153	0.226	4.933	0.839	0.483	0.1896
0.134	5.760	0.866	0.401	0.1161	0.228	4.921	0.838	0.485	0.1912
0.135	5.747	0.866	0.402	0.1169	0.230	4.909	0.838	0.486	0.1927
0.136	5.733	0.866	0.403	0.1177	0.232	4.898	0.837	0.488	0.1943
0.137	5.720	0.865	0.404	0.1185	0.234	4.886	0.837	0.489	0.1959
0.138	5.707	0.865	0.405	0.1194	0.236	4.875	0.837	0.490	0.1974
0.139	5.694	0.865	0.406	0.1202	0.238	4.864	0.836	0.492	0.1990
0.140	5.681	0.864	0.407	0.1210	0.240	4.853	0.836	0.493	0.2005
0.142	5.656	0.863	0.410	0.1226	0.242	4.842	0.835	0.495	0.2021
0.144	5.631	0.863	0.412	0.1242	0.244	4.831	0.835	0.496	0.2037
0.146	5.607	0.862	0.414	0.1259	0.246	4.821	0.834	0.497	0.2052
0.148	5.584	0.861	0.416	0.1275	0.248	4.810	0.834	0.499	0.2068
0.150	5.560	0.861	0.418	0.1291	0.250	4.800	0.833	0.500	0.2083
0.152	5.538	0.860	0.420	0.1307	0.252	4.790	0.833	0.501	0.2099
0.154	5.516	0.859	0.422	0.1323	0.254	4.780	0.832	0.503	0.2114
0.156	5.494	0.859	0.424	0.1340	0.256	4.770	0.832	0.504	0.2130
0.158	5.473	0.858	0.426	0.1356	0.258	4.760	0.832	0.505	0.2145
0.160	5.452	0.857	0.428	0.1372	0.260	4.750	0.831	0.507	0.2161

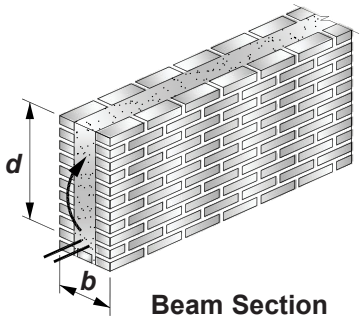
Table ASD-36 Moment Capacity of Walls and Beams for Balanced Design Conditions for f'_m = 1500 psi and f_y = 60,000 psi

$$M = \frac{K_f bd^2}{12{,}000} \text{ (ft kips)}$$

Tables based on b = 12" with d in inches and K_{fb} in psi



Wall Section



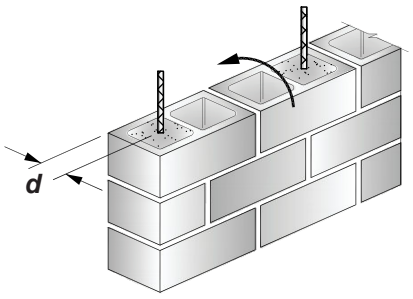
Beam Section

Effective Depth to Reinf. Steel, d, (inches)	Concrete Masonry			Clay Masonry		
	f_b = 500 psi f_s = 24,000 psi K_{fb} = 69.33	$4f_b/3$ = 667 psi $4f_s/3$ = 32,000 psi K_{fb} = 92.44	ρ_b = 0.003220	f_b = 500 psi f_s = 24,000 psi K_{fb} = 80.19	$4f_b/3$ = 667 psi $4f_s/3$ = 32,000 psi K_{fb} = 106.9	ρ_b = 0.003805
	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)
2.75	0.52	0.70	0.11	0.61	0.81	0.13
3	0.62	0.83	0.12	0.72	0.96	0.14
3.5	0.85	1.13	0.14	0.98	1.31	0.16
4	1.11	1.48	0.15	1.28	1.71	0.18
4.5	1.40	1.87	0.17	1.62	2.17	0.21
5	1.73	2.31	0.19	2.00	2.67	0.23
5.5	2.10	2.80	0.21	2.43	3.23	0.25
6	2.50	3.33	0.23	2.89	3.85	0.27
6.5	2.93	3.91	0.25	3.39	4.52	0.30
7	3.40	4.53	0.27	3.93	5.24	0.32
7.5	3.90	5.20	0.29	4.51	6.01	0.34
8	4.44	5.92	0.31	5.13	6.84	0.37
9	5.62	7.49	0.35	6.50	8.66	0.41
10	6.93	9.24	0.39	8.02	10.69	0.46
11	8.39	11.18	0.43	9.70	12.94	0.50
12	9.98	13.31	0.46	11.55	15.40	0.55
13	11.72	15.62	0.50	13.55	18.07	0.59
14	13.59	18.12	0.54	15.72	20.96	0.64
15	15.60	20.80	0.58	18.04	24.06	0.68
16	17.75	23.66	0.62	20.53	27.37	0.73
20	27.73	36.97	0.77	32.08	42.77	0.91
24	39.93	53.24	0.93	46.19	61.59	1.10
28	54.35	72.47	1.08	62.87	83.83	1.28
32	70.99	94.65	1.24	82.12	109.5	1.46
36	89.85	119.8	1.39	103.9	138.6	1.64
40	110.9	147.9	1.55	128.3	171.1	1.83
44	134.2	179.0	1.70	155.2	207.0	2.01
48	159.7	213.0	1.86	184.8	246.4	2.19
52	187.5	250.0	2.01	216.8	289.1	2.37
56	217.4	289.9	2.16	251.5	335.3	2.56
60	249.6	332.8	2.32	288.7	384.9	2.74
64	284.0	378.6	2.47	328.5	438.0	2.92
68	320.6	427.4	2.63	370.8	494.4	3.10

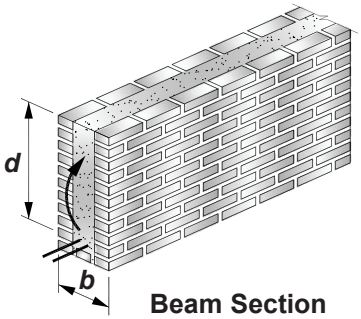
Table ASD-37 Moment Capacity of Walls and Beams for Balanced Design Conditions for f'_m = 2000 psi and f_y = 60,000 psi

$$M = \frac{K_f b d^2}{12{,}000} \text{ (ft kips)}$$

Tables based on b = 12" with d in inches and K_{fb} in psi



Wall Section



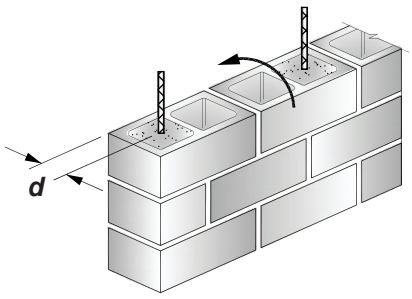
Beam Section

Effective Depth to Reinf. Steel, d, (inches)	Concrete Masonry			Clay Masonry		
	f_b = 667 psi f_s = 24,000 psi K_{fb} = 92.44	$4f_b/3$ = 889 psi $4f_s/3$ = 32,000 psi K_{fb} = 123.2	ρ_b = 0.004294	f_b = 667 psi f_s = 24,000 psi K_{fb} = 106.9	$4f_b/3$ = 889 psi $4f_s/3$ = 32,000 psi K_{fb} = 142.6	ρ_b = 0.005073
	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)
2.75	0.70	0.93	0.14	0.81	1.08	0.17
3	0.83	1.11	0.15	0.96	1.28	0.18
3.5	1.13	1.51	0.18	1.31	1.75	0.21
4	1.48	1.97	0.21	1.71	2.28	0.24
4.5	1.87	2.50	0.23	2.17	2.89	0.27
5	2.31	3.08	0.26	2.67	3.56	0.30
5.5	2.80	3.73	0.28	3.23	4.31	0.33
6	3.33	4.44	0.31	3.85	5.13	0.37
6.5	3.91	5.21	0.33	4.52	6.02	0.40
7	4.53	6.04	0.36	5.24	6.99	0.43
7.5	5.20	6.93	0.39	6.01	8.02	0.46
8	5.92	7.89	0.41	6.84	9.12	0.49
9	7.49	9.98	0.46	8.66	11.55	0.55
10	9.24	12.32	0.52	10.69	14.26	0.61
11	11.18	14.91	0.57	12.94	17.25	0.67
12	13.31	17.75	0.62	15.40	20.53	0.73
13	15.62	20.83	0.67	18.07	24.09	0.79
14	18.12	24.16	0.72	20.96	27.94	0.85
15	20.80	27.73	0.77	24.06	32.08	0.91
16	23.66	31.55	0.82	27.37	36.50	0.97
20	36.97	49.30	1.03	42.77	57.03	1.22
24	53.24	70.99	1.24	61.59	82.12	1.46
28	72.47	96.63	1.44	83.83	111.8	1.70
32	94.65	126.2	1.65	109.5	146.0	1.95
36	119.8	159.7	1.86	138.6	184.8	2.19
40	147.9	197.2	2.06	171.1	228.1	2.43
44	179.0	238.6	2.27	207.0	276.0	2.68
48	213.0	284.0	2.47	246.4	328.5	2.92
52	250.0	333.3	2.68	289.1	385.5	3.17
56	289.9	386.5	2.89	335.3	447.1	3.41
60	332.8	443.7	3.09	384.9	513.2	3.65
64	378.6	504.8	3.30	438.0	584.0	3.90
68	427.4	569.9	3.50	494.4	659.2	4.14

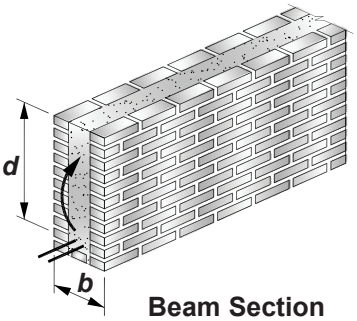
Table ASD-38 Moment Capacity of Walls and Beams for Balanced Design Conditions for f'_m = 2500 psi and f_y = 60,000 psi

$$M = \frac{K_f b d^2}{12{,}000} \text{ (ft kips)}$$

Tables based on b = 12" with d in inches and K_{fb} in psi



Wall Section



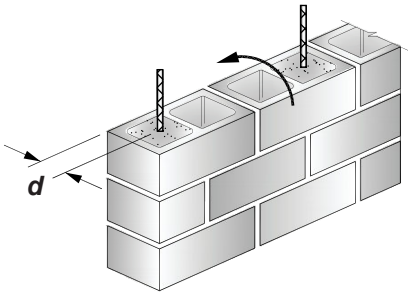
Beam Section

Effective Depth to Reinf. Steel, d, (inches)	Concrete Masonry			Clay Masonry		
	f_b = 833 psi f_s = 24,000 psi K_{fb} = 115.5	$4f_b/3$ = 1111 psi $4f_s/3$ = 32,000 psi K_{fb} = 154.1	ρ_b = 0.005368	f_b = 833 psi f_s = 24,000 psi K_{fb} = 133.7	$4f_b/3$ = 1111 psi $4f_s/3$ = 32,000 psi K_{fb} = 178.2	ρ_b = 0.006341
	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)
2.75	0.87	1.17	0.18	1.01	1.35	0.21
3	1.04	1.39	0.19	1.20	1.60	0.23
3.5	1.42	1.89	0.23	1.64	2.18	0.27
4	1.85	2.46	0.26	2.14	2.85	0.30
4.5	2.34	3.12	0.29	2.71	3.61	0.34
5	2.89	3.85	0.32	3.34	4.46	0.38
5.5	3.50	4.66	0.35	4.04	5.39	0.42
6	4.16	5.55	0.39	4.81	6.42	0.46
6.5	4.88	6.51	0.42	5.65	7.53	0.49
7	5.66	7.55	0.45	6.55	8.73	0.53
7.5	6.50	8.67	0.48	7.52	10.02	0.57
8	7.39	9.86	0.52	8.55	11.41	0.61
9	9.36	12.48	0.58	10.83	14.43	0.68
10	11.55	15.41	0.64	13.37	17.82	0.76
11	13.98	18.64	0.71	16.17	21.56	0.84
12	16.64	22.18	0.77	19.25	25.66	0.91
13	19.53	26.04	0.84	22.59	30.12	0.99
14	22.65	30.20	0.90	26.20	34.93	1.07
15	26.00	34.66	0.97	30.07	40.10	1.14
16	29.58	39.44	1.03	34.22	45.62	1.22
20	46.22	61.62	1.29	53.46	71.28	1.52
24	66.55	88.74	1.55	76.99	102.6	1.83
28	90.59	120.8	1.80	104.8	139.7	2.13
32	118.3	157.8	2.06	136.9	182.5	2.43
36	149.8	199.7	2.32	173.2	231.0	2.74
40	184.9	246.5	2.58	213.8	285.1	3.04
44	223.7	298.3	2.83	258.8	345.0	3.35
48	266.2	355.0	3.09	307.9	410.6	3.65
52	312.4	416.6	3.35	361.4	481.9	3.96
56	362.4	483.1	3.61	419.1	558.9	4.26
60	416.0	554.6	3.86	481.2	641.6	4.57
64	473.3	631.0	4.12	547.4	729.9	4.87
68	534.3	712.4	4.38	618.0	824.0	5.17

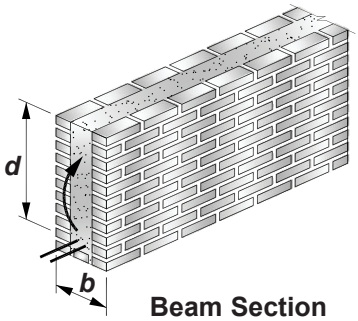
Table ASD-39 Moment Capacity of Walls and Beams for Balanced Design Conditions for f'_m = 3000 psi and f_y = 60,000 psi

$$M = \frac{K_f b d^2}{12{,}000} \text{ (ft kips)}$$

Tables based on b = 12" with d in inches and K_{fb} in psi



Wall Section



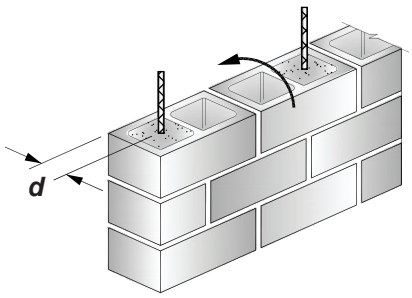
Beam Section

Effective Depth to Reinf. Steel, d, (inches)	Concrete Masonry			Clay Masonry		
	f_b = 1000 psi f_s = 24,000 psi K_{fb} = 138.7	$4f_b/3$ = 1333 psi $4f_s/3$ = 32,000 psi K_{fb} = 184.9	ρ_b = 0.006441	f_b = 1000 psi f_s = 24,000 psi K_{fb} = 160.4	$4f_b/3$ = 1333 psi $4f_s/3$ = 32,000 psi K_{fb} = 213.8	ρ_b = 0.007609
	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)
2.75	1.05	1.40	0.21	1.21	1.62	0.25
3	1.25	1.66	0.23	1.44	1.92	0.27
3.5	1.70	2.26	0.27	1.96	2.62	0.32
4	2.22	2.96	0.31	2.57	3.42	0.37
4.5	2.81	3.74	0.35	3.25	4.33	0.41
5	3.47	4.62	0.39	4.01	5.35	0.46
5.5	4.19	5.59	0.43	4.85	6.47	0.50
6	4.99	6.66	0.46	5.77	7.70	0.55
6.5	5.86	7.81	0.50	6.78	9.04	0.59
7	6.79	9.06	0.54	7.86	10.48	0.64
7.5	7.80	10.40	0.58	9.02	12.03	0.68
8	8.87	11.83	0.62	10.26	13.69	0.73
9	11.23	14.97	0.70	12.99	17.32	0.82
10	13.87	18.49	0.77	16.04	21.38	0.91
11	16.78	22.37	0.85	19.41	25.88	1.00
12	19.97	26.62	0.93	23.10	30.79	1.10
13	23.43	31.24	1.00	27.11	36.14	1.19
14	27.18	36.23	1.08	31.44	41.91	1.28
15	31.20	41.60	1.16	36.09	48.12	1.37
16	35.50	47.33	1.24	41.06	54.75	1.46
20	55.46	73.95	1.55	64.15	85.54	1.83
24	79.86	106.5	1.86	92.38	123.2	2.19
28	108.7	144.9	2.16	125.7	167.7	2.56
32	142.0	189.3	2.47	164.2	219.0	2.92
36	179.7	239.6	2.78	207.9	277.2	3.29
40	221.8	295.8	3.09	256.6	342.2	3.65
44	268.4	357.9	3.40	310.5	414.0	4.02
48	319.5	425.9	3.71	369.5	492.7	4.38
52	374.9	499.9	4.02	433.7	578.2	4.75
56	434.8	579.8	4.33	503.0	670.6	5.11
60	499.2	665.5	4.64	577.4	769.8	5.48
64	567.9	757.2	4.95	656.9	875.9	5.84
68	641.1	854.8	5.26	741.6	988.8	6.21

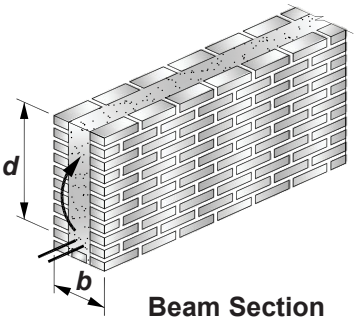
Table ASD-40 Moment Capacity of Walls and Beams for Balanced Design Conditions for f'_m = 3500 psi and f_y = 60,000 psi

$$M = \frac{K_f b d^2}{12{,}000} \text{ (ft kips)}$$

Tables based on b = 12" with d in inches and K_{fb} in psi



Wall Section



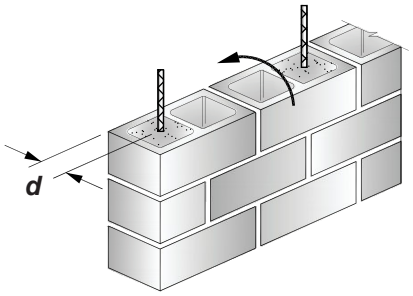
Beam Section

Effective Depth to Reinf. Steel, d, (inches)	Concrete Masonry			Clay Masonry		
	f_b = 1167 psi f_s = 24,000 psi K_{fb} = 161.8	$4f_b/3$ = 1556 psi $4f_s/3$ = 32,000 psi K_{fb} = 215.7	ρ_b = 0.007515	f_b = 1167 psi f_s = 24,000 psi K_{fb} = 187.1	$4f_b/3$ = 1556 psi $4f_s/3$ = 32,000 psi K_{fb} = 249.5	ρ_b = 0.008877
	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)
2.75	1.22	1.63	0.25	1.42	1.89	0.29
3	1.46	1.94	0.27	1.68	2.25	0.32
3.5	1.98	2.64	0.32	2.29	3.06	0.37
4	2.59	3.45	0.36	2.99	3.99	0.43
4.5	3.28	4.37	0.41	3.79	5.05	0.48
5	4.04	5.39	0.45	4.68	6.24	0.53
5.5	4.89	6.52	0.50	5.66	7.55	0.59
6	5.82	7.76	0.54	6.74	8.98	0.64
6.5	6.83	9.11	0.59	7.91	10.54	0.69
7	7.93	10.57	0.63	9.17	12.23	0.75
7.5	9.10	12.13	0.68	10.53	14.03	0.80
8	10.35	13.80	0.72	11.98	15.97	0.85
9	13.10	17.47	0.81	15.16	20.21	0.96
10	16.18	21.57	0.90	18.71	24.95	1.07
11	19.57	26.10	0.99	22.64	30.19	1.17
12	23.29	31.06	1.08	26.94	35.93	1.28
13	27.34	36.45	1.17	31.62	42.16	1.38
14	31.71	42.27	1.26	36.68	48.90	1.49
15	36.40	48.53	1.35	42.10	56.14	1.60
16	41.41	55.21	1.44	47.90	63.87	1.70
20	64.70	86.27	1.80	74.85	99.80	2.13
24	93.18	124.2	2.16	107.8	143.7	2.56
28	126.8	169.1	2.52	146.7	195.6	2.98
32	165.6	220.9	2.89	191.6	255.5	3.41
36	209.6	279.5	3.25	242.5	323.3	3.84
40	258.8	345.1	3.61	299.4	399.2	4.26
44	313.2	417.6	3.97	362.3	483.0	4.69
48	372.7	496.9	4.33	431.1	574.8	5.11
52	437.4	583.2	4.69	506.0	674.6	5.54
56	507.3	676.4	5.05	586.8	782.4	5.97
60	582.3	776.5	5.41	673.6	898.2	6.39
64	662.6	883.4	5.77	766.4	1022	6.82
68	748.0	997.3	6.13	865.2	1154	7.24

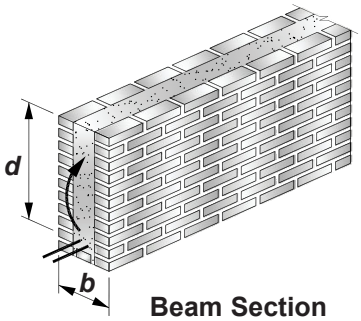
Table ASD-41 Moment Capacity of Walls and Beams for Balanced Design Conditions for f'_m = 4000 psi and f_y = 60,000 psi

$$M = \frac{K_f b d^2}{12{,}000} \text{ (ft kips)}$$

Tables based on b = 12" with d in inches and K_{fb} in psi



Wall Section



Beam Section

Effective Depth to Reinf. Steel, d, (inches)	Concrete Masonry			Clay Masonry		
	f_b = 1333 psi f_s = 24,000 psi K_{fb} = 184.9	$4f_b/3$ = 1778 psi $4f_s/3$ = 32,000 psi K_{fb} = 246.5	ρ_b = 0.008588	f_b = 1333 psi f_s = 24,000 psi K_{fb} = 213.8	$4f_b/3$ = 1778 psi $4f_s/3$ = 32,000 psi K_{fb} = 285.1	ρ_b = 0.01015
	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)	Moment (ft kips)	Moment (ft kips)	A_s (sq. in.)
2.75	1.40	1.86	0.28	1.62	2.16	0.33
3	1.66	2.22	0.31	1.92	2.57	0.37
3.5	2.26	3.02	0.36	2.62	3.49	0.43
4	2.96	3.94	0.41	3.42	4.56	0.49
4.5	3.74	4.99	0.46	4.33	5.77	0.55
5	4.62	6.16	0.52	5.35	7.13	0.61
5.5	5.59	7.46	0.57	6.47	8.63	0.67
6	6.66	8.87	0.62	7.70	10.26	0.73
6.5	7.81	10.41	0.67	9.04	12.05	0.79
7	9.06	12.08	0.72	10.48	13.97	0.85
7.5	10.40	13.87	0.77	12.03	16.04	0.91
8	11.83	15.78	0.82	13.69	18.25	0.97
9	14.97	19.97	0.93	17.32	23.10	1.10
10	18.49	24.65	1.03	21.38	28.51	1.22
11	22.37	29.83	1.13	25.88	34.50	1.34
12	26.62	35.50	1.24	30.79	41.06	1.46
13	31.24	41.66	1.34	36.14	48.19	1.58
14	36.23	48.31	1.44	41.91	55.89	1.70
15	41.60	55.46	1.55	48.12	64.15	1.83
16	47.33	63.10	1.65	54.75	72.99	1.95
20	73.95	98.60	2.06	85.54	114.0	2.43
24	106.5	142.0	2.47	123.2	164.2	2.92
28	144.9	193.2	2.89	167.7	223.5	3.41
32	189.3	252.4	3.30	219.0	292.0	3.90
36	239.6	319.5	3.71	277.2	369.5	4.38
40	295.8	394.4	4.12	342.2	456.2	4.87
44	357.9	477.2	4.53	414.0	552.0	5.36
48	425.9	567.9	4.95	492.7	656.9	5.84
52	499.9	666.5	5.36	578.2	771.0	6.33
56	579.8	773.0	5.77	670.6	894.2	6.82
60	665.5	887.4	6.18	769.9	1026	7.30
64	757.2	1010	6.60	875.9	1168	7.79
68	854.8	1140	7.01	988.8	1318	8.28

Table ASD-46a Moment Capacity (ft k/ft) of Clay Masonry Walls with $A_s = 0.0007bt$ $b = 12$" and $F_s = 24{,}000$ psi

Wall Type	Nominal Thickness	Actual Thickness t (inches)	Effective Depth, d (inches)	f'_m	1500	2000	2500	3000	3500	4000	A_s (sq.in./ft)
				f_b	500	667	833	1000	1167	1333	
				E_m	1,050,000	1,400,000	1,750,000	2,100,000	2,450,000	2,800,000	
				n	27.6	20.7	16.6	13.8	11.8	10.4	
Hollow Clay Brick Units	6	5.5	2.7[1]	M_m	0.41	0.49	0.55	0.62	0.67	0.73	0.046
				M_s	0.23	0.23	0.23	0.23	0.24	0.24	
	8	7.5	3.7[1]	M_m	0.77	0.91	1.04	1.15	1.26	1.36	0.063
				M_s	0.43	0.43	0.44	0.44	0.44	0.44	
			5.2[2]	M_m	1.32	1.56	1.78	1.97	2.15	2.32	
				M_s	0.61	0.61	0.62	0.62	0.62	0.63	
Two Wythe Clay Brick Walls	9	9	4.5[1]	M_m	1.13	1.34	1.53	1.70	1.85	2.00	0.076
				M_s	0.63	0.63	0.64	0.64	0.64	0.64	
	10	10	5.0[1]	M_m	1.39	1.65	1.89	2.10	2.29	2.47	0.084
				M_s	0.77	0.78	0.79	0.79	0.79	0.80	
	12	12	6.0[1]	M_m	2.00	2.38	2.71	3.02	3.30	3.56	0.101
				M_s	1.11	1.12	1.13	1.14	1.14	1.15	
			7.5[2]	M_m	2.86	3.39	3.86	4.29	4.68	5.04	
				M_s	1.40	1.41	1.42	1.43	1.44	1.44	
	16	16	11.5[2]	M_m	6.36	7.52	8.56	9.49	10.35	11.15	0.134
				M_s	2.88	2.90	2.92	2.94	2.95	2.95	

1. Based on $d = t/2$.
2. Based on a 1 in. distance between the center of reinforcement and the inside of the face shell or wythe, therefore, $d = t$ - 1" - common face shell or wythe thickness.

Table ASD-46b Moment Capacity (ft k/ft) of Concrete Masonry Walls with $A_s = 0.0007bt$ b = 12" and F_s = 24,000 psi

Wall Type	Nominal Thickness	Actual Thickness t (inches)	Effective Depth, d (inches)		f'_m	1500	2000	2500	3000	3500	4000	A_s (sq.in./ft)
					f_b	500	667	833	1000	1167	1333	
					E_m	1,350,000	1,800,000	2,250,000	2,700,000	3,150,000	3,600,000	
					n	21.5	16.1	12.9	10.7	9.2	8.1	
Concrete Masonry Units	6	5.625	2.8[1]	M_m		0.40	0.47	0.54	0.60	0.65	0.70	0.047
				M_s		0.25	0.25	0.25	0.25	0.25	0.25	
	8	7.625	3.8[1]	M_m		0.73	0.87	0.99	1.09	1.19	1.29	0.064
				M_s		0.45	0.46	0.46	0.46	0.46	0.47	
			5.1[2]	M_m		1.16	1.37	1.56	1.73	1.88	2.02	
				M_s		0.61	0.62	0.62	0.62	0.63	0.63	
			5.3[2]	M_m		1.23	1.46	1.65	1.83	2.00	2.15	
				M_s		0.64	0.64	0.65	0.65	0.65	0.65	
	10	9.625	4.8[1]	M_m		1.17	1.38	1.57	1.74	1.90	2.05	0.081
				M_s		0.72	0.73	0.73	0.74	0.74	0.74	
			7.1[2]	M_m		2.16	2.55	2.89	3.20	3.49	3.76	
				M_s		1.08	1.09	1.09	1.10	1.10	1.10	
	12	11.625	5.8[1]	M_m		1.70	2.01	2.29	2.54	2.78	2.99	0.098
				M_s		1.05	1.06	1.07	1.08	1.08	1.08	
			9.1[2]	M_m		3.46	4.08	4.63	5.12	5.58	6.01	
				M_s		1.67	1.68	1.69	1.70	1.71	1.71	
Concrete Masonry Component (Expandable) Wall	10	10	5.0[1]	M_m		1.26	1.49	1.70	1.88	2.05	2.21	0.084
				M_s		0.78	0.79	0.79	0.80	0.80	0.80	
			6.7[2]	M_m		2.00	2.37	2.69	2.98	3.24	3.49	
				M_s		1.05	1.06	1.07	1.07	1.08	1.08	
	12	12	6.0[1]	M_m		1.81	2.15	2.44	2.71	2.96	3.19	0.101
				M_s		1.12	1.13	1.14	1.15	1.15	1.15	
			8.7[2]	M_m		3.27	3.86	4.37	4.85	5.28	5.68	
				M_s		1.65	1.66	1.67	1.68	1.68	1.69	
	16	16	12.7[2]	M_m		6.70	7.90	8.96	9.92	10.80	11.62	0.134
				M_s		3.21	3.24	3.25	3.27	3.28	3.29	

1. Based on $d = t/2$.
2. Based on a 1 in. distance between the center of reinforcement and the inside of the face shell or wythe, therefore, $d = t$ - 1" - common face shell or wythe thickness.

Table ASD-47a Moment Capacity (ft k/ft) of Clay Masonry Walls with $A_s = 0.0013bt$ $b = 12$" and $F_s = 24{,}000$ psi

Wall Type	Nominal Thickness	Actual Thickness t (inches)	Effective Depth, d (inches)	f'_m	1500	2000	2500	3000	3500	4000	A_s (sq.in./ft)
				f_b	500	667	833	1000	1167	1333	
				E_m	1,050,000	1,400,000	1,750,000	2,100,000	2,450,000	2,800,000	
				n	27.6	20.7	16.6	13.8	11.8	10.4	
Hollow Clay Brick Units	6	5.5	2.7[1]	M_m	0.52	0.62	0.71	0.79	0.87	0.94	0.086
				M_s	0.41	0.42	0.42	0.43	0.43	0.43	
	8	7.5	3.7[1]	M_m	0.97	1.16	1.33	1.49	1.63	1.77	0.117
				M_s	0.77	0.78	0.79	0.80	0.80	0.81	
			5.2[2]	M_m	1.68	2.01	2.30	2.56	2.80	3.03	
				M_s	1.11	1.12	1.13	1.13	1.14	1.14	
Two Wythe Clay Brick Walls	9	9	4.5[1]	M_m	1.42	1.71	1.96	2.19	2.40	2.60	0.140
				M_s	1.13	1.15	1.16	1.16	1.17	1.18	
	10	10	5.0[1]	M_m	1.76	2.11	2.42	2.70	2.96	3.21	0.156
				M_s	1.40	1.42	1.43	1.44	1.45	1.45	
	12	12	6.0[1]	M_m	2.53	3.03	3.48	3.89	4.27	4.62	0.187
				M_s	2.01	2.04	2.06	2.07	2.08	2.09	
			7.5[2]	M_m	3.64	4.35	4.98	5.56	6.09	6.58	
				M_s	2.54	2.57	2.59	2.61	2.62	2.63	
	16	16	11.5[2]	M_m	8.12	9.69	11.08	12.34	13.50	14.59	0.250
				M_s	5.22	5.28	5.33	5.36	5.38	5.40	

1. Based on $d = t/2$.
2. Based on a 1 in. distance between the center of reinforcement and the inside of the face shell or wythe, therefore, $d = t$ - 1" - common face shell or wythe thickness.

Table ASD-47b Moment Capacity (ft k/ft) of Concrete Masonry Walls with $A_s = 0.0013bt$ $b = 12$" and $F_s = 24{,}000$ psi

Wall Type	Nominal Thickness	Actual Thickness t (inches)	Effective Depth, d (inches)	f'_m	1500	2000	2500	3000	3500	4000	A_s (sq.in./ft)
				f_b	500	667	833	1000	1167	1333	
				E_m	1,350,000	1,800,000	2,250,000	2,700,000	3,150,000	3,600,000	
				n	21.5	16.1	12.9	10.7	9.2	8.1	
Concrete Masonry Units	6	5.625	2.8[1]	M_m	0.51	0.61	0.69	0.77	0.85	0.91	0.088
				M_s	0.45	0.45	0.46	0.46	0.46	0.46	
	8	7.625	3.8[1]	M_m	0.93	1.11	1.27	1.42	1.55	1.68	0.119
				M_s	0.82	0.83	0.84	0.84	0.85	0.85	
			5.1[2]	M_m	1.49	1.77	2.02	2.25	2.46	2.66	
				M_s	1.11	1.12	1.13	1.14	1.14	1.15	
			5.3[2]	M_m	1.59	1.89	2.15	2.39	2.62	2.82	
				M_s	1.16	1.17	1.18	1.18	1.19	1.19	
	10	9.625	4.8[1]	M_m	1.48	1.77	2.03	2.26	2.48	2.68	0.150
				M_s	1.31	1.32	1.34	1.34	1.35	1.36	
			7.1[2]	M_m	2.78	3.30	3.77	4.19	4.58	4.94	
				M_s	1.96	1.98	2.00	2.01	2.02	2.02	
	12	11.625	5.8[1]	M_m	2.16	2.59	2.96	3.30	3.61	3.91	0.181
				M_s	1.91	1.93	1.95	1.96	1.97	1.98	
			9.1[2]	M_m	4.46	5.30	6.04	6.71	7.33	7.90	
				M_s	3.04	3.07	3.10	3.11	3.12	3.13	
Concrete Masonry Component (Expandable) Wall	10	10	5.0[1]	M_m	1.60	1.91	2.19	2.44	2.67	2.89	0.156
				M_s	1.41	1.43	1.44	1.45	1.46	1.46	
			6.7[2]	M_m	2.57	3.06	3.49	3.88	4.25	4.58	
				M_s	1.92	1.94	1.95	1.96	1.97	1.98	
	12	12	6.0[1]	M_m	2.31	2.76	3.15	3.52	3.85	4.16	0.187
				M_s	2.03	2.06	2.08	2.09	2.10	2.11	
			8.7[2]	M_m	4.21	5.00	5.70	6.33	6.92	7.47	
				M_s	2.99	3.03	3.05	3.06	3.08	3.09	
	16	16	12.7[2]	M_m	8.65	10.26	11.69	12.99	14.19	15.30	0.250
				M_s	5.85	5.91	5.95	5.98	6.00	6.02	

1. Based on $d = t/2$.
2. Based on a 1" distance between the center of reinforcement and the inside of the face shell or wythe, therefore, $d = t$ - 1" - common face shell or wythe thickness.

Table ASD-48a Moment Capacity (ft k/ft) of Clay Masonry Walls with $A_s = 0.001bt$ b = 12" and F_s = 24,000 psi

Wall Type	Nominal Thickness	Actual Thickness t (inches)	Effective Depth, d (inches)	f'_m	1500	2000	2500	3000	3500	4000	A_s (sq.in./ft)
				f_b	500	667	833	1000	1167	1333	
				E_m	1,050,000	1,400,000	1,750,000	2,100,000	2,450,000	2,800,000	
				n	27.6	20.7	16.6	13.8	11.8	10.4	
Hollow Clay Brick Units	6	5.5	2.7[1]	M_m	0.47	0.56	0.64	0.71	0.78	0.85	0.066
				M_s	0.32	0.33	0.33	0.33	0.33	0.33	
	8	7.5	3.7[1]	M_m	0.88	1.05	1.20	1.34	1.47	1.58	0.090
				M_s	0.60	0.61	0.62	0.62	0.62	0.62	
			5.2[2]	M_m	1.52	1.81	2.06	2.30	2.51	2.71	
				M_s	0.86	0.87	0.87	0.88	0.88	0.89	
Two Wythe Clay Brick Walls	9	9	4.5[1]	M_m	1.29	1.54	1.77	1.97	2.16	2.33	0.108
				M_s	0.88	0.89	0.90	0.90	0.91	0.91	
	10	10	5.0[1]	M_m	1.60	1.91	2.18	2.43	2.66	2.88	0.120
				M_s	1.09	1.10	1.11	1.12	1.12	1.13	
	12	12	6.0[1]	M_m	2.30	2.74	3.14	3.50	3.83	4.14	0.144
				M_s	1.57	1.58	1.60	1.61	1.62	1.62	
			7.5[2]	Mm	3.30	3.93	4.48	4.99	5.45	5.89	
				M_s	1.98	2.00	2.01	2.02	2.03	2.04	
	16	16	11.5[2]	M_m	7.34	8.72	9.95	11.06	12.08	13.04	0.192
				M_s	4.06	4.10	4.13	4.15	4.17	4.19	

1. Based on $d = t/2$.
2. Based on a 1 in. distance between the center of reinforcement and the inside of the face shell or wythe, therefore, $d = t$ - 1" - common face shell or wythe thickness.

Table ASD-48b Moment Capacity (ft k/ft) of Concrete Masonry Walls with $A_s = 0.001bt$ b = 12" and F_s = 24,000 psi

Wall Type	Nominal Thickness	Actual Thickness t (inches)	Effective Depth, d (inches)	f'_m / f_b / E_m / n	1500 / 500 / 1,350,000 / 21.5	2000 / 667 / 1,800,000 / 16.1	2500 / 833 / 2,250,000 / 12.9	3000 / 1000 / 2,700,000 / 10.7	3500 / 1167 / 3,150,000 / 9.2	4000 / 1333 / 3,600,000 / 8.1	A_s (sq.in./ft)
Concrete Masonry Units	6	5.625	2.8[1]	M_m	0.46	0.55	0.62	0.69	0.76	0.82	0.068
				M_s	0.35	0.35	0.35	0.36	0.36	0.36	
	8	7.625	3.8[1]	M_m	0.84	1.00	1.14	1.27	1.39	1.50	0.092
				M_s	0.64	0.65	0.65	0.65	0.66	0.66	
			5.1[2]	M_m	1.34	1.59	1.81	2.01	2.20	2.37	
				M_s	0.86	0.87	0.88	0.88	0.89	0.89	
			5.3[2]	M_m	1.43	1.69	1.93	2.14	2.34	2.52	
				M_s	0.90	0.91	0.91	0.92	0.92	0.92	
	10	9.625	4.8[1]	M_m	1.34	1.60	1.82	2.03	2.22	2.40	0.116
				M_s	1.02	1.03	1.04	1.04	1.05	1.05	
			7.1[2]	M_m	2.50	2.97	3.37	3.74	4.08	4.40	
				M_s	1.52	1.54	1.55	1.55	1.56	1.57	
	12	11.625	5.8[1]	M_m	1.96	2.33	2.66	2.96	3.24	3.49	0.140
				M_s	1.48	1.50	1.51	1.52	1.53	1.53	
			9.1[2]	M_m	4.02	4.75	5.40	5.99	6.54	7.04	
				M_s	2.36	2.38	2.40	2.41	2.42	2.43	
Concrete Masonry Component (Expandable) Wall	10	10	5.0[1]	M_m	1.45	1.72	1.97	2.19	2.40	2.59	0.120
				M_s	1.10	1.11	1.12	1.13	1.13	1.13	
			6.7[2]	M_m	2.32	2.75	3.13	3.47	3.79	4.09	
				M_s	1.49	1.50	1.51	1.52	1.53	1.53	
	12	12	6.0[1]	M_m	2.09	2.48	2.84	3.15	3.45	3.72	0.144
				M_s	1.58	1.60	1.61	1.62	1.63	1.63	
			8.7[2]	M_m	3.79	4.48	5.10	5.66	6.18	6.66	
				M_s	2.33	2.35	2.36	2.37	2.38	2.39	
	16	16	12.7[2]	M_m	7.78	9.20	10.46	11.60	12.66	13.64	0.192
				M_s	4.54	4.58	4.61	4.63	4.65	4.66	

1. Based on $d = t/2$.
2. Based on a 1 in. distance between the center of reinforcement and the inside of the face shell or wythe, therefore, $d = t$ - 1" - common face shell or wythe thickness.

Table ASD-56 Allowable Shear Stress Capacity[1,2,3] (psi) for Nominal 6" Wide Sections Reinforcing Steel Designed to Carry Entire Shear Force with F_s = 24,000 psi

$A_v = \frac{Vs}{F_s d}$ **(MSJC Code Eq 2 - 26)** Since $F_v = \frac{V}{bd}$; $F_v = \frac{A_v F_s}{bs}$

or rewritten, $V = \frac{A_v F_s d}{s}$ **where :** b = **5.625",** F_s = **24,000 psi**

Spacing of Shear Reinforcing Bars (inches)	Allowable Shear Stress, F_v (psi)[1,2]						
	Shear Reinforcing Bar Size and Area (square inches)						
	#3	#4	#5	#6	#7	#8	#9
	0.11	0.20	0.31	0.44	0.60	0.79	1.00
8	59	107	150	150	150	150	150
12	39	71	110	150	150	150	150
16	29	53	83	117	150	150	150
20	23	43	66	94	128	150	150
24	20	36	55	78	107	140	150
28	17	30	47	67	91	120	150
32	15	27	41	59	80	105	133
36	13	24	37	52	71	94	119
40	12	21	33	47	64	84	107
48	10	18	28	39	53	70	89

1. For flexural members, F_v may not exceed 150 psi nor $3\sqrt{f'_m}$
2. F_v may be limited to lower values for shear walls. See Table ASD-6 for specific values for shear walls.
3. Table values may be incresed by one-third when considering wind or earthquake forces (MSJC Code Section 2.1.2.3).

Diagram ASD-56 Spacing of Shear Reinforcement for Nominal 6" Wide Sections (Dashed = Allowable Increase for Wind)

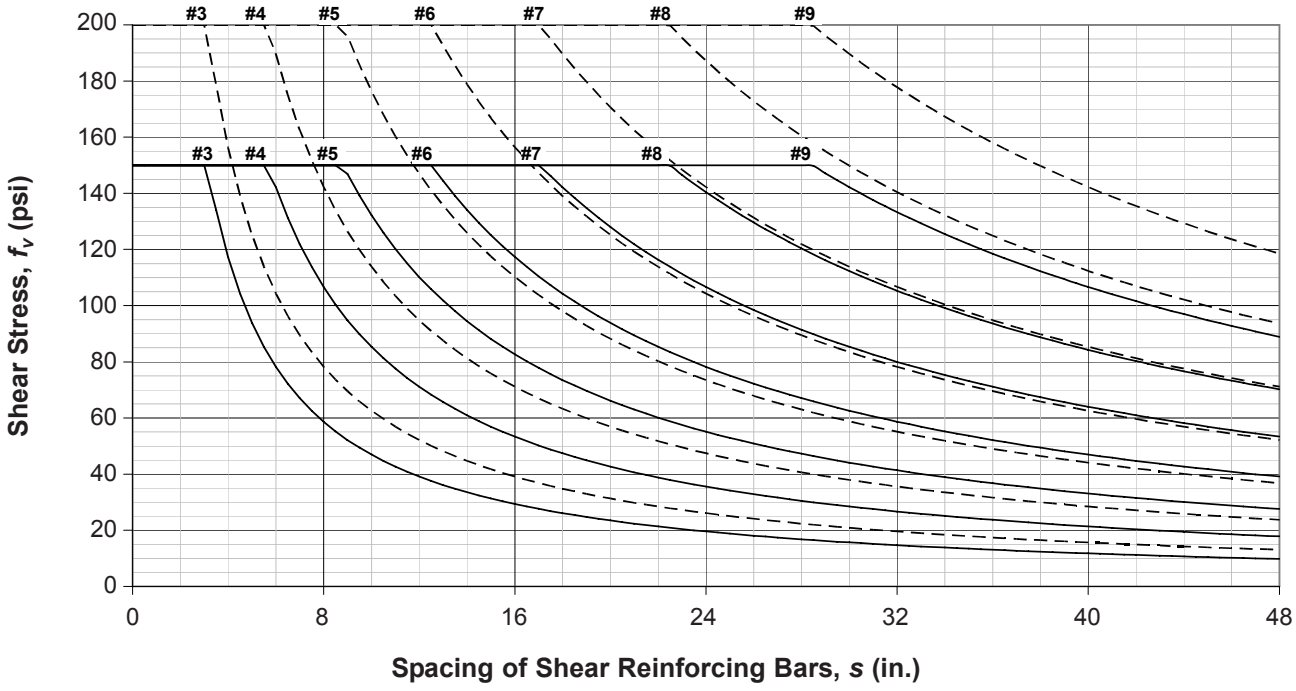


Table ASD-58 Allowable Shear Stress Capacity[1,2,3] (psi) for Nominal 8" Wide Sections Reinforcing Steel Designed to Carry Entire Shear Force with F_s = 24,000 psi

$A_v = \frac{Vs}{F_s d}$ **(MSJC Code Eq 2 - 26)** **Since** $F_v = \frac{V}{bd}$; $F_v = \frac{A_v F_s}{bs}$

or rewritten, $V = \frac{A_v F_s d}{s}$ **where :** $b = 7.625"$, $F_s = 24{,}000$ **psi**

Spacing of Shear Reinforcing Bars (inches)	Allowable Shear Stress, F_v (psi)[1,2]						
	Shear Reinforcing Bar Size and Area (square inches)						
	#3	#4	#5	#6	#7	#8	#9
	0.11	0.20	0.31	0.44	0.60	0.79	1.00
8	43	79	122	150	150	150	150
12	29	52	81	115	150	150	150
16	22	39	61	87	118	150	150
20	17	31	49	69	94	124	150
24	14	26	41	58	79	104	131
28	12	22	35	49	67	89	112
32	11	20	30	43	59	78	98
36	10	17	27	38	52	69	87
40	9	16	24	35	47	62	79
48	7	13	20	29	39	52	66

1. For flexural members, F_v may not exceed 150 psi nor $3\sqrt{f'_m}$
2. F_v may be limited to lower values for shear walls. See Table ASD-6 for specific values for shear walls.
3. Table values may be incresed by one-third when considering wind or earthquake forces (MSJC Code Section 2.1.2.3).

Diagram ASD-58 Spacing of Shear Reinforcement for Nominal 8" Wide Sections (Dashed = Allowable Increase for Wind)

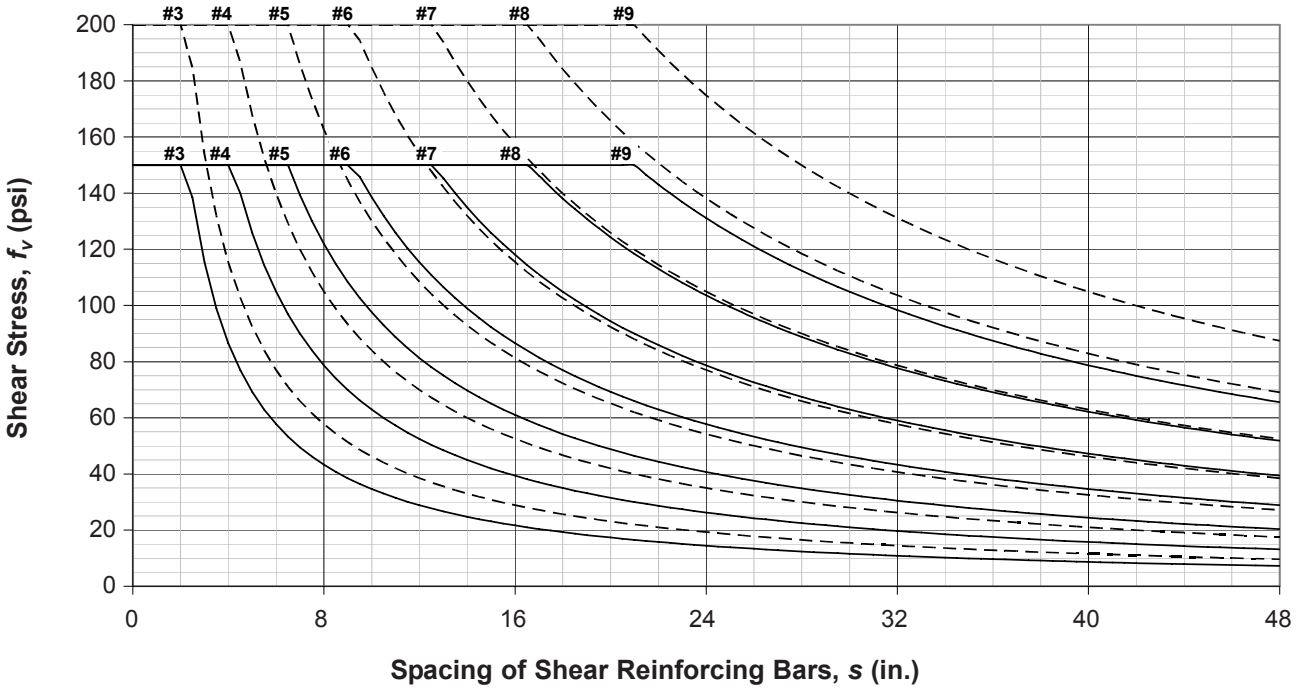


Table ASD-60 Allowable Shear Stress Capacity[1,2,3] (psi) for Nominal 10" Wide Sections Reinforcing Steel Designed to Carry Entire Shear Force with F_s = 24,000 psi

$A_v = \frac{Vs}{F_s d}$ **(MSJC Code Eq 2 - 26)** Since $F_v = \frac{V}{bd}$; $F_v = \frac{A_v F_s}{bs}$

or rewritten, $V = \frac{A_v F_s d}{s}$ where : b = 9.625", F_s = 24,000 psi

Spacing of Shear Reinforcing Bars (inches)	Allowable Shear Stress, F_v (psi)[1,2]						
	Shear Reinforcing Bar Size and Area (square inches)						
	#3	#4	#5	#6	#7	#8	#9
	0.11	0.20	0.31	0.44	0.60	0.79	1.00
8	34	62	97	137	150	150	150
12	23	42	64	91	125	150	150
16	17	31	48	69	94	123	150
20	14	25	39	55	75	98	125
24	11	21	32	46	62	82	104
28	10	18	28	39	53	70	89
32	9	16	24	34	47	62	78
36	8	14	21	30	42	55	69
40	7	12	19	27	37	49	62
48	6	10	16	23	31	41	52

1. For flexural members, F_v may not exceed 150 psi nor $3\sqrt{f'_m}$
2. F_v may be limited to lower values for shear walls. See Table ASD-6 for specific values for shear walls.
3. Table values may be incresed by one-third when considering wind or earthquake forces (MSJC Code Section 2.1.2.3).

Diagram ASD-60 Spacing of Shear Reinforcement for Nominal 10" Wide Sections (Dashed = Allowable Increase for Wind)

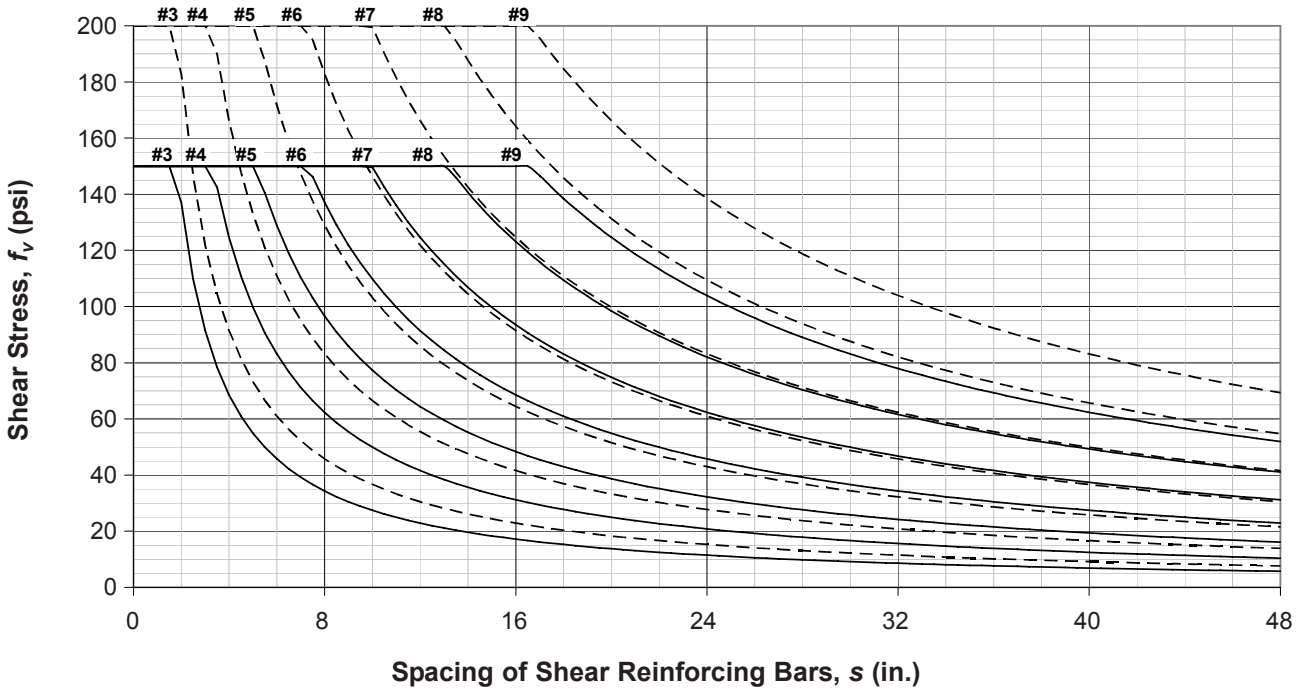


Table ASD-62 Allowable Shear Stress Capacity[1,2,3] (psi) for Nominal 12" Wide Sections Reinforcing Steel Designed to Carry Entire Shear Force with F_s = 24,000 psi

$A_v = \frac{Vs}{F_s d}$ **(MSJC Code Eq 2 - 26)** **Since** $F_v = \frac{V}{bd}$; $F_v = \frac{A_v F_s}{bs}$

or rewritten, $V = \frac{A_v F_s d}{s}$ **where :** $b = 11.625"$, $F_s = 24{,}000$ **psi**

Spacing of Shear Reinforcing Bars (inches)	Allowable Shear Stress, F_v (psi)[1,2]						
	Shear Reinforcing Bar Size and Area (square inches)						
	#3	#4	#5	#6	#7	#8	#9
	0.11	0.20	0.31	0.44	0.60	0.79	1.00
8	28	52	80	114	150	150	150
12	19	34	53	76	103	136	150
16	14	26	40	57	77	102	129
20	11	21	32	45	62	82	103
24	9	17	27	38	52	68	86
28	8	15	23	32	44	58	74
32	7	13	20	28	39	51	65
36	6	11	18	25	34	45	57
40	6	10	16	23	31	41	52
48	5	9	13	19	26	34	43

1. For flexural members, F_v may not exceed 150 psi nor $3\sqrt{f'_m}$
2. F_v may be limited to lower values for shear walls. See Table ASD-6 for specific values for shear walls.
3. Table values may be incresed by one-third when considering wind or earthquake forces (MSJC Code Section 2.1.2.3).

Diagram ASD-62 Spacing of Shear Reinforcement for Nominal 12" Wide Sections (Dashed = Allowable Increase for Wind)

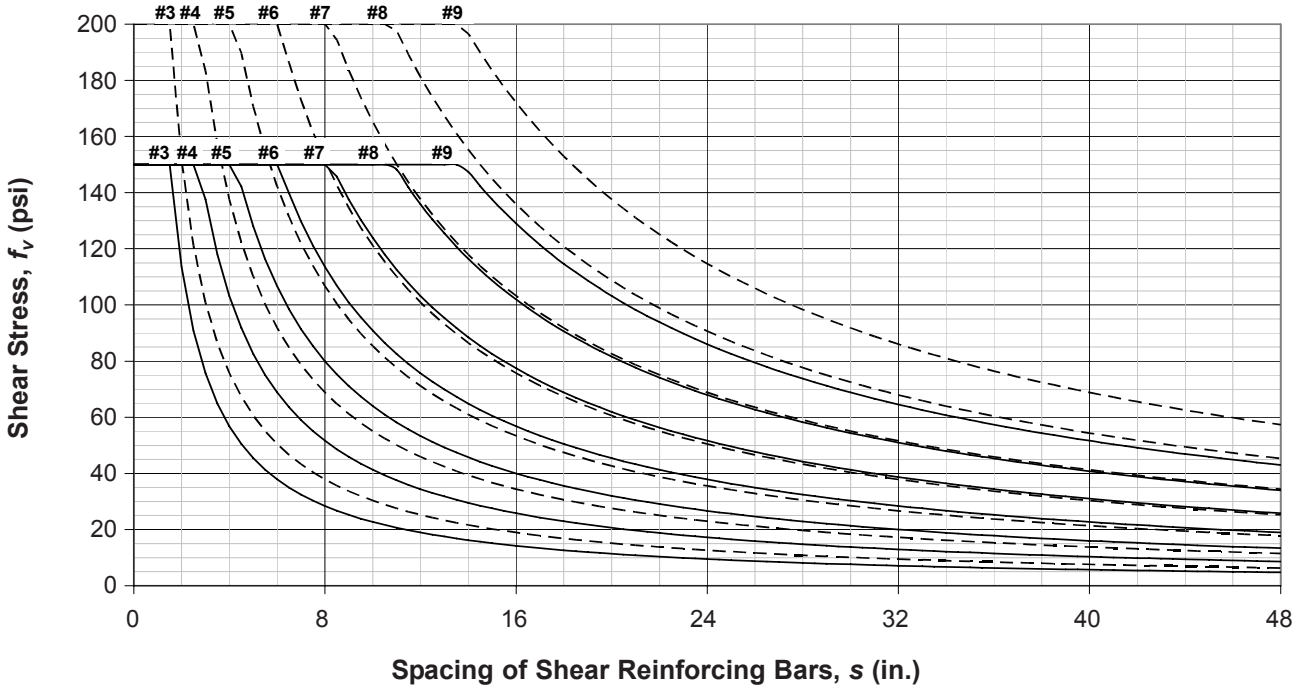
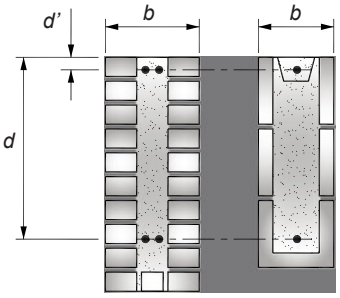


Table ASD-74a Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Clay Masonry) f'_m = 1500 psi, F_s = 24,000 psi, and n = 27.6

DESIGN DATA

f'_m = 1500 psi
f_b = 500 psi
E_m = 1,050,000 psi
n = 27.6
K_{fb} = 80.2
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi
k = 0.365
ρ_b = 0.0038



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		80.2	90	95	100	105	110	115	120	125	130
0.02	ρ'	—	0.0002	0.0003	0.0004	0.0005	0.0006	0.0007	0.0008	0.0009	0.0010
	ρ	0.0038	0.0042	0.0044	0.0046	0.0049	0.0051	0.0053	0.0055	0.0057	0.0059
0.04	ρ'	—	0.0002	0.0003	0.0004	0.0005	0.0006	0.0008	0.0009	0.0010	0.0011
	ρ	0.0038	0.0042	0.0044	0.0047	0.0049	0.0051	0.0053	0.0055	0.0057	0.0060
0.06	ρ'	—	0.0002	0.0003	0.0005	0.0006	0.0007	0.0008	0.0009	0.0011	0.0012
	ρ	0.0038	0.0042	0.0045	0.0047	0.0049	0.0051	0.0053	0.0056	0.0058	0.0060
0.08	ρ'	—	0.0003	0.0004	0.0005	0.0006	0.0008	0.0009	0.0010	0.0011	0.0013
	ρ	0.0038	0.0042	0.0045	0.0047	0.0049	0.0051	0.0054	0.0056	0.0058	0.0061
0.10	ρ'	—	0.0003	0.0004	0.0006	0.0007	0.0008	0.0010	0.0011	0.0013	0.0014
	ρ	0.0038	0.0043	0.0045	0.0047	0.0049	0.0052	0.0054	0.0056	0.0059	0.0061
0.12	ρ'	—	0.0003	0.0005	0.0006	0.0008	0.0009	0.0011	0.0012	0.0014	0.0016
	ρ	0.0038	0.0043	0.0045	0.0047	0.0050	0.0052	0.0054	0.0057	0.0059	0.0062
0.14	ρ'	—	0.0003	0.0005	0.0007	0.0009	0.0010	0.0012	0.0014	0.0016	0.0017
	ρ	0.0038	0.0043	0.0045	0.0048	0.0050	0.0052	0.0055	0.0057	0.0060	0.0062
0.16	ρ'	—	0.0004	0.0006	0.0008	0.0010	0.0012	0.0014	0.0016	0.0017	0.0019
	ρ	0.0038	0.0043	0.0045	0.0048	0.0050	0.0053	0.0055	0.0058	0.0060	0.0063
0.18	ρ'	—	0.0004	0.0007	0.0009	0.0011	0.0013	0.0015	0.0018	0.0020	0.0022
	ρ	0.0038	0.0043	0.0046	0.0048	0.0051	0.0053	0.0056	0.0058	0.0061	0.0063
0.20	ρ'	—	0.0005	0.0008	0.0010	0.0013	0.0015	0.0018	0.0020	0.0023	0.0025
	ρ	0.0038	0.0043	0.0046	0.0048	0.0051	0.0054	0.0056	0.0059	0.0061	0.0064
0.22	ρ'	—	0.0006	0.0009	0.0012	0.0015	0.0018	0.0021	0.0024	0.0027	0.0030
	ρ	0.0038	0.0043	0.0046	0.0049	0.0051	0.0054	0.0057	0.0059	0.0062	0.0065
0.24	ρ'	—	0.0007	0.0010	0.0014	0.0018	0.0021	0.0025	0.0028	0.0032	0.0035
	ρ	0.0038	0.0043	0.0046	0.0049	0.0052	0.0054	0.0057	0.0060	0.0063	0.0065
0.26	ρ'	—	0.0008	0.0013	0.0017	0.0021	0.0026	0.0030	0.0034	0.0039	0.0043
	ρ	0.0038	0.0044	0.0046	0.0049	0.0052	0.0055	0.0058	0.0060	0.0063	0.0066
0.28	ρ'	—	0.0011	0.0016	0.0022	0.0027	0.0033	0.0038	0.0044	0.0049	0.0055
	ρ	0.0038	0.0044	0.0047	0.0049	0.0052	0.0055	0.0058	0.0061	0.0064	0.0067
0.30	ρ'	—	0.0014	0.0022	0.0029	0.0037	0.0044	0.0051	0.0059	0.0066	0.0073
	ρ	0.0038	0.0044	0.0047	0.0050	0.0053	0.0056	0.0059	0.0062	0.0065	0.0068

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-74a Steel Ratio ρ and ρ' Versus K_f for f'_m = 1,500 psi, (Clay Masonry)

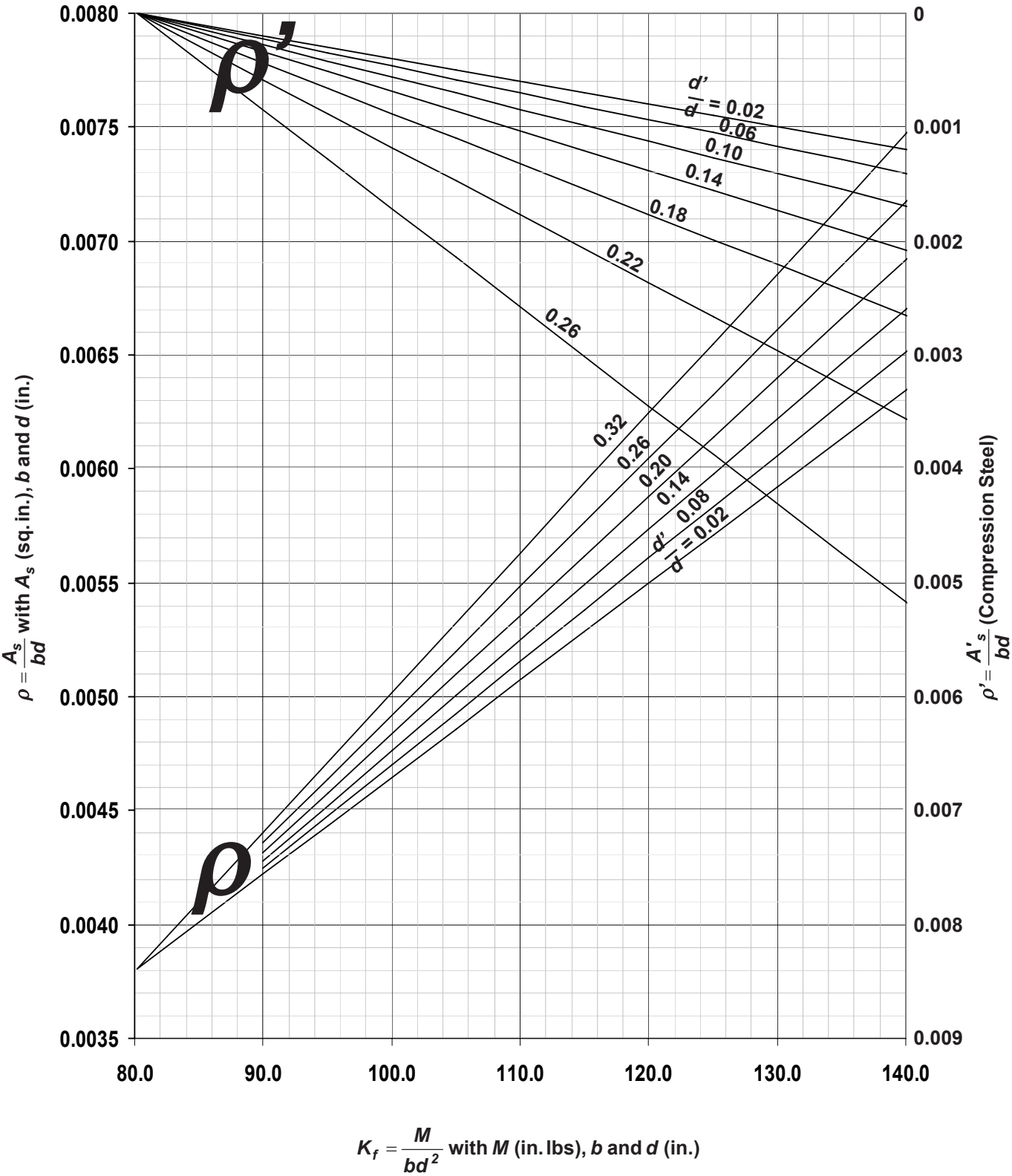
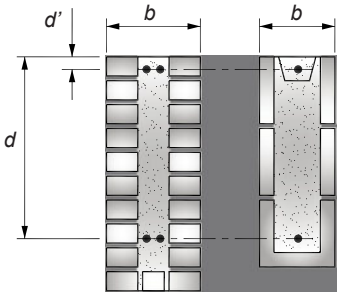


Table ASD-74b Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Concrete Masonry) f'_m = 1500 psi, F_s = 24,000 psi, and n = 21.5

DESIGN DATA

f'_m = 1500 psi
f_y = 60,000 psi
f_b = 500 psi
F_s = 24,000 psi
E_m = 1,350,000 psi
E_s = 29,000,000 psi
n = 21.5
k = 0.309
K_{fb} = 69.3
ρ_b = 0.0032



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\,(\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\,(\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		69.3	75	80	85	90	95	100	105	110	115
0.02	ρ'	—	0.0001	0.0003	0.0004	0.0005	0.0007	0.0008	0.0009	0.0011	0.0012
	ρ	0.0032	0.0035	0.0037	0.0039	0.0041	0.0043	0.0045	0.0047	0.0049	0.0052
0.04	ρ'	—	0.0002	0.0003	0.0004	0.0006	0.0007	0.0009	0.0010	0.0012	0.0013
	ρ	0.0032	0.0035	0.0037	0.0039	0.0041	0.0043	0.0046	0.0048	0.0050	0.0052
0.06	ρ'	—	0.0002	0.0003	0.0005	0.0007	0.0008	0.0010	0.0011	0.0013	0.0014
	ρ	0.0032	0.0035	0.0037	0.0039	0.0041	0.0044	0.0046	0.0048	0.0050	0.0052
0.08	ρ'	—	0.0002	0.0004	0.0005	0.0007	0.0009	0.0011	0.0012	0.0014	0.0016
	ρ	0.0032	0.0035	0.0037	0.0039	0.0042	0.0044	0.0046	0.0048	0.0051	0.0053
0.10	ρ'	—	0.0002	0.0004	0.0006	0.0008	0.0010	0.0012	0.0014	0.0016	0.0018
	ρ	0.0032	0.0035	0.0037	0.0039	0.0042	0.0044	0.0046	0.0049	0.0051	0.0053
0.12	ρ'	—	0.0003	0.0005	0.0007	0.0009	0.0011	0.0014	0.0016	0.0018	0.0020
	ρ	0.0032	0.0035	0.0037	0.0040	0.0042	0.0044	0.0047	0.0049	0.0051	0.0054
0.14	ρ'	—	0.0003	0.0005	0.0008	0.0010	0.0013	0.0016	0.0018	0.0021	0.0023
	ρ	0.0032	0.0035	0.0037	0.0040	0.0042	0.0045	0.0047	0.0049	0.0052	0.0054
0.16	ρ'	—	0.0003	0.0006	0.0009	0.0012	0.0015	0.0018	0.0021	0.0024	0.0027
	ρ	0.0032	0.0035	0.0037	0.0040	0.0042	0.0045	0.0047	0.0050	0.0052	0.0055
0.18	ρ'	—	0.0004	0.0007	0.0011	0.0014	0.0018	0.0021	0.0025	0.0028	0.0032
	ρ	0.0032	0.0035	0.0038	0.0040	0.0043	0.0045	0.0048	0.0050	0.0053	0.0055
0.20	ρ'	—	0.0005	0.0009	0.0013	0.0017	0.0022	0.0026	0.0030	0.0034	0.0039
	ρ	0.0032	0.0035	0.0038	0.0040	0.0043	0.0046	0.0048	0.0051	0.0053	0.0056
0.22	ρ'	—	0.0006	0.0011	0.0017	0.0022	0.0027	0.0032	0.0038	0.0043	0.0048
	ρ	0.0032	0.0035	0.0038	0.0041	0.0043	0.0046	0.0049	0.0051	0.0054	0.0057
0.24	ρ'	—	0.0008	0.0015	0.0022	0.0029	0.0036	0.0043	0.0050	0.0057	0.0064
	ρ	0.0032	0.0035	0.0038	0.0041	0.0044	0.0046	0.0049	0.0052	0.0055	0.0057
0.26	ρ'	—	0.0011	0.0022	0.0032	0.0042	0.0052	0.0062	0.0072	0.0082	0.0092
	ρ	0.0032	0.0035	0.0038	0.0041	0.0044	0.0047	0.0049	0.0052	0.0055	0.0058
0.28	ρ'	—	0.0020	0.0037	0.0055	0.0073	0.0090	0.0108	0.0125	0.0143	0.0160
	ρ	0.0032	0.0035	0.0038	0.0041	0.0044	0.0047	0.0050	0.0053	0.0056	0.0059
0.30	ρ'	—	0.0065	0.0123	0.0180	0.0237	0.0295	0.0352	0.0410	0.0467	0.0524
	ρ	0.0032	0.0036	0.0039	0.0042	0.0045	0.0047	0.0050	0.0053	0.0056	0.0059

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-74b Steel Ratio ρ and ρ' Versus K_f for f'_m = 1,500 psi, (Concrete Masonry)

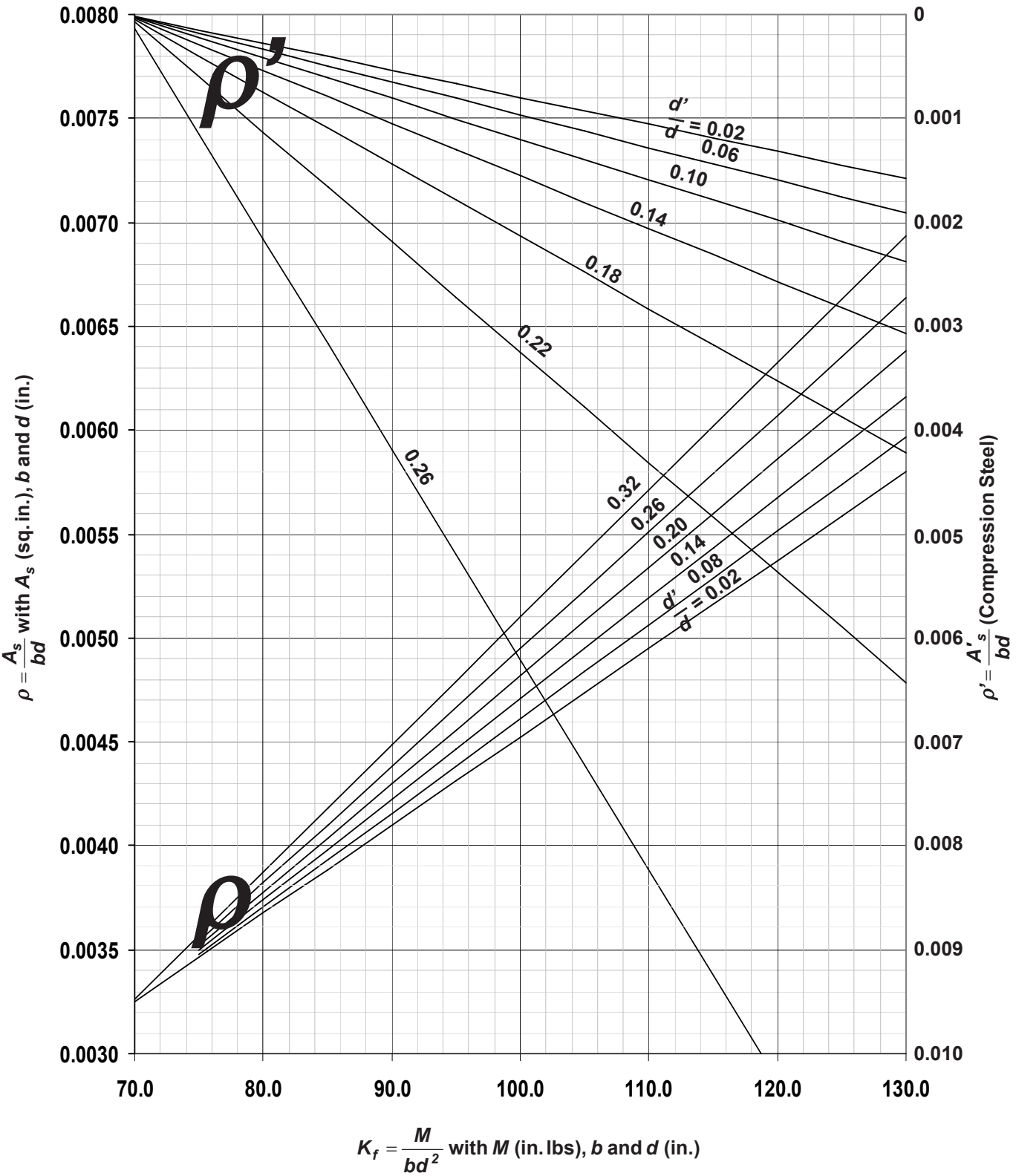
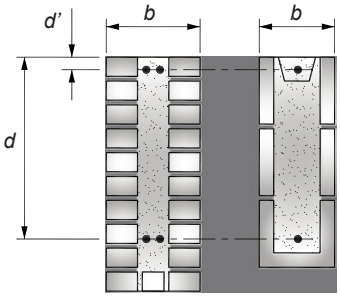


Table ASD-75a Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Clay Masonry) f'_m = 2000 psi, F_s = 24,000 psi, and n = 20.7

DESIGN DATA

f'_m = 2000 psi
f_y = 60,000 psi
f_b = 667 psi
F_s = 24,000 psi
E_m = 1,400,000 psi
E_s = 29,000,000 psi
n = 20.7
k = 0.365
K_{fb} = 106.9
ρ_b = 0.0051



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\,(\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\,(\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		106.9	115	120	125	130	135	140	145	150	155
0.02	ρ'	—	0.0002	0.0003	0.0004	0.0005	0.0006	0.0007	0.0008	0.0009	0.0010
	ρ	0.0051	0.0054	0.0056	0.0058	0.0061	0.0063	0.0065	0.0067	0.0069	0.0071
0.04	ρ'	—	0.0002	0.0003	0.0004	0.0005	0.0006	0.0007	0.0008	0.0009	0.0010
	ρ	0.0051	0.0054	0.0056	0.0059	0.0061	0.0063	0.0065	0.0067	0.0069	0.0072
0.06	ρ'	—	0.0002	0.0003	0.0004	0.0005	0.0007	0.0008	0.0009	0.0010	0.0011
	ρ	0.0051	0.0054	0.0057	0.0059	0.0061	0.0063	0.0065	0.0068	0.0070	0.0072
0.08	ρ'	—	0.0002	0.0003	0.0005	0.0006	0.0007	0.0009	0.0010	0.0011	0.0012
	ρ	0.0051	0.0054	0.0057	0.0059	0.0061	0.0063	0.0066	0.0068	0.0070	0.0073
0.10	ρ'	—	0.0002	0.0004	0.0005	0.0007	0.0008	0.0009	0.0011	0.0012	0.0014
	ρ	0.0051	0.0054	0.0057	0.0059	0.0061	0.0064	0.0066	0.0068	0.0071	0.0073
0.12	ρ'	—	0.0003	0.0004	0.0006	0.0007	0.0009	0.0010	0.0012	0.0014	0.0015
	ρ	0.0051	0.0055	0.0057	0.0059	0.0062	0.0064	0.0066	0.0069	0.0071	0.0073
0.14	ρ'	—	0.0003	0.0005	0.0006	0.0008	0.0010	0.0012	0.0013	0.0015	0.0017
	ρ	0.0051	0.0055	0.0057	0.0059	0.0062	0.0064	0.0067	0.0069	0.0072	0.0074
0.16	ρ'	—	0.0003	0.0005	0.0007	0.0009	0.0011	0.0013	0.0015	0.0017	0.0019
	ρ	0.0051	0.0055	0.0057	0.0060	0.0062	0.0065	0.0067	0.0070	0.0072	0.0075
0.18	ρ'	—	0.0004	0.0006	0.0008	0.0010	0.0013	0.0015	0.0017	0.0019	0.0021
	ρ	0.0051	0.0055	0.0057	0.0060	0.0062	0.0065	0.0068	0.0070	0.0073	0.0075
0.20	ρ'	—	0.0004	0.0007	0.0009	0.0012	0.0014	0.0017	0.0020	0.0022	0.0025
	ρ	0.0051	0.0055	0.0058	0.0060	0.0063	0.0065	0.0068	0.0071	0.0073	0.0076
0.22	ρ'	—	0.0005	0.0008	0.0011	0.0014	0.0017	0.0020	0.0023	0.0026	0.0029
	ρ	0.0051	0.0055	0.0058	0.0060	0.0063	0.0066	0.0068	0.0071	0.0074	0.0076
0.24[2]	ρ'	—	0.0006	0.0009	0.0013	0.0016	0.0020	0.0024	0.0027	0.0031	0.0034
	ρ	0.0051	0.0055	0.0058	0.0061	0.0063	0.0066	0.0069	0.0072	0.0074	0.0077
0.26	ρ'	—	0.0007	0.0011	0.0016	0.0020	0.0024	0.0029	0.0033	0.0037	0.0042
	ρ	0.0051	0.0055	0.0058	0.0061	0.0064	0.0067	0.0069	0.0072	0.0075	0.0078
0.28	ρ'	—	0.0009	0.0014	0.0020	0.0025	0.0031	0.0037	0.0042	0.0048	0.0053
	ρ	0.0051	0.0055	0.0058	0.0061	0.0064	0.0067	0.0070	0.0073	0.0076	0.0079
0.30	ρ'	—	0.0012	0.0019	0.0027	0.0034	0.0042	0.0049	0.0056	0.0064	0.0071
	ρ	0.0051	0.0056	0.0059	0.0061	0.0064	0.0067	0.0070	0.0073	0.0076	0.0079

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-75a Steel Ratio ρ and ρ' Versus K_f for f'_m = 2,000 psi, (Clay Masonry)

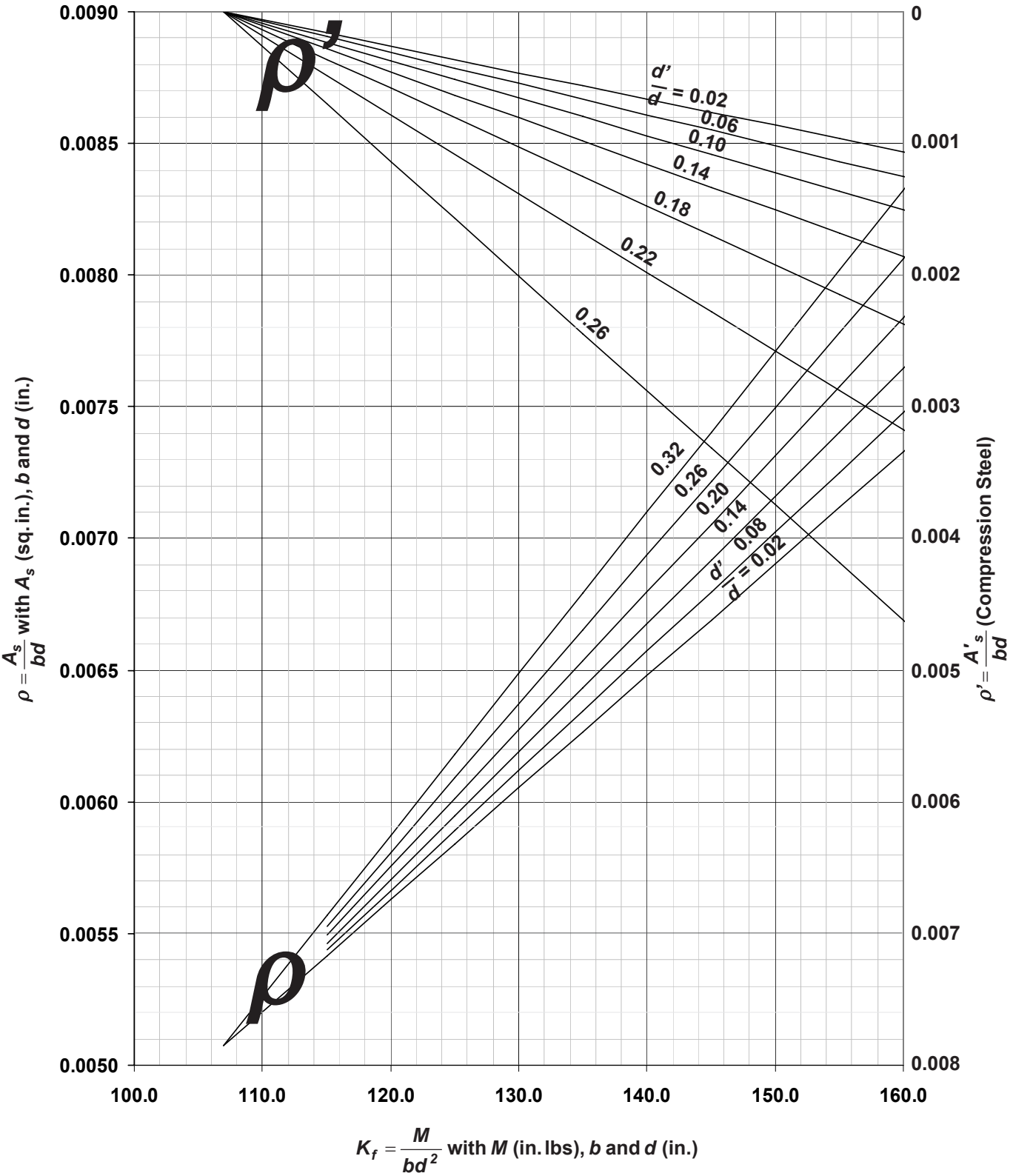
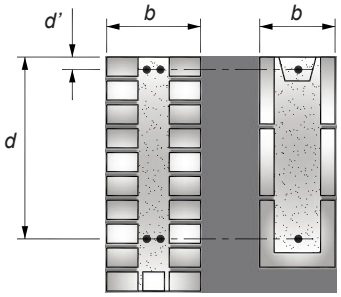


Table ASD-75b Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Concrete Masonry) f'_m = 2000 psi, F_s = 24,000 psi, and n = 16.1

DESIGN DATA

f'_m = 2000 psi
f_b = 667 psi
E_m = 1,800,000 psi
n = 16.1
K_{fb} = 92.4
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi
k = 0.309
ρ_b = 0.0043



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		92.4	100	105	110	115	120	125	130	135	140
0.02	ρ'	—	0.0002	0.0003	0.0005	0.0006	0.0007	0.0009	0.0010	0.0011	0.0012
	ρ	0.0043	0.0046	0.0048	0.0050	0.0053	0.0055	0.0057	0.0059	0.0061	0.0063
0.04	ρ'	—	0.0002	0.0004	0.0005	0.0006	0.0008	0.0009	0.0011	0.0012	0.0014
	ρ	0.0043	0.0046	0.0048	0.0051	0.0053	0.0055	0.0057	0.0059	0.0061	0.0064
0.06	ρ'	—	0.0002	0.0004	0.0006	0.0007	0.0009	0.0010	0.0012	0.0013	0.0015
	ρ	0.0043	0.0046	0.0049	0.0061	0.0053	0.0055	0.0057	0.0060	0.0062	0.0064
0.08	ρ'	—	0.0003	0.0004	0.0006	0.0008	0.0010	0.0011	0.0013	0.0015	0.0017
	ρ	0.0043	0.0046	0.0049	0.0051	0.0053	0.0055	0.0058	0.0060	0.0062	0.0064
0.10	ρ'	—	0.0003	0.0005	0.0007	0.0009	0.0011	0.0013	0.0015	0.0017	0.0019
	ρ	0.0043	0.0046	0.0049	0.0051	0.0053	0.0056	0.0058	0.0060	0.0063	0.0065
0.12	ρ'	—	0.0003	0.0006	0.0008	0.0010	0.0012	0.0015	0.0017	0.0019	0.0021
	ρ	0.0043	0.0047	0.0049	0.0051	0.0054	0.0056	0.0058	0.0061	0.0063	0.0065
0.14	ρ'	—	0.0004	0.0006	0.0009	0.0012	0.0014	0.0017	0.0019	0.0022	0.0024
	ρ	0.0043	0.0047	0.0049	0.0051	0.0054	0.0056	0.0059	0.0061	0.0064	0.0066
0.16	ρ'	—	0.0004	0.0007	0.0010	0.0013	0.0016	0.0019	0.0022	0.0025	0.0028
	ρ	0.0043	0.0047	0.0049	0.0052	0.0054	0.0057	0.0059	0.0062	0.0064	0.0067
0.18	ρ'	—	0.0005	0.0009	0.0012	0.0016	0.0019	0.0023	0.0026	0.0030	0.0033
	ρ	0.0043	0.0047	0.0049	0.0052	0.0054	0.0057	0.0059	0.0062	0.0065	0.0067
0.20	ρ'	—	0.0006	0.0011	0.0015	0.0019	0.0023	0.0028	0.0032	0.0036	0.0040
	ρ	0.0043	0.0047	0.0049	0.0052	0.0055	0.0057	0.0060	0.0063	0.0065	0.0068
0.22	ρ'	—	0.0008	0.0013	0.0019	0.0024	0.0029	0.0035	0.0040	0.0045	0.0051
	ρ	0.0043	0.0047	0.0050	0.0052	0.0055	0.0058	0.0060	0.0063	0.0066	0.0068
0.24	ρ'	—	0.0011	0.0018	0.0025	0.0032	0.0039	0.0046	0.0053	0.0060	0.0067
	ρ	0.0043	0.0047	0.0050	0.0053	0.0055	0.0058	0.0061	0.0064	0.0066	0.0069
0.26	ρ'	—	0.0015	0.0026	0.0036	0.0046	0.0056	0.0066	0.0077	0.0087	0.0097
	ρ	0.0043	0.0047	0.0050	0.0053	0.0056	0.0058	0.0061	0.0064	0.0067	0.0070
0.28	ρ'	—	0.0027	0.0044	0.0062	0.0080	0.0097	0.0115	0.0133	0.0151	0.0168
	ρ	0.0043	0.0047	0.0050	0.0053	0.0056	0.0059	0.0062	0.0065	0.0068	0.0070
0.30	ρ'	—	0.0088	0.0145	0.0203	0.0261	0.0319	0.0377	0.0435	0.0493	0.0550
	ρ	0.0043	0.0047	0.0050	0.0053	0.0056	0.0059	0.0062	0.0065	0.0068	0.0071

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-75b Steel Ratio ρ and ρ' Versus K_f for f'_m = 2,000 psi, (Concrete Masonry)

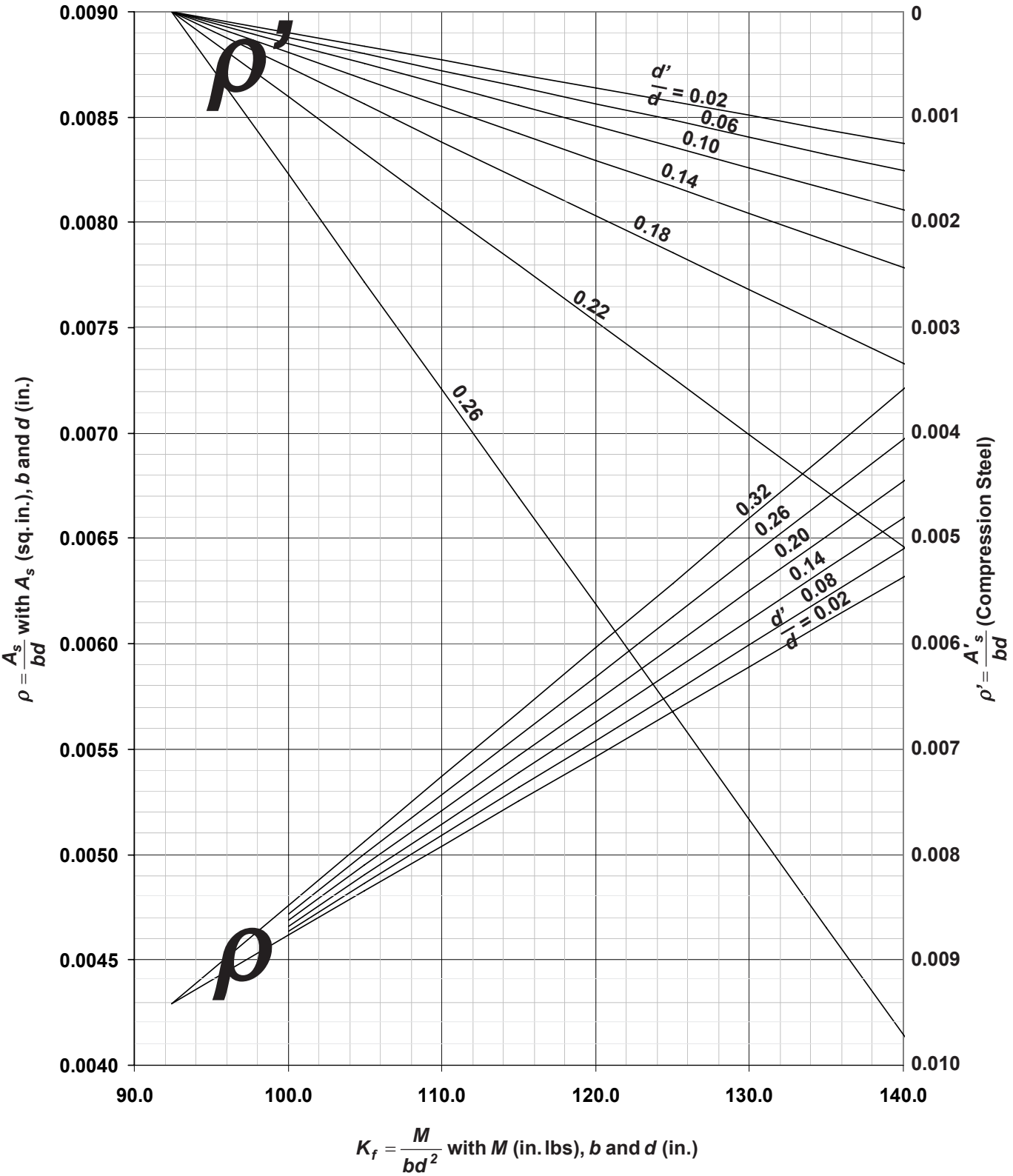
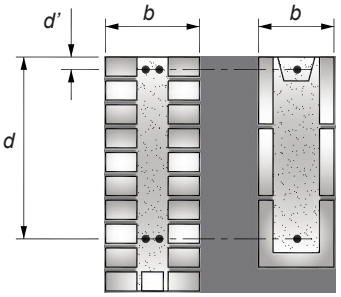


Table ASD-76a Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Clay Masonry) f'_m = 2500 psi, F_s = 24,000 psi, and n = 16.6

DESIGN DATA

f'_m = 2500 psi
f_y = 60,000 psi
f_b = 833 psi
F_s = 24,000 psi
E_m = 1,750,000 psi
E_s = 29,000,000 psi
n = 16.6
k = 0.365
K_{fb} = 133.7
ρ_b = 0.0063



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		133.7	145	150	155	160	165	170	175	180	185
0.02	ρ'	—	0.0002	0.0003	0.0004	0.0005	0.0006	0.0007	0.0008	0.0009	0.0010
	ρ	0.0063	0.0068	0.0070	0.0072	0.0075	0.0077	0.0079	0.0081	0.0083	0.0085
0.04	ρ'	—	0.0002	0.0004	0.0005	0.0006	0.0007	0.0008	0.0009	0.0010	0.0011
	ρ	0.0063	0.0068	0.0071	0.0073	0.0075	0.0077	0.0079	0.0081	0.0084	0.0086
0.06	ρ'	—	0.0003	0.0004	0.0005	0.0006	0.0007	0.0009	0.0010	0.0011	0.0012
	ρ	0.0063	0.0068	0.0071	0.0073	0.0075	0.0077	0.0080	0.0082	0.0084	0.0086
0.08	ρ'	—	0.0003	0.0004	0.0006	0.0007	0.0008	0.0009	0.0011	0.0012	0.0013
	ρ	0.0063	0.0069	0.0071	0.0073	0.0075	0.0078	0.0080	0.0082	0.0084	0.0087
0.10	ρ'	—	0.0003	0.0005	0.0006	0.0008	0.0009	0.0010	0.0012	0.0013	0.0015
	ρ	0.0063	0.0069	0.0071	0.0073	0.0076	0.0078	0.0080	0.0083	0.0085	0.0087
0.12	ρ'	—	0.0004	0.0005	0.0007	0.0008	0.0010	0.0011	0.0013	0.0015	0.0016
	ρ	0.0063	0.0069	0.0071	0.0074	0.0076	0.0078	0.0081	0.0083	0.0085	0.0088
0.14	ρ'	—	0.0004	0.0006	0.0008	0.0009	0.0011	0.0013	0.0015	0.0016	0.0018
	ρ	0.0063	0.0069	0.0071	0.0074	0.0076	0.0079	0.0081	0.0083	0.0086	0.0088
0.16	ρ'	—	0.0004	0.0006	0.0008	0.0010	0.0012	0.0014	0.0016	0.0018	0.0020
	ρ	0.0063	0.0069	0.0072	0.0074	0.0076	0.0079	0.0081	0.0084	0.0086	0.0089
0.18	ρ'	—	0.0005	0.0007	0.0010	0.0012	0.0014	0.0016	0.0019	0.0021	0.0023
	ρ	0.0063	0.0069	0.0072	0.0074	0.0077	0.0079	0.0082	0.0084	0.0087	0.0089
0.20	ρ'	—	0.0006	0.0008	0.0011	0.0014	0.0016	0.0019	0.0021	0.0024	0.0026
	ρ	0.0063	0.0069	0.0072	0.0075	0.0077	0.0080	0.0082	0.0085	0.0088	0.0090
0.22	ρ'	—	0.0007	0.0010	0.0013	0.0016	0.0019	0.0022	0.0025	0.0028	0.0031
	ρ	0.0063	0.0069	0.0072	0.0075	0.0077	0.0080	0.0083	0.0085	0.0088	0.0091
0.24	ρ'	—	0.0008	0.0012	0.0015	0.0019	0.0022	0.0026	0.0030	0.0033	0.0037
	ρ	0.0063	0.0070	0.0072	0.0075	0.0078	0.0081	0.0083	0.0086	0.0089	0.0092
0.26	ρ'	—	0.0010	0.0014	0.0019	0.0023	0.0027	0.0032	0.0036	0.0041	0.0045
	ρ	0.0063	0.0070	0.0073	0.0075	0.0078	0.0081	0.0084	0.0087	0.0090	0.0092
0.28	ρ'	—	0.0013	0.0018	0.0024	0.0029	0.0035	0.0040	0.0046	0.0051	0.0057
	ρ	0.0063	0.0070	0.0073	0.0076	0.0079	0.0082	0.0084	0.0087	0.0090	0.0093
0.30	ρ'	—	0.0017	0.0024	0.0032	0.0039	0.0047	0.0054	0.0062	0.0069	0.0077
	ρ	0.0063	0.0070	0.0073	0.0076	0.0079	0.0082	0.0085	0.0088	0.0091	0.0094

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-76a Steel Ratio ρ and ρ' Versus K_f for f'_m = 2,500 psi, (Clay Masonry)

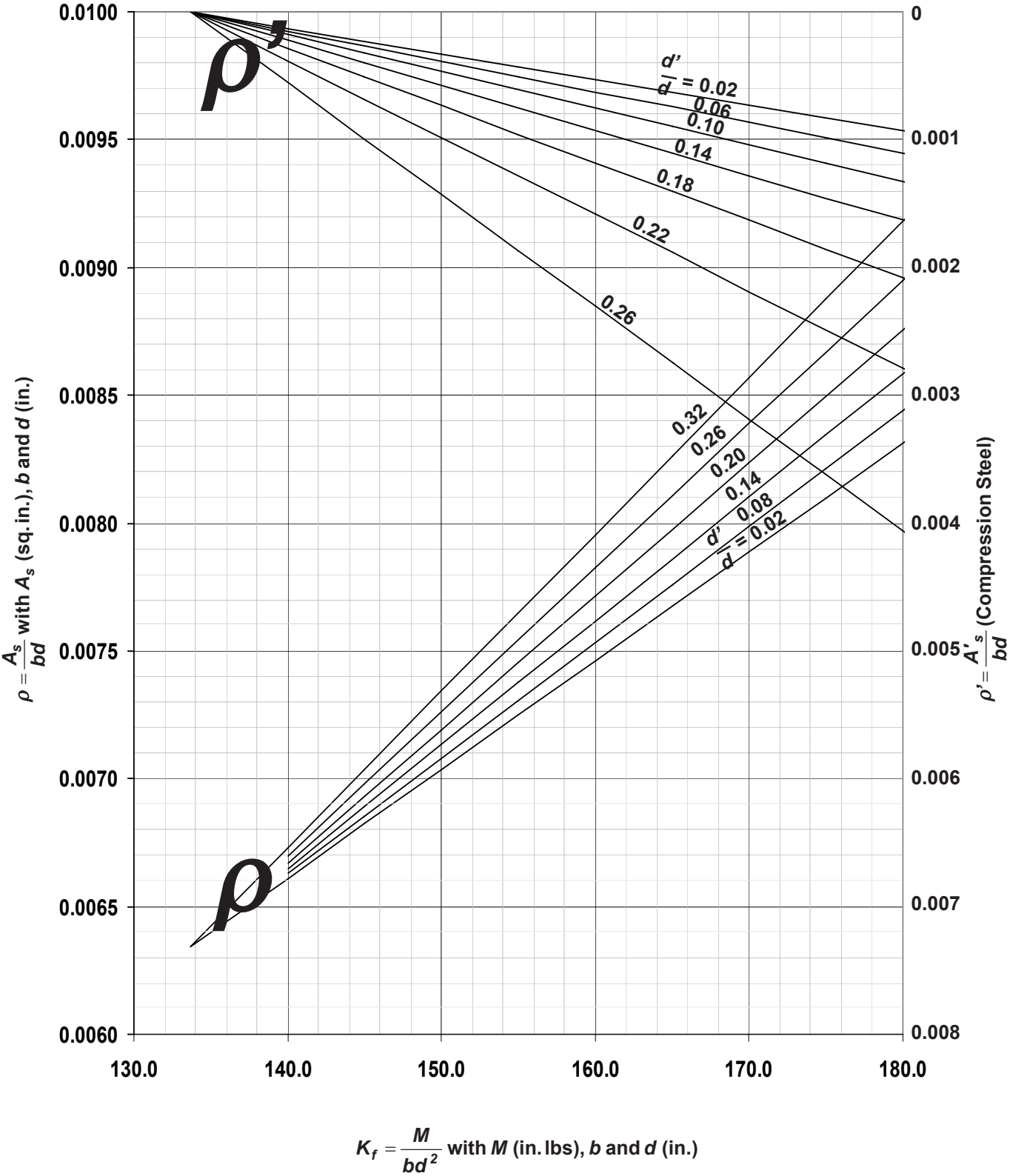
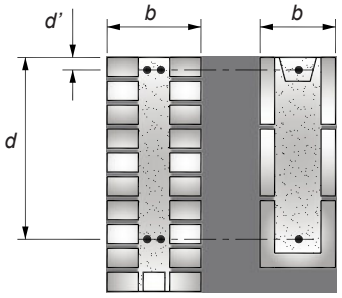


Table ASD-76b Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Concrete Masonry) f'_m = 2500 psi, F_s = 24,000 psi, and n = 12.9

DESIGN DATA

f'_m = 2500 psi
f_b = 833 psi
E_m = 2,250,000 psi
n = 12.9
K_{fb} = 115.5
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi
k = 0.309
ρ_b = 0.0054



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		115.5	130	135	140	145	150	155	160	165	170
0.02	ρ'	—	0.0004	0.0005	0.0006	0.0008	0.0009	0.0010	0.0012	0.0013	0.0014
	ρ	0.0054	0.0060	0.0062	0.0064	0.0066	0.0068	0.0070	0.0073	0.0075	0.0077
0.04	ρ'	—	0.0004	0.0006	0.0007	0.0009	0.0010	0.0011	0.0013	0.0014	0.0016
	ρ	0.0054	0.0060	0.0062	0.0064	0.0066	0.0069	0.0071	0.0073	0.0075	0.0077
0.06	ρ'	—	0.0005	0.0006	0.0008	0.0009	0.0011	0.0013	0.0014	0.0016	0.0017
	ρ	0.0054	0.0060	0.0062	0.0065	0.0067	0.0069	0.0071	0.0073	0.0076	0.0078
0.08	ρ'	—	0.0005	0.0007	0.0009	0.0010	0.0012	0.0014	0.0016	0.0018	0.0019
	ρ	0.0054	0.0060	0.0062	0.0065	0.0067	0.0069	0.0072	0.0074	0.0076	0.0078
0.10	ρ'	—	0.0006	0.0008	0.0010	0.0012	0.0014	0.0016	0.0018	0.0020	0.0022
	ρ	0.0054	0.0060	0.0063	0.0065	0.0067	0.0070	0.0072	0.0074	0.0077	0.0079
0.12	ρ'	—	0.0007	0.0009	0.0011	0.0013	0.0015	0.0018	0.0020	0.0022	0.0024
	ρ	0.0054	0.0061	0.0063	0.0065	0.0068	0.0070	0.0072	0.0075	0.0077	0.0079
0.14	ρ'	—	0.0007	0.0010	0.0013	0.0015	0.0018	0.0020	0.0023	0.0025	0.0028
	ρ	0.0054	0.0061	0.0063	0.0066	0.0068	0.0070	0.0073	0.0075	0.0078	0.0080
0.16	ρ'	—	0.0009	0.0012	0.0015	0.0018	0.0021	0.0024	0.0027	0.0030	0.0033
	ρ	0.0054	0.0061	0.0063	0.0066	0.0068	0.0070	0.0073	0.0076	0.0078	0.0081
0.18	ρ'	—	0.0010	0.0014	0.0017	0.0021	0.0024	0.0028	0.0031	0.0035	0.0038
	ρ	0.0054	0.0061	0.0064	0.0066	0.0069	0.0071	0.0074	0.0076	0.0079	0.0081
0.20	ρ'	—	0.0012	0.0017	0.0021	0.0025	0.0030	0.0034	0.0038	0.0042	0.0047
	ρ	0.0054	0.0061	0.0064	0.0066	0.0069	0.0072	0.0074	0.0077	0.0079	0.0082
0.22	ρ'	—	0.0016	0.0021	0.0026	0.0032	0.0037	0.0042	0.0048	0.0053	0.0059
	ρ	0.0054	0.0061	0.0064	0.0067	0.0069	0.0072	0.0075	0.0077	0.0080	0.0083
0.24	ρ'	—	0.0021	0.0028	0.0035	0.0042	0.0049	0.0056	0.0063	0.0070	0.0078
	ρ	0.0054	0.0062	0.0064	0.0067	0.0070	0.0073	0.0075	0.0078	0.0081	0.0084
0.26	ρ'	—	0.0030	0.0040	0.0050	0.0061	0.0071	0.0081	0.0091	0.0102	0.0112
	ρ	0.0054	0.0062	0.0065	0.0067	0.0070	0.0073	0.0076	0.0079	0.0082	0.0084
0.28	ρ'	—	0.0052	0.0069	0.0087	0.0105	0.0123	0.0141	0.0158	0.0176	0.0194
	ρ	0.0054	0.0062	0.0065	0.0068	0.0071	0.0074	0.0077	0.0079	0.0082	0.0085
0.30	ρ'	—	0.0169	0.0227	0.0285	0.0344	0.0402	0.0460	0.0519	0.0577	0.0635
	ρ	0.0054	0.0062	0.0065	0.0068	0.0071	0.0074	0.0077	0.0080	0.0083	0.0086

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-76b Steel Ratio ρ and ρ' Versus K_f for f'_m = 2,500 psi, (Concrete Masonry)

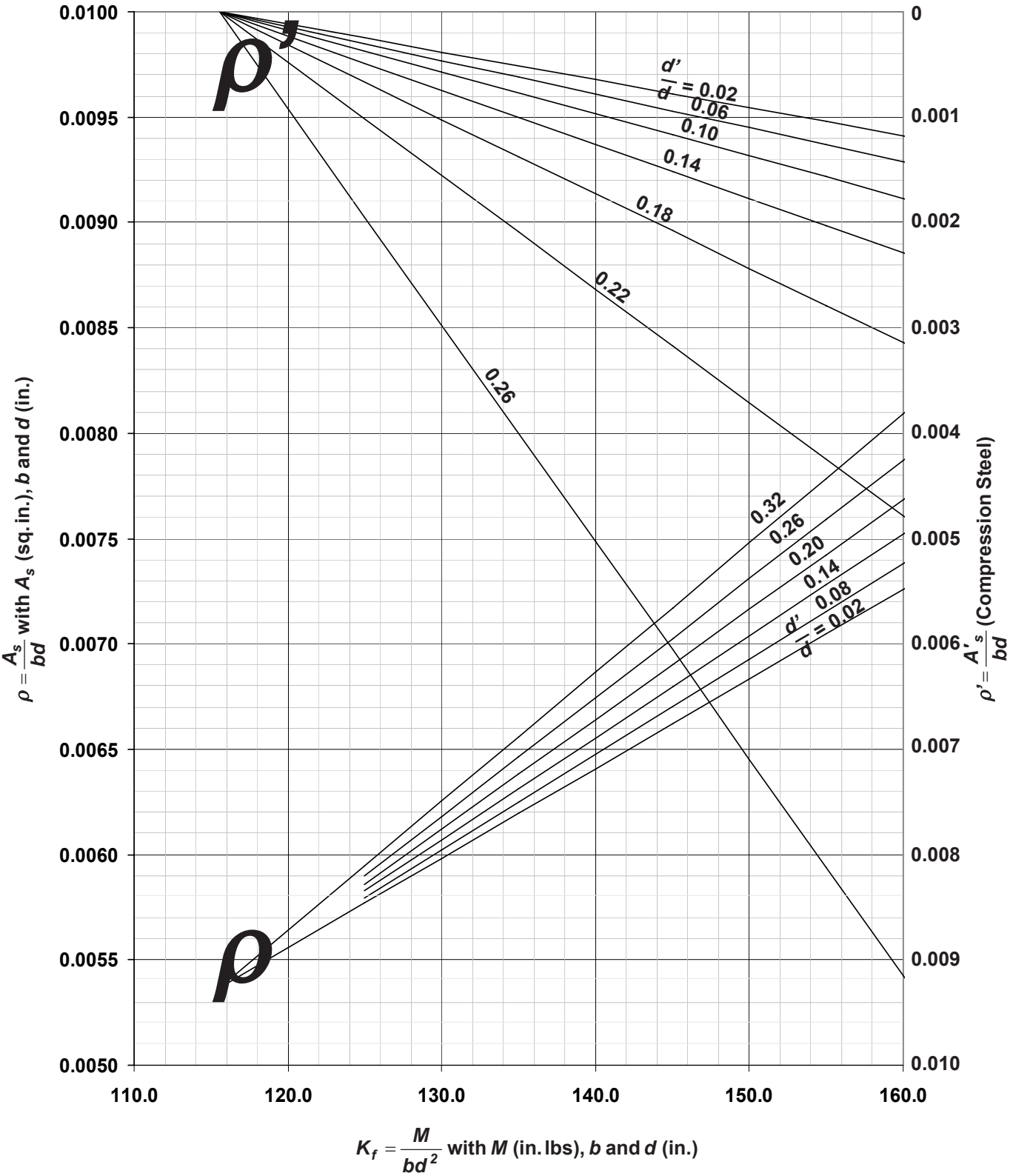
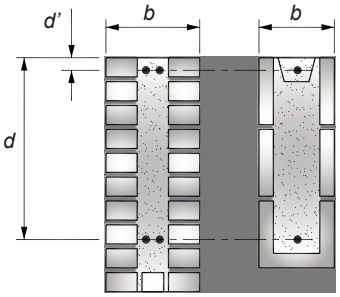


Table ASD-77a Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Clay Masonry) f'_m = 3000 psi, F_s = 24,000 psi, and n = 13.8

DESIGN DATA

f'_m = 3000 psi	f_y = 60,000 psi
f_b = 1000 psi	F_s = 24,000 psi
E_m = 2,100,000 psi	E_s = 29,000,000 psi
n = 13.8	k = 0.365
K_{fb} = 160.4	ρ_b = 0.0076



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\,(\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\,(\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		160.4	175	180	185	190	195	200	205	210	215
0.02	ρ'	—	0.0003	0.0004	0.0005	0.0006	0.0007	0.0008	0.0009	0.0010	0.0011
	ρ	0.0076	0.0082	0.0084	0.0087	0.0089	0.0091	0.0093	0.0095	0.0097	0.0099
0.04	ρ'	—	0.0003	0.0004	0.0005	0.0007	0.0008	0.0009	0.0010	0.0011	0.0012
	ρ	0.0076	0.0082	0.0085	0.0087	0.0089	0.0091	0.0093	0.0095	0.0098	0.0100
0.06	ρ'	—	0.0003	0.0005	0.0006	0.0007	0.0008	0.0009	0.0011	0.0012	0.0013
	ρ	0.0076	0.0083	0.0085	0.0087	0.0089	0.0091	0.0094	0.0096	0.0098	0.0100
0.08	ρ'	—	0.0004	0.0005	0.0006	0.0008	0.0009	0.0010	0.0012	0.0013	0.0014
	ρ	0.0076	0.0083	0.0085	0.0087	0.0090	0.0092	0.0094	0.0096	0.0099	0.0101
0.10	ρ'	—	0.0004	0.0006	0.0007	0.0009	0.0010	0.0011	0.0013	0.0014	0.0016
	ρ	0.0076	0.0083	0.0085	0.0087	0.0090	0.0092	0.0094	0.0097	0.0099	0.0101
0.12	ρ'	—	0.0005	0.0006	0.0008	0.0009	0.0011	0.0013	0.0014	0.0016	0.0017
	ρ	0.0076	0.0083	0.0085	0.0088	0.0090	0.0092	0.0095	0.0097	0.0100	0.0102
0.14	ρ'	—	0.0005	0.0007	0.0009	0.0010	0.0012	0.0014	0.0016	0.0018	0.0019
	ρ	0.0076	0.0083	0.0086	0.0088	0.0090	0.0093	0.0095	0.0098	0.0100	0.0103
0.16	ρ'	—	0.0006	0.0008	0.0010	0.0012	0.0014	0.0016	0.0018	0.0020	0.0022
	ρ	0.0076	0.0083	0.0086	0.0088	0.0091	0.0093	0.0096	0.0098	0.0101	0.0103
0.18	ρ'	—	0.0007	0.0009	0.0011	0.0013	0.0016	0.0018	0.0020	0.0022	0.0025
	ρ	0.0076	0.0084	0.0086	0.0089	0.0091	0.0094	0.0096	0.0099	0.0101	0.0104
0.20	ρ'	—	0.0008	0.0010	0.0013	0.0015	0.0018	0.0021	0.0023	0.0026	0.0028
	ρ	0.0076	0.0084	0.0086	0.0089	0.0092	0.0094	0.0097	0.0099	0.0102	0.0105
0.22	ρ'	—	0.0009	0.0012	0.0015	0.0018	0.0021	0.0024	0.0027	0.0030	0.0033
	ρ	0.0076	0.0084	0.0087	0.0089	0.0092	0.0095	0.0097	0.0100	0.0103	0.0105
0.24	ρ'	—	0.0011	0.0014	0.0018	0.0021	0.0025	0.0029	0.0032	0.0036	0.0039
	ρ	0.0076	0.0084	0.0087	0.0090	0.0092	0.0095	0.0098	0.0101	0.0103	0.0106
0.26	ρ'	—	0.0013	0.0017	0.0022	0.0026	0.0030	0.0035	0.0039	0.0044	0.0048
	ρ	0.0076	0.0084	0.0087	0.0090	0.0093	0.0096	0.0098	0.0101	0.0104	0.0107
0.28	ρ'	—	0.0016	0.0022	0.0028	0.0033	0.0039	0.0044	0.0050	0.0055	0.0061
	ρ	0.0076	0.0085	0.0087	0.0090	0.0093	0.0096	0.0099	0.0102	0.0105	0.0108
0.30	ρ'	—	0.0022	0.0029	0.0037	0.0044	0.0052	0.0060	0.0067	0.0075	0.0082
	ρ	0.0076	0.0085	0.0088	0.0091	0.0094	0.0097	0.0100	0.0103	0.0106	0.0109

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-77a Steel Ratio ρ and ρ' Versus K_f for f'_m = 3,000 psi, (Clay Masonry)

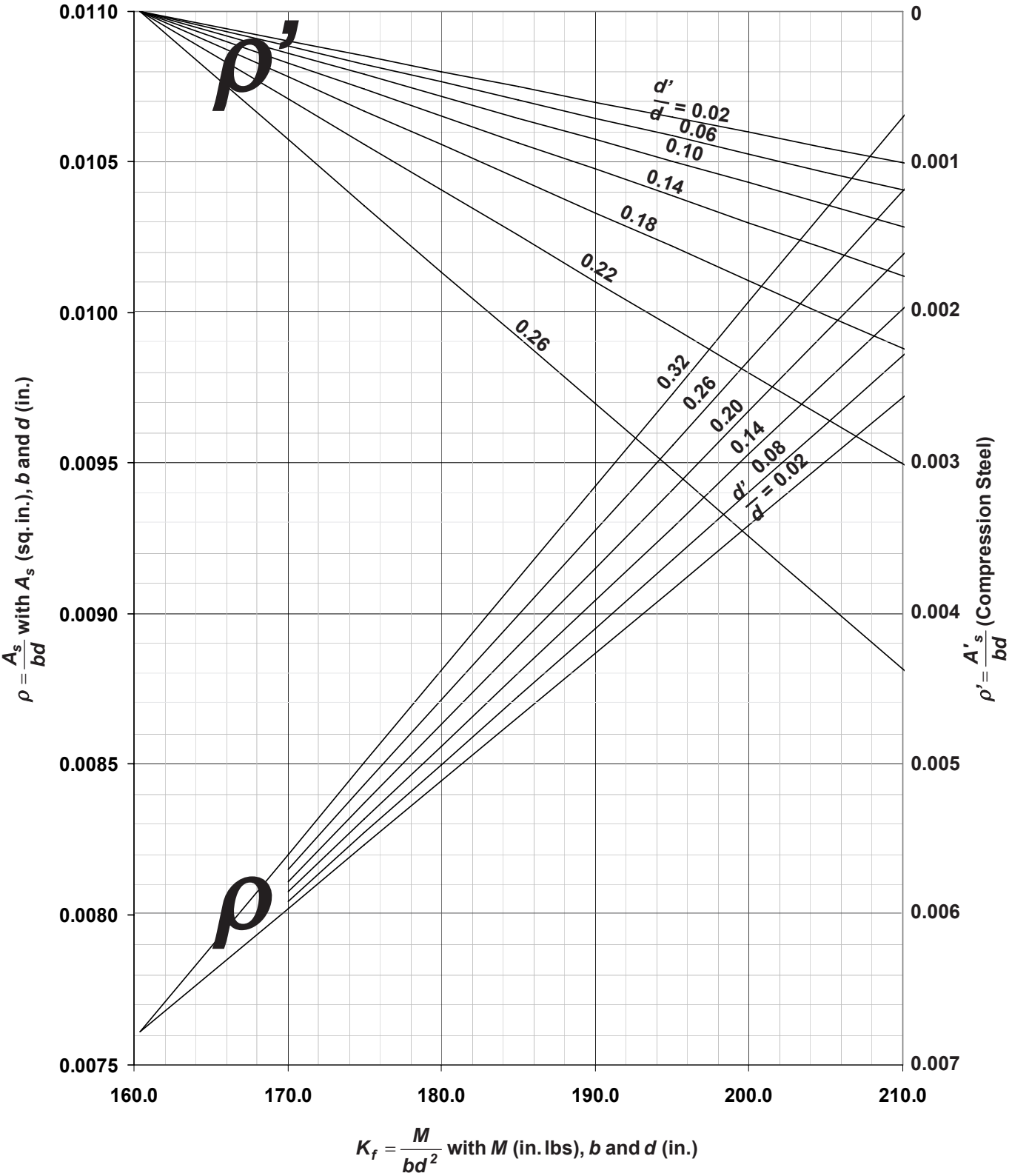
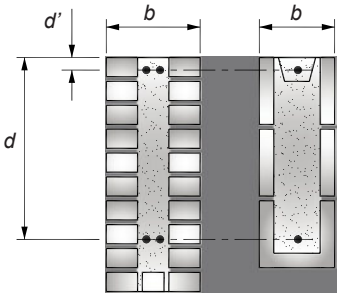


Table ASD-77b Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Concrete Masonry) f'_m = 3000 psi, F_s = 24,000 psi, and n = 10.7

DESIGN DATA

f'_m = 3000 psi
f_b = 1000 psi
E_m = 2,700,000 psi
n = 10.7
K_{fb} = 138.7
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi
k = 0.309
ρ_b = 0.0064



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		138.7	150	155	160	165	170	175	180	185	190
0.02	ρ'	—	0.0003	0.0004	0.0006	0.0007	0.0008	0.0010	0.0011	0.0012	0.0014
	ρ	0.0064	0.0069	0.0071	0.0073	0.0076	0.0078	0.0080	0.0082	0.0084	0.0086
0.04	ρ'	—	0.0003	0.0005	0.0006	0.0008	0.0009	0.0011	0.0012	0.0014	0.0015
	ρ	0.0064	0.0069	0.0072	0.0074	0.0076	0.0078	0.0080	0.0082	0.0085	0.0087
0.06	ρ'	—	0.0004	0.0005	0.0007	0.0008	0.0010	0.0012	0.0013	0.0015	0.0017
	ρ	0.0064	0.0069	0.0072	0.0074	0.0076	0.0078	0.0081	0.0083	0.0085	0.0087
0.08	ρ'	—	0.0004	0.0006	0.0008	0.0009	0.0011	0.0013	0.0015	0.0017	0.0018
	ρ	0.0064	0.0070	0.0072	0.0074	0.0076	0.0079	0.0081	0.0083	0.0085	0.0088
0.10	ρ'	—	0.0005	0.0007	0.0009	0.0011	0.0013	0.0015	0.0017	0.0019	0.0021
	ρ	0.0064	0.0070	0.0072	0.0074	0.0077	0.0079	0.0081	0.0084	0.0086	0.0088
0.12	ρ'	—	0.0005	0.0007	0.0010	0.0012	0.0014	0.0016	0.0019	0.0021	0.0023
	ρ	0.0064	0.0070	0.0072	0.0075	0.0077	0.0079	0.0082	0.0084	0.0086	0.0089
0.14	ρ'	—	0.0006	0.0008	0.0011	0.0014	0.0016	0.0019	0.0021	0.0024	0.0027
	ρ	0.0064	0.0070	0.0072	0.0075	0.0077	0.0080	0.0082	0.0084	0.0087	0.0089
0.16	ρ'	—	0.0007	0.0010	0.0013	0.0016	0.0019	0.0022	0.0025	0.0028	0.0031
	ρ	0.0064	0.0070	0.0073	0.0075	0.0077	0.0080	0.0082	0.0085	0.0087	0.0090
0.18	ρ'	—	0.0008	0.0012	0.0015	0.0019	0.0022	0.0026	0.0029	0.0033	0.0037
	ρ	0.0064	0.0070	0.0073	0.0075	0.0078	0.0080	0.0083	0.0085	0.0088	0.0091
0.20	ρ'	—	0.0010	0.0014	0.0018	0.0023	0.0027	0.0031	0.0036	0.0040	0.0044
	ρ	0.0064	0.0070	0.0073	0.0076	0.0078	0.0081	0.0083	0.0086	0.0089	0.0091
0.22	ρ'	—	0.0012	0.0018	0.0023	0.0029	0.0034	0.0039	0.0045	0.0050	0.0056
	ρ	0.0064	0.0070	0.0073	0.0076	0.0078	0.0081	0.0084	0.0086	0.0089	0.0092
0.24	ρ'	—	0.0016	0.0023	0.0031	0.0038	0.0045	0.0052	0.0059	0.0067	0.0074
	ρ	0.0064	0.0071	0.0073	0.0076	0.0079	0.0082	0.0084	0.0087	0.0090	0.0093
0.26	ρ'	—	0.0024	0.0034	0.0044	0.0055	0.0065	0.0075	0.0086	0.0096	0.0107
	ρ	0.0064	0.0071	0.0074	0.0076	0.0079	0.0082	0.0085	0.0088	0.0091	0.0093
0.28	ρ'	—	0.0041	0.0059	0.0077	0.0095	0.0113	0.0131	0.0149	0.0167	0.0185
	ρ	0.0064	0.0071	0.0074	0.0077	0.0080	0.0083	0.0085	0.0088	0.0091	0.0094
0.30	ρ'	—	0.0133	0.0192	0.0251	0.0310	0.0369	0.0427	0.0486	0.0545	0.0604
	ρ	0.0064	0.0071	0.0074	0.0077	0.0080	0.0083	0.0086	0.0089	0.0092	0.0095

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-77b Steel Ratio ρ and ρ' Versus K_f for f'_m = 3,000 psi, (Concrete Masonry)

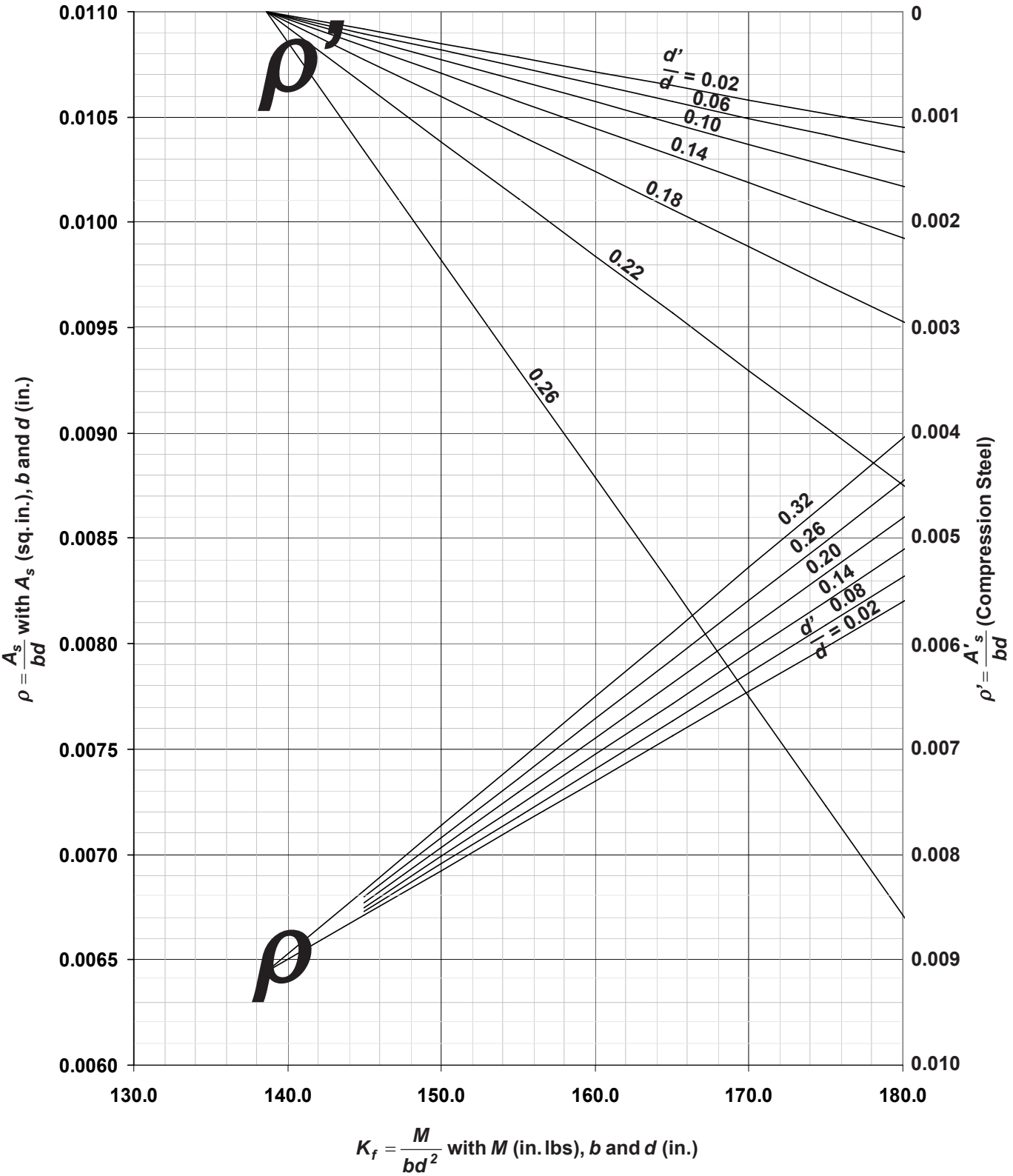
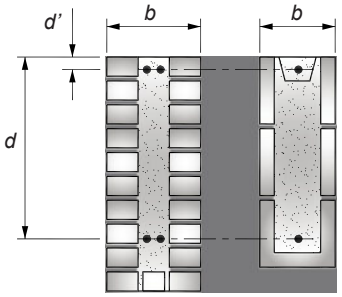


Table ASD-78a Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Clay Masonry) f'_m = 3500 psi, F_s = 24,000 psi, and n = 11.8

DESIGN DATA

f'_m = 3500 psi
f_y = 60,000 psi
f_b = 1167 psi
F_s = 24,000 psi
E_m = 2,450,000 psi
E_s = 29,000,000 psi
n = 11.8
k = 0.365
K_{fb} = 187.1
ρ_b = 0.0089



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		187.1	205	215	225	235	245	255	265	275	285
0.02	ρ'	—	0.0004	0.0006	0.0008	0.0010	0.0012	0.0014	0.0016	0.0018	0.0020
	ρ	0.0089	0.0096	0.0101	0.0105	0.0109	0.0113	0.0118	0.0122	0.0126	0.0130
0.04	ρ'	—	0.0004	0.0006	0.0008	0.0011	0.0013	0.0015	0.0017	0.0019	0.0022
	ρ	0.0089	0.0097	0.0101	0.0105	0.0110	0.0114	0.0118	0.0123	0.0127	0.0131
0.06	ρ'	—	0.0004	0.0007	0.0009	0.0012	0.0014	0.0016	0.0019	0.0021	0.0024
	ρ	0.0089	0.0097	0.0101	0.0106	0.0110	0.0114	0.0119	0.0123	0.0128	0.0132
0.08	ρ'	—	0.0005	0.0007	0.0010	0.0013	0.0015	0.0018	0.0020	0.0023	0.0026
	ρ	0.0089	0.0097	0.0101	0.0106	0.0110	0.0115	0.0120	0.0124	0.0129	0.0133
0.10	ρ'	—	0.0005	0.0008	0.0011	0.0014	0.0017	0.0020	0.0023	0.0025	0.0028
	ρ	0.0089	0.0097	0.0102	0.0106	0.0111	0.0116	0.0120	0.0125	0.0129	0.0134
0.12	ρ'	—	0.0006	0.0009	0.0012	0.0015	0.0019	0.0022	0.0025	0.0028	0.0031
	ρ	0.0089	0.0097	0.0102	0.0107	0.0111	0.0116	0.0121	0.0126	0.0130	0.0135
0.14	ρ'	—	0.0006	0.0010	0.0014	0.0017	0.0021	0.0024	0.0028	0.0031	0.0035
	ρ	0.0089	0.0097	0.0102	0.0107	0.0112	0.0117	0.0122	0.0127	0.0131	0.0136
0.16	ρ'	—	0.0007	0.0011	0.0015	0.0019	0.0023	0.0027	0.0031	0.0035	0.0039
	ρ	0.0089	0.0098	0.0103	0.0108	0.0113	0.0117	0.0122	0.0127	0.0132	0.0137
0.18	ρ'	—	0.0008	0.0013	0.0017	0.0022	0.0026	0.0031	0.0035	0.0040	0.0044
	ρ	0.0089	0.0098	0.0103	0.0108	0.0113	0.0118	0.0123	0.0128	0.0133	0.0139
0.20	ρ'	—	0.0009	0.0015	0.0020	0.0025	0.0030	0.0035	0.0041	0.0046	0.0051
	ρ	0.0089	0.0098	0.0103	0.0109	0.0114	0.0119	0.0124	0.0129	0.0135	0.0140
0.22	ρ'	—	0.0011	0.0017	0.0023	0.0029	0.0035	0.0041	0.0047	0.0054	0.0060
	ρ	0.0089	0.0098	0.0104	0.0109	0.0114	0.0120	0.0125	0.0130	0.0136	0.0141
0.24	ρ'	—	0.0013	0.0020	0.0027	0.0035	0.0042	0.0049	0.0056	0.0064	0.0071
	ρ	0.0089	0.0099	0.0104	0.0110	0.0115	0.0121	0.0126	0.0131	0.0137	0.0142
0.26	ρ'	—	0.0016	0.0025	0.0034	0.0042	0.0051	0.0060	0.0069	0.0078	0.0087
	ρ	0.0089	0.0099	0.0104	0.0110	0.0116	0.0121	0.0127	0.0133	0.0138	0.0144
0.28	ρ'	—	0.0020	0.0031	0.0043	0.0054	0.0065	0.0076	0.0088	0.0099	0.0110
	ρ	0.0089	0.0099	0.0105	0.0111	0.0116	0.0122	0.0128	0.0134	0.0140	0.0145
0.30	ρ'	—	0.0027	0.0042	0.0057	0.0072	0.0088	0.0103	0.0118	0.0133	0.0148
	ρ	0.0089	0.0099	0.0105	0.0111	0.0117	0.0123	0.0129	0.0135	0.0141	0.0147

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-78a Steel Ratio ρ and ρ' Versus K_f for f'_m = 3,500 psi, (Clay Masonry)

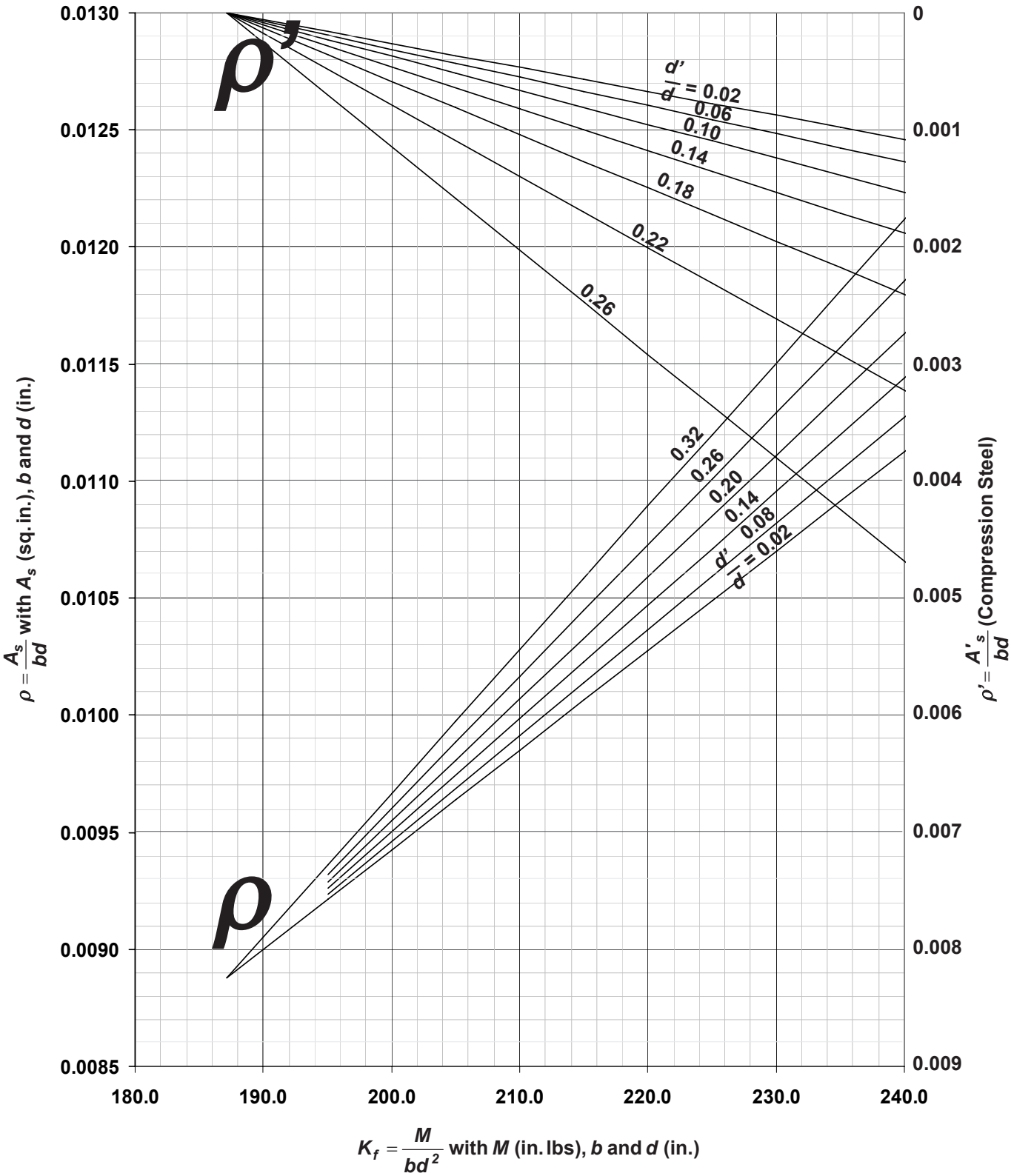
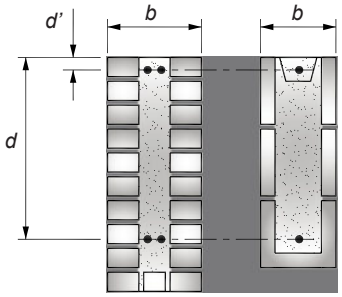


Table ASD-78b Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Concrete Masonry) f'_m = 3500 psi, F_s = 24,000 psi, and n = 9.2

DESIGN DATA

f'_m = 3500 psi
f_b = 1167 psi
E_m = 3,150,000 psi
n = 9.2
K_{fb} = 161.8
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi
k = 0.309
ρ_b = 0.0075



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\,(\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\,(\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		161.8	180	190	200	210	220	230	240	250	260
0.02	ρ'	—	0.0005	0.0008	0.0010	0.0013	0.0016	0.0018	0.0021	0.0024	0.0026
	ρ	0.0075	0.0083	0.0087	0.0091	0.0096	0.0100	0.0104	0.0108	0.0113	0.0117
0.04	ρ'	—	0.0005	0.0008	0.0011	0.0014	0.0017	0.0020	0.0023	0.0026	0.0029
	ρ	0.0075	0.0083	0.0087	0.0092	0.0096	0.0100	0.0105	0.0109	0.0113	0.0118
0.06	ρ'	—	0.0006	0.0009	0.0012	0.0016	0.0019	0.0022	0.0025	0.0029	0.0032
	ρ	0.0075	0.0083	0.0088	0.0092	0.0097	0.0101	0.0105	0.0110	0.0114	0.0119
0.08	ρ'	—	0.0007	0.0010	0.0014	0.0017	0.0021	0.0025	0.0028	0.0032	0.0035
	ρ	0.0075	0.0083	0.0088	0.0092	0.0097	0.0102	0.0106	0.0111	0.0115	0.0120
0.10	ρ'	—	0.0007	0.0011	0.0015	0.0019	0.0024	0.0028	0.0032	0.0036	0.0040
	ρ	0.0075	0.0084	0.0088	0.0093	0.0097	0.0102	0.0107	0.0111	0.0116	0.0121
0.12	ρ'	—	0.0008	0.0013	0.0017	0.0022	0.0027	0.0031	0.0036	0.0040	0.0045
	ρ	0.0075	0.0084	0.0089	0.0093	0.0098	0.0103	0.0107	0.0112	0.0117	0.0122
0.14	ρ'	—	0.0010	0.0015	0.0020	0.0025	0.0030	0.0036	0.0041	0.0046	0.0051
	ρ	0.0075	0.0084	0.0089	0.0094	0.0099	0.0103	0.0108	0.0113	0.0118	0.0123
0.16	ρ'	—	0.0011	0.0017	0.0023	0.0029	0.0035	0.0041	0.0048	0.0054	0.0060
	ρ	0.0075	0.0084	0.0089	0.0094	0.0099	0.0104	0.0109	0.0114	0.0119	0.0124
0.18	ρ'	—	0.0013	0.0020	0.0027	0.0035	0.0042	0.0049	0.0056	0.0063	0.0071
	ρ	0.0075	0.0084	0.0089	0.0095	0.0100	0.0105	0.0110	0.0115	0.0120	0.0125
0.20	ρ'	—	0.0016	0.0025	0.0033	0.0042	0.0051	0.0059	0.0068	0.0077	0.0086
	ρ	0.0075	0.0085	0.0090	0.0095	0.0100	0.0105	0.0111	0.0116	0.0121	0.0126
0.22	ρ'	—	0.0020	0.0031	0.0042	0.0053	0.0064	0.0075	0.0086	0.0097	0.0107
	ρ	0.0075	0.0085	0.0090	0.0096	0.0101	0.0106	0.0112	0.0117	0.0122	0.0128
0.24	ρ'	—	0.0026	0.0041	0.0055	0.0070	0.0084	0.0099	0.0113	0.0128	0.0142
	ρ	0.0075	0.0085	0.0091	0.0096	0.0102	0.0107	0.0113	0.0118	0.0124	0.0129
0.26	ρ'	—	0.0038	0.0059	0.0080	0.0101	0.0122	0.0143	0.0164	0.0185	0.0205
	ρ	0.0075	0.0085	0.0091	0.0097	0.0102	0.0108	0.0114	0.0119	0.0125	0.0130
0.28	ρ'	—	0.0066	0.0102	0.0139	0.0175	0.0211	0.0247	0.0283	0.0320	0.0356
	ρ	0.0075	0.0086	0.0091	0.0097	0.0103	0.0109	0.0115	0.0120	0.0126	0.0132
0.30	ρ'	—	0.0216	0.0335	0.0453	0.0572	0.0690	0.0809	0.0928	0.1046	0.1165
	ρ	0.0075	0.0086	0.0092	0.0098	0.0104	0.0110	0.0116	0.0122	0.0128	0.0134

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-78b Steel Ratio ρ and ρ' Versus K_f for f'_m = 3,500 psi, (Concrete Masonry)

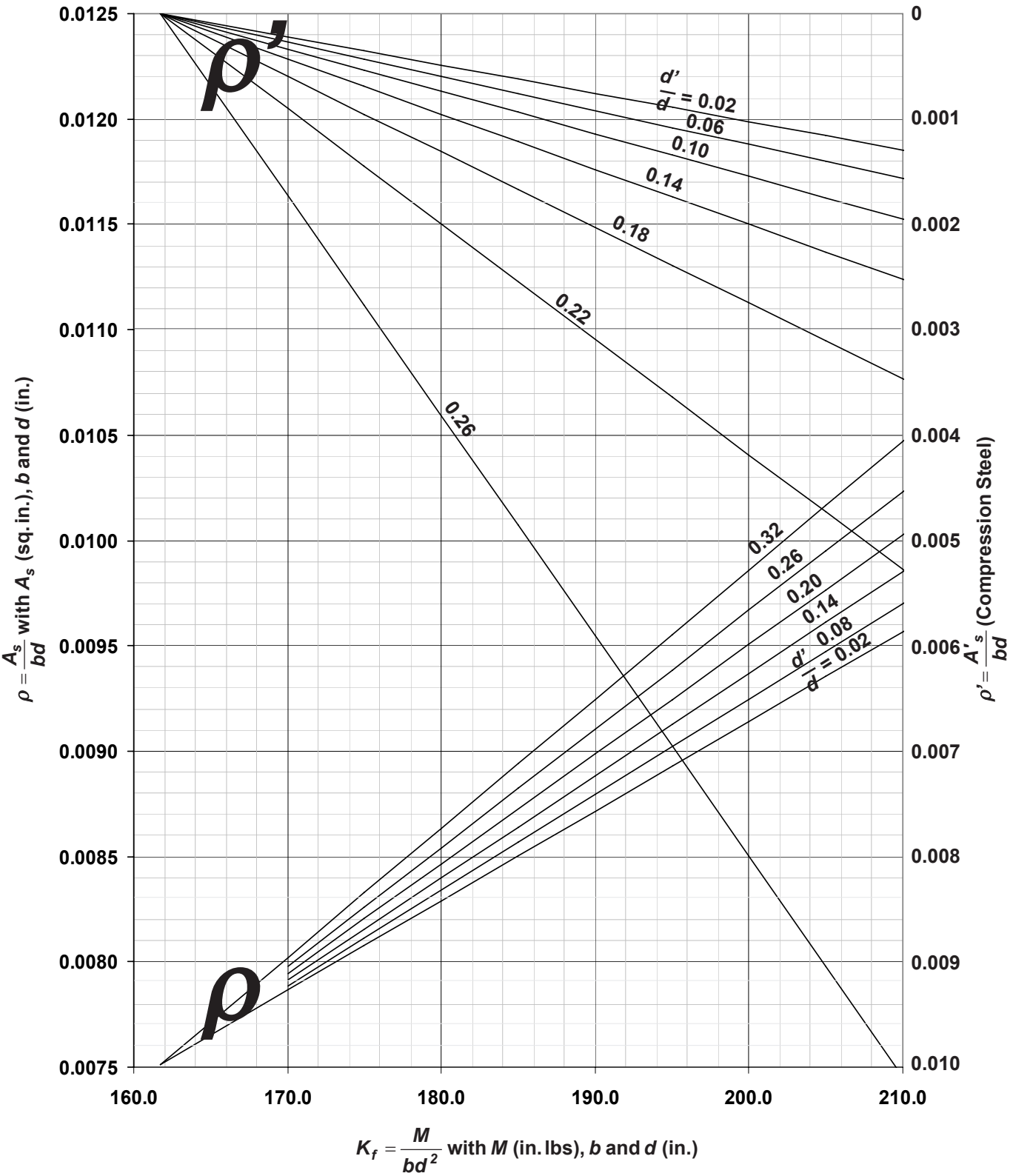
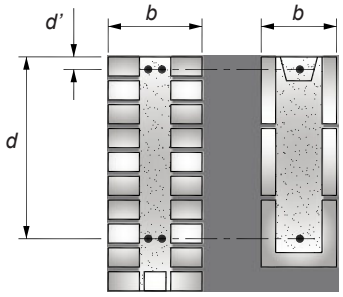


Table ASD-79a Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Clay Masonry) f'_m = 4000 psi, F_s = 24,000 psi, and n = 10.4

DESIGN DATA

f'_m = 4000 psi
f_b = 1333 psi
E_m = 2,800,000 psi
n = 10.4
K_{fb} = 213.8
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi
k = 0.365
ρ_b = 0.0101



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n-1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		213.8	230	240	250	260	270	280	290	300	310
0.02	ρ'	—	0.0003	0.0005	0.0007	0.0009	0.0012	0.0014	0.0016	0.0018	0.0020
	ρ	0.0101	0.0108	0.0113	0.0117	0.0121	0.0125	0.0130	0.0134	0.0138	0.0142
0.04	ρ'	—	0.0004	0.0006	0.0008	0.0010	0.0012	0.0015	0.0017	0.0019	0.0021
	ρ	0.0101	0.0108	0.0113	0.0117	0.0121	0.0126	0.0130	0.0135	0.0139	0.0143
0.06	ρ'	—	0.0004	0.0006	0.0009	0.0011	0.0014	0.0016	0.0018	0.0021	0.0023
	ρ	0.0101	0.0109	0.0113	0.0117	0.0122	0.0126	0.0131	0.0135	0.0140	0.0144
0.08	ρ'	—	0.0004	0.0007	0.0010	0.0012	0.0015	0.0018	0.0020	0.0023	0.0025
	ρ	0.0101	0.0109	0.0113	0.0118	0.0122	0.0127	0.0131	0.0136	0.0140	0.0145
0.10	ρ'	—	0.0005	0.0008	0.0011	0.0013	0.0016	0.0019	0.0022	0.0025	0.0028
	ρ	0.0101	0.0109	0.0114	0.0118	0.0123	0.0127	0.0132	0.0137	0.0141	0.0146
0.12	ρ'	—	0.0005	0.0008	0.0012	0.0015	0.0018	0.0021	0.0025	0.0028	0.0031
	ρ	0.0101	0.0109	0.0114	0.0119	0.0123	0.0128	0.0133	0.0138	0.0142	0.0147
0.14	ρ'	—	0.0006	0.0009	0.0013	0.0017	0.0020	0.0024	0.0027	0.0031	0.0034
	ρ	0.0101	0.0109	0.0114	0.0119	0.0124	0.0129	0.0134	0.0138	0.0143	0.0148
0.16	ρ'	—	0.0007	0.0011	0.0015	0.0019	0.0023	0.0027	0.0031	0.0035	0.0039
	ρ	0.0101	0.0109	0.0114	0.0119	0.0124	0.0129	0.0134	0.0139	0.0144	0.0149
0.18	ρ'	—	0.0007	0.0012	0.0017	0.0021	0.0026	0.0030	0.0035	0.0039	0.0044
	ρ	0.0101	0.0110	0.0115	0.0120	0.0125	0.0130	0.0135	0.0140	0.0145	0.0150
0.20	ρ'	—	0.0008	0.0014	0.0019	0.0024	0.0030	0.0035	0.0040	0.0045	0.0051
	ρ	0.0101	0.0110	0.0115	0.0120	0.0125	0.0131	0.0136	0.0141	0.0146	0.0152
0.22	ρ'	—	0.0010	0.0016	0.0022	0.0028	0.0034	0.0041	0.0047	0.0053	0.0059
	ρ	0.0101	0.0110	0.0115	0.0121	0.0126	0.0131	0.0137	0.0142	0.0147	0.0153
0.24	ρ'	—	0.0012	0.0019	0.0026	0.0034	0.0041	0.0048	0.0056	0.0063	0.0070
	ρ	0.0101	0.0110	0.0116	0.0121	0.0127	0.0132	0.0138	0.0143	0.0149	0.0154
0.26	ρ'	—	0.0014	0.0023	0.0032	0.0041	0.0050	0.0059	0.0068	0.0077	0.0086
	ρ	0.0101	0.0111	0.0116	0.0122	0.0127	0.0133	0.0139	0.0144	0.0150	0.0156
0.28	ρ'	—	0.0018	0.0030	0.0041	0.0052	0.0064	0.0075	0.0086	0.0098	0.0109
	ρ	0.0101	0.0111	0.0117	0.0122	0.0128	0.0134	0.0140	0.0146	0.0151	0.0157
0.30	ρ'	—	0.0025	0.0040	0.0055	0.0070	0.0085	0.0101	0.0116	0.0131	0.0146
	ρ	0.0101	0.0111	0.0117	0.0123	0.0129	0.0135	0.0141	0.0147	0.0153	0.0159

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-79a Steel Ratio ρ and ρ' Versus K_f for f'_m = 4,000 psi, (Clay Masonry)

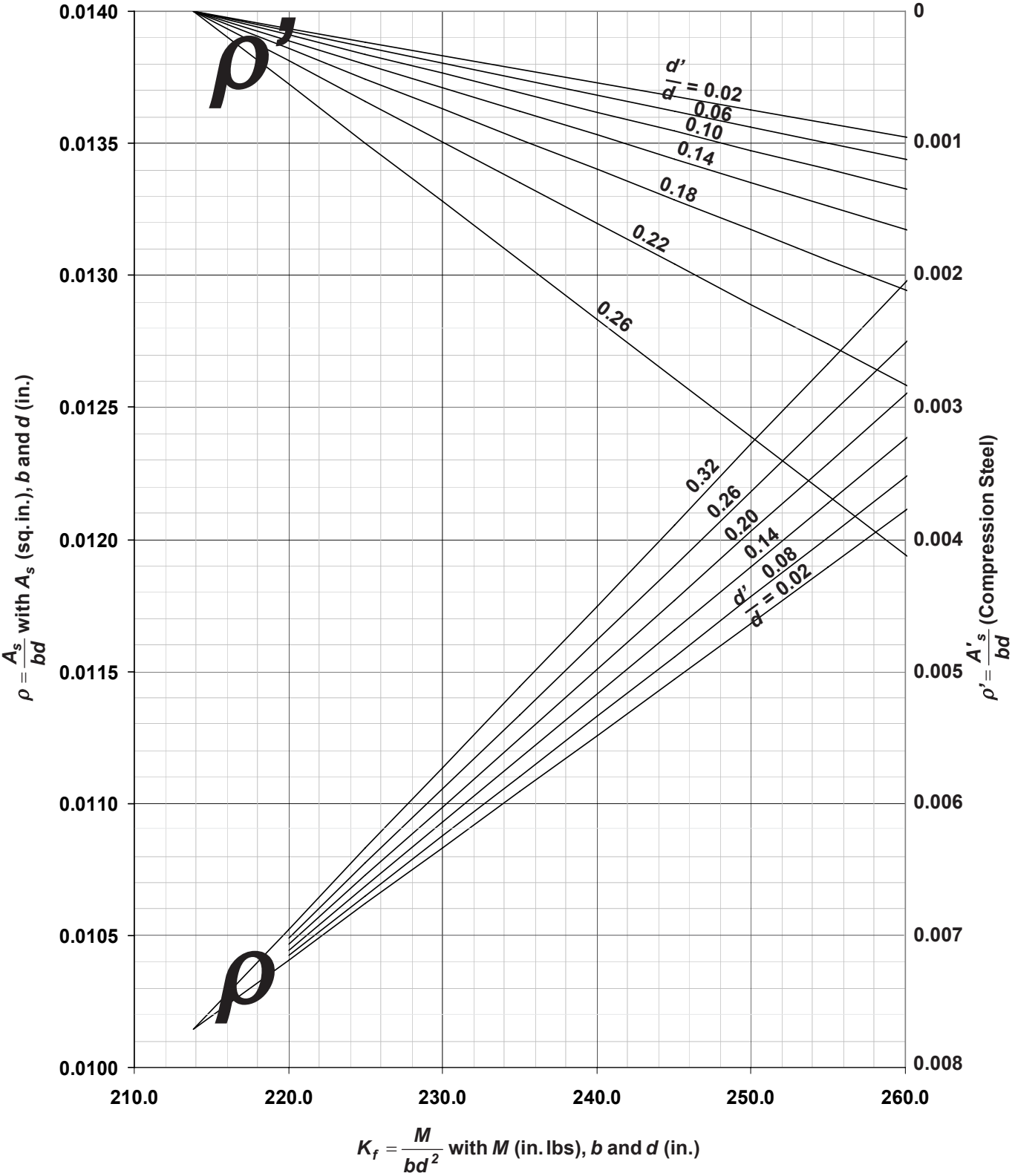
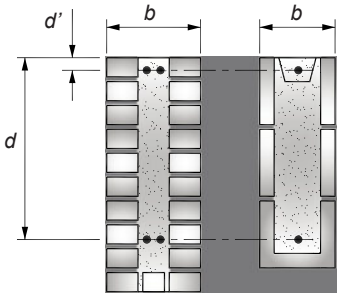


Table ASD-79b Coefficients ρ and ρ' for Tension and Compression Steel in a Flexural Member (Concrete Masonry) f'_m = 4000 psi, F_s = 24,000 psi, and n = 8.1

DESIGN DATA

f'_m = 4000 psi
f_b = 1333 psi
E_m = 3,600,000 psi
n = 8.1
K_{fb} = 184.9
f_y = 60,000 psi
F_s = 24,000 psi
E_s = 29,000,000 psi
k = 0.309
ρ_b = 0.0086



DESIGN EQUATIONS

$$K_f^1 = \frac{M}{F} = \frac{M\ (\text{ft kips})}{bd^2/12{,}000} \text{ or } \frac{M\ (\text{in. lbs})}{bd^2}$$

$$\rho = \rho_b + \frac{K_f - K_{fb}}{F_s(1 - d'/d)}$$

$$\rho' = \frac{K_f - K_{fb}}{(2n - 1)\left[\frac{k - d'/d}{k}\right]\left[1 - \frac{d'}{d}\right]2F_b}$$

d'/d[2]	Steel Ratio ρ', ρ	K_{fb}	K_f[1]								
		184.9	200	210	220	230	240	250	260	270	280
0.02	ρ'	—	0.0004	0.0007	0.0010	0.0012	0.0015	0.0018	0.0020	0.0023	0.0026
	ρ	0.0086	0.0092	0.0097	0.0101	0.0105	0.0109	0.0114	0.0118	0.0122	0.0126
0.04	ρ'	—	0.0004	0.0007	0.0010	0.0013	0.0016	0.0019	0.0022	0.0025	0.0028
	ρ	0.0086	0.0092	0.0097	0.0101	0.0105	0.0110	0.0114	0.0118	0.0123	0.0127
0.06	ρ'	—	0.0005	0.0008	0.0012	0.0015	0.0018	0.0021	0.0025	0.0028	0.0031
	ρ	0.0086	0.0093	0.0097	0.0101	0.0106	0.0110	0.0115	0.0119	0.0124	0.0128
0.08	ρ'	—	0.0006	0.0009	0.0013	0.0016	0.0020	0.0024	0.0027	0.0031	0.0035
	ρ	0.0086	0.0093	0.0097	0.0102	0.0106	0.0111	0.0115	0.0120	0.0124	0.0129
0.10	ρ'	—	0.0006	0.0010	0.0014	0.0018	0.0022	0.0027	0.0031	0.0035	0.0039
	ρ	0.0086	0.0093	0.0098	0.0102	0.0107	0.0111	0.0116	0.0121	0.0125	0.0130
0.12	ρ'	—	0.0007	0.0012	0.0016	0.0021	0.0025	0.0030	0.0035	0.0039	0.0044
	ρ	0.0086	0.0093	0.0098	0.0103	0.0107	0.0112	0.0117	0.0121	0.0126	0.0131
0.14	ρ'	—	0.0008	0.0013	0.0019	0.0024	0.0029	0.0034	0.0040	0.0045	0.0050
	ρ	0.0086	0.0093	0.0098	0.0103	0.0108	0.0113	0.0117	0.0122	0.0127	0.0132
0.16	ρ'	—	0.0009	0.0015	0.0022	0.0028	0.0034	0.0040	0.0046	0.0052	0.0058
	ρ	0.0086	0.0093	0.0098	0.0103	0.0108	0.0113	0.0118	0.0123	0.0128	0.0133
0.18	ρ'	—	0.0011	0.0018	0.0025	0.0033	0.0040	0.0047	0.0054	0.0062	0.0069
	ρ	0.0086	0.0094	0.0099	0.0104	0.0109	0.0114	0.0119	0.0124	0.0129	0.0134
0.20	ρ'	—	0.0013	0.0022	0.0031	0.0040	0.0048	0.0057	0.0066	0.0075	0.0084
	ρ	0.0086	0.0094	0.0099	0.0104	0.0109	0.0115	0.0120	0.0125	0.0130	0.0135
0.22	ρ'	—	0.0017	0.0028	0.0039	0.0050	0.0061	0.0072	0.0083	0.0094	0.0105
	ρ	0.0086	0.0094	0.0099	0.0105	0.0110	0.0115	0.0121	0.0126	0.0131	0.0137
0.24	ρ'	—	0.0022	0.0037	0.0051	0.0066	0.0080	0.0095	0.0110	0.0124	0.0139
	ρ	0.0086	0.0094	0.0100	0.0105	0.0111	0.0116	0.0122	0.0127	0.0133	0.0138
0.26	ρ'	—	0.0032	0.0053	0.0074	0.0095	0.0116	0.0137	0.0158	0.0180	0.0201
	ρ	0.0086	0.0094	0.0100	0.0106	0.0111	0.0117	0.0123	0.0128	0.0134	0.0139
0.28	ρ'	—	0.0055	0.0092	0.0128	0.0165	0.0201	0.0238	0.0274	0.0311	0.0348
	ρ	0.0086	0.0095	0.0100	0.0106	0.0112	0.0118	0.0124	0.0129	0.0135	0.0141
0.30	ρ'	—	0.0181	0.0300	0.0420	0.0539	0.0659	0.0779	0.0898	0.1018	0.1137
	ρ	0.0086	0.0095	0.0101	0.0107	0.0113	0.0119	0.0125	0.0131	0.0137	0.0143

1. When K_f is determined where wind or seismic conditions are considered and a 1/3 increase in stress is permitted, multiply the K_f obtained by 3/4 to use this table.
2. For d'/d values greater than 0.24 the effect of the compression steel becomes increasingly negligible.

Diagram ASD-79b Steel Ratio ρ and ρ' Versus K_f for f'_m = 4,000 psi, (Concrete Masonry)

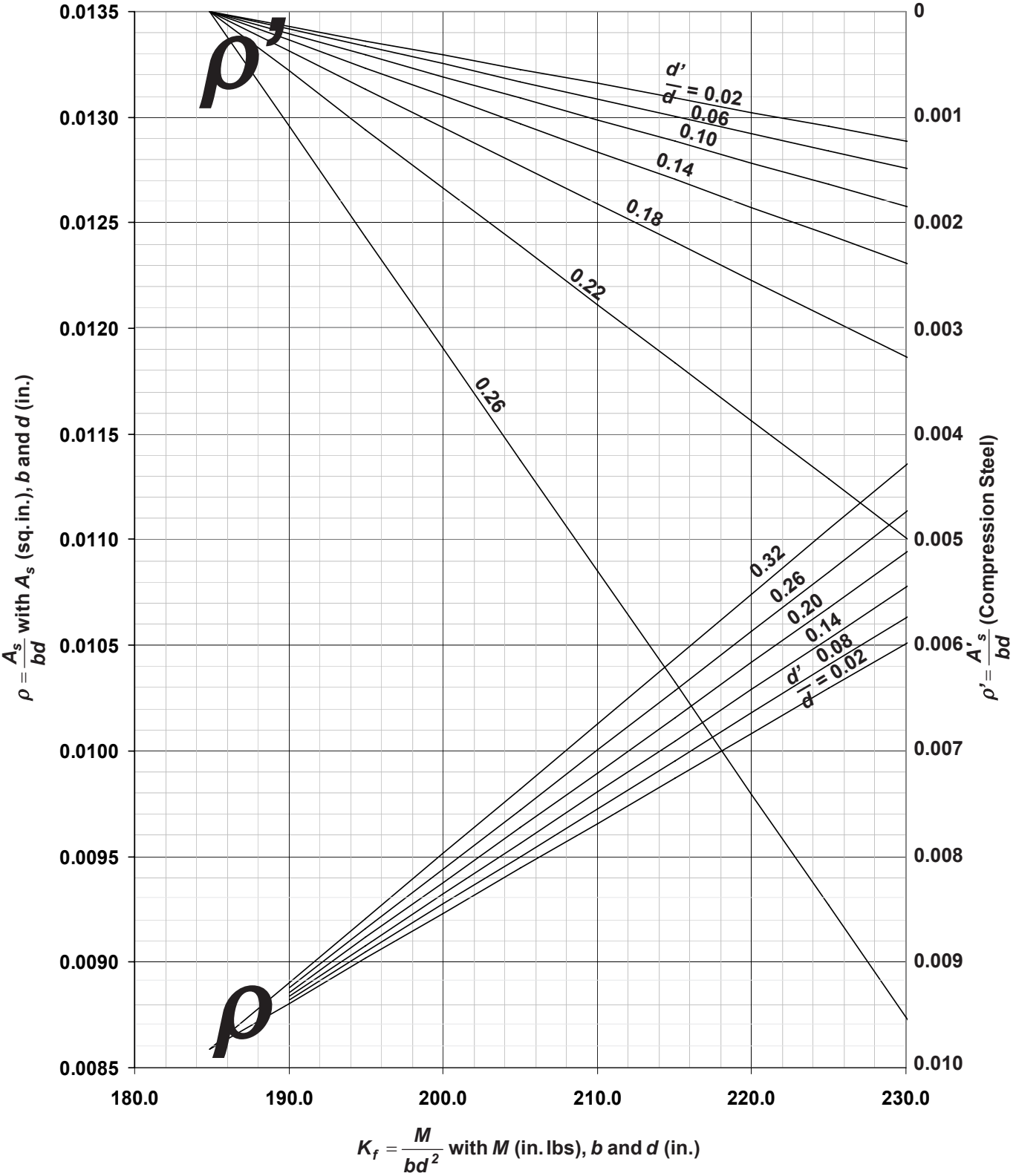
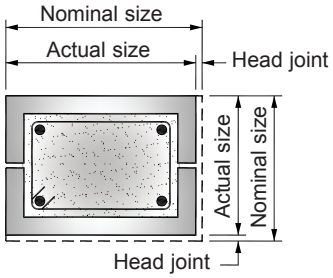


Table ASD-84a Tied Masonry Compression Capacity for Columns Constructed with 3/8" Head Joints[1]

$$P_a = (P_{masonry} + P_{steel})R = \frac{(0.25f'_m A_n + 0.65F_s A_{st})}{1000}R$$

f_y = 60,000 psi

F_s = 24,000 psi



Nominal Column Size (inches)	Effective Area A_n (sq. in.)	$P_{masonry}$ (kips) = (0.25 f'_m A_n)/1000 f'_m (psi)						P_{steel} (kips) = (0.65 $F_s A_{st}$)/1000	
		1500	2000	2500	3000	3500	4000	Min.[2]	Max.[3]
8 x 8	58.1	21.8	29.1	36.3	43.6	50.9	58.1	2.3	36.3
8 x 10	73.4	27.5	36.7	45.9	55.0	64.2	73.4	2.9	45.8
8 x 12	88.6	33.2	44.3	55.4	66.5	77.6	88.6	3.5	55.3
8 x 14	103.9	39.0	51.9	64.9	77.9	90.9	103.9	4.1	64.8
8 x 16	119.1	44.7	59.6	74.5	89.4	104.2	119.1	4.6	74.3
8 x 18	134.4	50.4	67.2	84.0	100.8	117.6	134.4	5.2	83.9
8 x 20	149.6	56.1	74.8	93.5	112.2	130.9	149.6	5.8	93.4
8 x 22	164.9	61.8	82.4	103.1	123.7	144.3	164.9	6.4	102.9
8 x 24	180.1	67.6	90.1	112.6	135.1	157.6	180.1	7.0	112.4
10 x 10	92.6	34.7	46.3	57.9	69.5	81.1	92.6	3.6	57.8
10 x 12	111.9	42.0	55.9	69.9	83.9	97.9	111.9	4.4	69.8
10 x 14	131.1	49.2	65.6	82.0	98.4	114.7	131.1	5.1	81.8
10 x 16	150.4	56.4	75.2	94.0	112.8	131.6	150.4	5.9	93.8
10 x 18	169.6	63.6	84.8	106.0	127.2	148.4	169.6	6.6	105.9
10 x 20	188.9	70.8	94.4	118.1	141.7	165.3	188.9	7.4	117.9
10 x 22	208.1	78.1	104.1	130.1	156.1	182.1	208.1	8.1	129.9
10 x 24	227.4	85.3	113.7	142.1	170.5	199.0	227.4	8.9	141.9
10 x 26	246.6	92.5	123.3	154.2	185.0	215.8	246.6	9.6	153.9
10 x 28	265.9	99.7	132.9	166.2	199.4	232.7	265.9	10.4	165.9
10 x 30	285.1	106.9	142.6	178.2	213.9	249.5	285.1	11.1	177.9
12 x 12	135.1	50.7	67.6	84.5	101.4	118.2	135.1	5.3	84.3
12 x 14	158.4	59.4	79.2	99.0	118.8	138.6	158.4	6.2	98.8
12 x 16	181.6	68.1	90.8	113.5	136.2	158.9	181.6	7.1	113.3
12 x 18	204.9	76.8	102.4	128.1	153.7	179.3	204.9	8.0	127.9
12 x 20	228.1	85.6	114.1	142.6	171.1	199.6	228.1	8.9	142.4
12 x 22	251.4	94.3	125.7	157.1	188.5	220.0	251.4	9.8	156.9
12 x 24	274.6	103.0	137.3	171.1	206.0	240.3	274.6	10.7	171.4
12 x 26	297.9	111.7	148.9	186.2	223.4	260.7	297.9	11.6	185.9
12 x 28	321.1	120.4	160.6	200.7	240.9	281.0	321.1	12.5	200.4
12 x 30	344.4	129.1	172.2	215.2	258.3	301.3	344.4	13.4	214.9
12 x 32	367.6	137.9	183.8	229.8	275.7	321.7	367.6	14.3	229.4
12 x 34	390.9	146.6	195.4	244.3	293.2	342.0	390.9	15.2	243.9
12 x 36	414.1	155.3	207.1	258.8	310.6	362.4	414.1	16.2	258.4
16 x 16	244.1	91.6	122.1	152.6	183.1	213.6	244.1	9.5	152.3
16 x 20	306.6	115.0	153.3	191.7	230.0	268.3	306.6	12.0	191.3
16 x 24	369.1	138.4	184.6	230.7	276.9	323.0	369.1	14.4	230.3
16 x 28	431.6	161.9	215.8	269.8	323.7	377.7	431.6	16.8	269.3
16 x 32	494.1	185.3	247.1	308.8	370.6	432.4	494.1	19.3	308.3
16 x 36	556.6	208.7	278.3	347.9	417.5	487.1	556.6	21.7	347.3
16 x 40	619.1	232.2	309.6	387.0	464.4	541.7	619.1	24.1	386.3
16 x 44	681.6	255.6	340.8	426.0	511.2	596.4	681.6	26.6	425.3
16 x 48	744.1	279.1	372.1	465.1	558.1	651.1	744.1	29.0	464.3

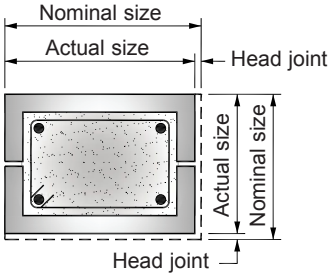
1. Per MSJC Code Section 2.3.3.2.1 Eqs. 2-17 & 2-18, for *R* use Tables ASD-9a, 9b, 9c.
2. Based on MSJC Code Section 2.1.6.4, $\rho_{min.}$ = 0.25%.
3. Based on MSJC Code Section 2.1.6.4, $\rho_{max.}$ = 4.0%.

Table ASD-84b Tied Masonry Compression Capacity for Columns Constructed with 3/8" Head Joints[1]

$$P_a = (P_{masonry} + P_{steel})R = \frac{(0.25f'_m A_n + 0.65F_s A_{st})}{1000} R$$

f_y = 60,000 psi

F_s = 24,000 psi



Nominal Column Size (inches)	Effective Area A_n (sq. in.)	$P_{masonry}$ (kips) = (0.25 f'_m A_n)/1000 f'_m (psi)						P_{steel} (kips) = (0.65 $F_s A_{st}$)/1000	
		1500	2000	2500	3000	3500	4000	Min.[2]	Max.[3]
20 x 20	385.1	144.4	192.6	240.7	288.9	337.0	385.1	15.0	240.3
20 x 24	463.6	173.9	231.8	289.8	347.7	405.7	463.6	18.1	289.3
20 x 28	542.1	203.3	271.1	338.8	406.6	474.4	542.1	21.1	338.3
20 x 32	620.6	232.7	310.3	387.9	465.5	543.1	620.6	24.2	387.3
20 x 36	699.1	262.2	349.6	437.0	524.4	611.7	699.1	27.3	436.3
20 x 40	777.6	291.6	388.8	486.0	583.2	680.4	777.6	30.3	485.2
20 x 44	856.1	321.1	428.1	535.1	642.1	749.1	856.1	33.4	534.2
20 x 48	934.6	350.5	467.3	584.2	701.0	817.8	934.6	36.5	583.2
20 x 52	1013.0	379.9	506.6	633.2	759.9	886.5	1013.0	39.5	632.2
20 x 56	1092.0	409.4	545.8	682.3	818.7	955.2	1092.0	42.6	681.2
20 x 60	1170.0	438.8	585.1	731.3	877.6	1024.0	1170.0	45.6	730.2
24 x 24	558.1	209.3	279.1	348.8	418.6	488.4	558.1	21.8	348.3
24 x 28	652.6	244.7	326.3	407.9	489.5	571.1	652.6	25.5	407.2
24 x 32	747.1	280.2	373.6	467.0	560.4	653.7	747.1	29.1	466.2
24 x 36	841.6	315.6	420.8	526.0	631.2	736.4	841.6	32.8	525.2
24 x 40	936.1	351.1	468.1	585.1	702.1	819.1	936.1	36.5	584.2
24 x 44	1031.0	386.5	515.3	644.2	773.0	901.8	1031.0	40.2	643.1
24 x 48	1125.0	421.9	562.6	703.2	843.9	984.5	1125.0	43.9	702.1
24 x 52	1220.0	457.4	609.8	762.3	914.7	1067.0	1220.0	47.6	761.1
24 x 56	1314.0	492.8	657.1	821.3	985.6	1150.0	1314.0	51.3	820.0
24 x 60	1409.0	528.2	704.3	880.4	1056.0	1233.0	1409.0	54.9	879.0
24 x 64	1503.0	563.7	751.6	939.5	1127.0	1315.0	1503.0	58.6	938.0
24 x 68	1598.0	599.1	798.8	998.5	1198.0	1398.0	1598.0	62.3	996.9
24 x 72	1692.0	634.6	846.1	1058.0	1269.0	1481.0	1692.0	66.0	1056.0
28 x 28	763.1	286.2	381.6	477.0	572.4	667.7	763.1	29.8	476.2
28 x 32	873.6	327.6	436.8	546.0	655.2	764.4	873.6	34.1	545.2
28 x 36	984.1	369.1	492.1	615.1	738.1	861.1	984.1	38.4	614.1
28 x 40	1095.0	410.5	547.3	684.2	821.0	957.8	1095.0	42.7	683.1
28 x 44	1205.0	451.9	602.6	753.2	903.9	1054.0	1205.0	47.0	752.0
28 x 48	1316.0	493.4	657.8	822.3	986.7	1151.0	1316.0	51.3	821.0
28 x 52	1426.0	534.8	713.1	891.3	1070.0	1248.0	1426.0	55.6	889.9
28 x 56	1537.0	576.2	768.3	960.4	1152.0	1345.0	1537.0	59.9	958.9
28 x 60	1647.0	617.7	823.6	1030.0	1235.0	1441.0	1647.0	64.2	1028.0
28 x 64	1758.0	659.1	878.8	1098.0	1318.0	1538.0	1758.0	68.5	1097.0
28 x 68	1868.0	700.6	934.1	1168.0	1401.0	1635.0	1868.0	72.9	1166.0
28 x 72	1979.0	742.0	989.3	1238.0	1484.0	1731.0	1979.0	77.2	1235.0
32 x 32	1000.0	375.1	500.1	625.1	750.1	875.1	1000.0	39.0	624.1
32 x 36	1127.0	422.5	563.3	704.2	845.0	985.8	1127.0	43.9	703.0
32 x 40	1253.0	469.9	626.6	783.2	939.9	1096.0	1253.0	48.9	782.0
32 x 44	1380.0	517.4	689.8	862.3	1035.0	1207.0	1380.0	53.8	860.9
32 x 48	1506.0	564.8	753.1	941.3	1130.0	1318.0	1506.0	58.7	939.8
32 x 52	1633.0	612.2	816.3	1020.0	1224.0	1429.0	1633.0	63.7	1019.0
32 x 56	1759.0	659.7	879.6	1100.0	1319.0	1539.0	1759.0	68.6	1098.0
32 x 60	1886.0	707.1	942.8	1178.0	1414.0	1650.0	1886.0	73.5	1177.0
32 x 64	2012.0	754.6	1006.0	1258.0	1509.0	1761.0	2012.0	78.5	1256.0
32 x 68	2139.0	802.0	1069.0	1337.0	1604.0	1871.0	2139.0	83.4	1334.0
32 x 72	2265.0	849.4	1133.0	1416.0	1699.0	1982.0	2265.0	88.3	1413.0

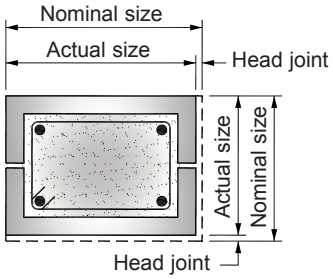
1. Per MSJC Code Section 2.3.3.2.1 Eqs. 2-17 & 2-18, for *R* use Tables ASD-9a, 9b, 9c.
2. Based on MSJC Code Section 2.1.6.4, $\rho_{min.}$ = 0.25%.
3. Based on MSJC Code Section 2.1.6.4, $\rho_{max.}$ = 4.0%.

Table ASD-85a Tied Masonry Compression Capacity for Columns Constructed with 1/2" Head Joints[1]

$$P_a = (P_{masonry} + P_{steel})R = \frac{(0.25f'_m A_n + 0.65F_s A_{st})}{1000}R$$

f_y = 60,000 psi

F_s = 24,000 psi



Nominal Column Size (inches)	Effective Area A_n (sq. in.)	$P_{masonry}$ (kips) = (0.25 f'_m A_n)/1000 f'_m (psi)						P_{steel} (kips) = (0.65 $F_s A_{st}$)/1000	
		1500	2000	2500	3000	3500	4000	Min.[2]	Max.[3]
8 x 8	56.3	21.1	28.1	35.2	42.2	49.2	56.3	2.2	35.1
8 x 10	71.3	26.7	35.6	44.5	53.4	62.3	71.3	2.8	44.5
8 x 12	86.3	32.3	43.1	53.9	64.7	75.5	86.3	3.4	53.8
8 x 14	101.3	38.0	50.6	63.3	75.9	88.6	101.3	3.9	63.2
8 x 16	116.3	43.6	58.1	72.7	87.2	101.7	116.3	4.5	72.5
8 x 18	131.3	49.2	65.6	82.0	98.4	114.8	131.3	5.1	81.9
8 x 20	146.3	54.8	73.1	91.4	109.7	128.0	146.3	5.7	91.3
8 x 22	161.3	60.5	80.6	100.8	120.9	141.1	161.3	6.3	100.6
8 x 24	176.3	66.1	88.1	110.2	132.2	154.2	176.3	6.9	110.0
10 x 10	90.3	33.8	45.1	56.4	67.7	79.0	90.3	3.5	56.3
10 x 12	109.3	41.0	54.6	68.3	81.9	95.6	109.3	4.3	68.2
10 x 14	128.3	48.1	64.1	80.2	96.2	112.2	128.3	5.0	80.0
10 x 16	147.3	55.2	73.5	92.0	110.4	128.8	147.3	5.7	91.9
10 x 18	166.3	62.3	83.1	103.9	124.7	145.5	166.3	6.5	103.7
10 x 20	185.3	69.5	92.6	115.8	138.9	162.1	185.3	7.2	115.6
10 x 22	204.3	76.6	102.1	127.7	153.2	178.7	204.3	8.0	127.5
10 x 24	223.3	83.7	111.6	139.5	167.4	195.3	223.3	8.7	139.3
10 x 26	242.3	90.8	121.1	151.4	181.7	212.0	242.3	9.4	151.2
10 x 28	261.3	98.0	130.6	163.3	195.9	228.6	261.3	10.2	163.0
10 x 30	280.3	105.1	140.1	175.2	210.2	245.2	280.3	10.9	174.9
12 x 12	132.3	49.6	66.1	82.7	99.2	115.7	132.3	5.2	82.5
12 x 14	155.3	58.2	77.6	97.0	116.4	135.8	155.3	6.1	96.9
12 x 16	178.3	66.8	89.1	111.4	133.7	156.0	178.3	7.0	111.2
12 x 18	201.3	75.5	100.6	125.8	150.9	176.1	201.3	7.8	125.6
12 x 20	224.3	84.1	112.1	140.2	168.2	196.2	224.3	8.7	139.9
12 x 22	247.3	92.7	123.6	154.5	185.4	216.3	247.3	9.6	154.3
12 x 24	270.3	101.3	135.1	168.9	202.7	236.5	270.3	10.5	168.6
12 x 26	293.3	110.0	146.6	183.3	219.9	256.6	293.3	11.4	183.0
12 x 28	316.3	118.6	158.1	197.7	237.2	276.7	316.3	12.3	197.3
12 x 30	339.3	127.2	169.6	212.0	254.4	296.8	339.3	13.2	211.7
12 x 32	362.3	135.8	181.1	226.4	271.7	317.0	362.3	14.1	226.0
12 x 34	385.3	144.5	192.6	240.8	288.9	337.1	385.3	15.0	240.4
12 x 36	408.3	153.1	204.1	255.2	306.2	357.2	408.3	15.9	254.7
16 x 16	240.3	90.1	120.1	150.2	180.2	210.2	240.3	9.4	149.9
16 x 20	302.3	113.3	151.1	188.9	226.7	264.5	302.3	11.8	188.6
16 x 24	364.3	136.6	182.1	227.7	273.2	318.7	364.3	14.2	227.3
16 x 28	426.3	159.8	213.1	266.4	319.7	373.0	426.3	16.6	266.0
16 x 32	488.3	183.1	244.1	305.2	366.2	427.2	488.3	19.0	304.7
16 x 36	550.3	206.3	275.1	343.9	412.7	481.5	550.3	21.5	343.4
16 x 40	612.3	229.6	306.1	382.7	459.2	535.7	612.3	23.9	382.0
16 x 44	674.3	252.8	337.1	421.4	505.7	590.0	674.3	26.3	420.7
16 x 48	736.3	276.1	368.1	460.2	552.2	644.2	736.3	28.7	459.4

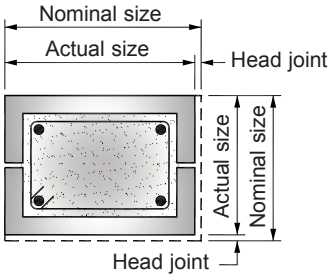
1. Per MSJC Code Section 2.3.3.2.1 Eqs. 2-17 & 2-18, for *R* use Tables ASD-9a, 9b, 9c.
2. Based on MSJC Code Section 2.1.6.4, $\rho_{min.}$ = 0.25%.
3. Based on MSJC Code Section 2.1.6.4, $\rho_{max.}$ = 4.0%.

Table ASD-85b Tied Masonry Compression Capacity for Columns Constructed with 1/2" Head Joints[1]

$$P_a = (P_{masonry} + P_{steel})R = \frac{(0.25f'_m A_n + 0.65F_s A_{st})}{1000} R$$

f_y = 60,000 psi

F_s = 24,000 psi



Nominal Column Size (inches)	Effective Area A_n (sq. in.)	$P_{masonry}$ (kips) = (0.25 f'_m A_n)/1000 f'_m (psi)						P_{steel} (kips) = (0.65 $F_s A_{st}$)/1000	
		1500	2000	2500	3000	3500	4000	Min.[2]	Max.[3]
20 x 20	380.3	142.6	190.1	237.7	285.2	332.7	380.3	14.8	237.3
20 x 24	458.3	171.8	229.1	286.4	343.7	401.0	458.3	17.9	285.9
20 x 28	536.3	201.1	268.1	335.2	402.2	469.2	536.3	20.9	334.6
20 x 32	614.3	230.3	307.1	383.9	460.7	537.5	614.3	24.0	383.3
20 x 36	692.3	259.6	346.1	432.7	519.2	605.7	692.3	27.0	432.0
20 x 40	770.3	288.8	385.1	481.4	577.7	674.0	770.3	30.0	480.6
20 x 44	848.3	318.1	424.1	530.2	636.2	742.2	848.3	33.1	529.3
20 x 48	926.3	347.3	463.1	578.9	694.7	810.5	926.3	36.1	578.0
20 x 52	1004.0	376.6	502.1	627.7	753.2	878.7	1004.0	39.2	626.7
20 x 56	1082.0	405.8	541.1	676.4	811.7	947.0	1082.0	42.2	675.3
20 x 60	1160.0	435.1	580.1	725.2	870.2	1015.0	1160.0	45.2	724.0
24 x 24	552.3	207.1	276.1	345.2	414.2	483.2	552.3	21.5	344.6
24 x 28	646.3	242.3	323.1	403.9	484.7	565.5	646.3	25.2	403.3
24 x 32	740.3	277.6	370.1	462.7	555.2	647.7	740.3	28.9	461.9
24 x 36	834.3	312.8	417.1	521.4	625.7	730.0	834.3	32.5	520.6
24 x 40	928.3	348.1	464.1	580.2	696.2	812.2	928.3	36.2	579.2
24 x 44	1022.0	383.3	511.1	638.9	766.7	894.5	1022.0	39.9	637.9
24 x 48	1116.0	418.6	558.1	697.7	837.2	976.7	1116.0	43.5	696.5
24 x 52	1210.0	453.8	605.1	756.4	907.7	1059.0	1210.0	47.2	755.2
24 x 56	1304.0	489.1	652.1	815.2	978.2	1141.0	1304.0	50.9	813.9
24 x 60	1398.0	524.3	699.1	873.9	1049.0	1224.0	1398.0	54.5	872.5
24 x 64	1492.0	559.6	746.1	932.7	1119.0	1306.0	1492.0	58.2	931.2
24 x 68	1586.0	594.8	793.1	991.4	1190.0	1388.0	1586.0	61.9	989.8
24 x 72	1680.0	630.1	840.1	1050.0	1260.0	1470.0	1680.0	65.5	1048.0
28 x 28	756.3	283.6	378.1	472.7	567.2	661.7	756.3	29.5	471.9
28 x 32	866.3	324.8	433.1	541.4	649.7	758.0	866.3	33.8	540.5
28 x 36	976.3	366.1	488.1	610.2	732.2	854.2	976.3	38.1	609.2
28 x 40	1086.0	407.3	543.1	678.9	814.7	950.5	1086.0	42.4	677.8
28 x 44	1196.0	448.6	598.1	747.7	897.2	1047.0	1196.0	46.7	746.5
28 x 48	1306.0	489.8	653.1	816.4	979.7	1143.0	1306.0	50.9	815.1
28 x 52	1416.0	531.1	708.1	885.2	1062.0	1239.0	1416.0	55.2	883.7
28 x 56	1526.0	572.3	763.1	953.9	1145.0	1336.0	1526.0	59.5	952.4
28 x 60	1636.0	613.6	818.1	1023.0	1227.0	1432.0	1636.0	63.8	1021.0
28 x 64	1746.0	654.8	873.1	1091.0	1310.0	1528.0	1746.0	68.1	1090.0
28 x 68	1856.0	696.1	928.1	1160.0	1392.0	1624.0	1856.0	72.4	1158.0
28 x 72	1966.0	737.3	983.1	1229.0	1475.0	1720.0	1966.0	76.7	1227.0
32 x 32	992.3	372.1	496.1	620.2	744.2	868.2	992.3	38.7	619.2
32 x 36	1118.0	419.3	559.1	698.9	838.7	978.5	1118.0	43.6	697.8
32 x 40	1244.0	466.6	622.1	777.7	933.2	1089.0	1244.0	48.5	776.4
32 x 44	1370.0	513.8	685.1	856.4	1028.0	1199.0	1370.0	53.4	855.0
32 x 48	1496.0	561.1	748.1	935.2	1122.0	1309.0	1496.0	58.4	933.7
32 x 52	1622.0	608.3	811.1	1014.0	1217.0	1420.0	1622.0	63.3	1012.0
32 x 56	1748.0	655.6	874.1	1093.0	1311.0	1530.0	1748.0	68.2	1091.0
32 x 60	1874.0	702.8	937.1	1171.0	1406.0	1640.0	1874.0	73.1	1170.0
32 x 64	2000.0	750.1	1000.0	1250.0	1500.0	1750.0	2000.0	78.0	1248.0
32 x 68	2126.0	797.3	1063.0	1329.0	1595.0	1860.0	2126.0	82.9	1327.0
32 x 72	2252.0	844.6	1126.0	1408.0	1689.0	1971.0	2252.0	87.8	1405.0

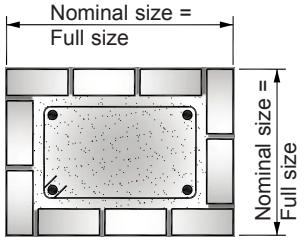
1. Per MSJC Code Section 2.3.3.2.1 Eqs. 2-17 & 2-18, for *R* use Tables ASD-9a, 9b, 9c.
2. Based on MSJC Code Section 2.1.6.4, $\rho_{min.}$ = 0.25%.
3. Based on MSJC Code Section 2.1.6.4, $\rho_{max.}$ = 4.0%.

Table ASD-86a Tied Masonry Compression Capacity for Columns Constructed so that the Nominal Column Dimension Equals the Actual Column Dimension[1]

$$P_a = (P_{masonry} + P_{steel})R = \frac{(0.25f'_m A_n + 0.65F_s A_{st})}{1000}R$$

f_y = 60,000 psi

F_s = 24,000 psi



Nominal Column Size (inches)	Effective Area A_n (sq. in.)	$P_{masonry}$ (kips) = (0.25 f'_m A_n)/1000 f'_m (psi)						P_{steel} (kips) = (0.65 $F_s A_{st}$)/1000	
		1500	2000	2500	3000	3500	4000	Min.[2]	Max.[3]
8 x 8	64	24.0	32.0	40.0	48.0	56.0	64.0	2.5	39.9
8 x 10	80	30.0	40.0	50.0	60.0	70.0	80.0	3.1	49.9
8 x 12	96	36.0	48.0	60.0	72.0	84.0	96.0	3.7	59.9
8 x 14	112	42.0	56.0	70.0	84.0	98.0	112.0	4.4	69.9
8 x 16	128	48.0	64.0	80.0	96.0	112.0	128.0	5.0	79.9
8 x 18	144	54.0	72.0	90.0	108.0	126.0	144.0	5.6	89.9
8 x 20	160	60.0	80.0	100.0	120.0	140.0	160.0	6.2	99.8
8 x 22	176	66.0	88.0	110.0	132.0	154.0	176.0	6.9	109.8
8 x 24	192	72.0	96.0	120.0	144.0	168.0	192.0	7.5	119.8
10 x 10	100	37.5	50.0	62.5	75.0	87.5	100.0	3.9	62.4
10 x 12	120	45.0	60.0	75.0	90.0	105.0	120.0	4.7	74.9
10 x 14	140	52.5	70.0	87.5	105.0	122.5	140.0	5.5	87.4
10 x 16	160	60.0	80.0	100.0	120.0	140.0	160.0	6.2	99.8
10 x 18	180	67.5	90.0	112.5	135.0	157.5	180.0	7.0	112.3
10 x 20	200	75.0	100.0	125.0	150.0	175.0	200.0	7.8	124.8
10 x 22	220	82.5	110.0	137.5	165.0	192.5	220.0	8.6	137.3
10 x 24	240	90.0	120.0	150.0	180.0	210.0	240.0	9.4	149.8
10 x 26	260	97.5	130.0	162.5	195.0	227.5	260.0	10.1	162.2
10 x 28	280	105.0	140.0	175.0	210.0	245.0	280.0	10.9	174.7
10 x 30	300	112.5	150.0	187.5	225.0	262.5	300.0	11.7	187.2
12 x 12	144	54.0	72.0	90.0	108.0	126.0	144.0	5.6	89.9
12 x 14	168	63.0	84.0	105.0	126.0	147.0	168.0	6.6	104.8
12 x 16	192	72.0	96.0	120.0	144.0	168.0	192.0	7.5	119.8
12 x 18	216	81.0	108.0	135.0	162.0	189.0	216.0	8.4	134.8
12 x 20	240	90.0	120.0	150.0	180.0	210.0	240.0	9.4	149.8
12 x 22	264	99.0	132.0	165.0	198.0	231.0	264.0	10.3	164.7
12 x 24	288	108.0	144.0	180.0	216.0	252.0	288.0	11.2	179.7
12 x 26	312	117.0	156.0	195.0	234.0	273.0	312.0	12.2	194.7
12 x 28	336	126.0	168.0	210.0	252.0	294.0	336.0	13.1	209.7
12 x 30	360	135.0	180.0	225.0	270.0	315.0	360.0	14.0	224.6
12 x 32	384	144.0	192.0	240.0	288.0	336.0	384.0	15.0	239.6
12 x 34	408	153.0	204.0	255.0	306.0	357.0	408.0	15.9	254.6
12 x 36	432	162.0	216.0	270.0	324.0	378.0	432.0	16.8	269.6
16 x 16	256	96.0	128.0	160.0	192.0	224.0	256.0	10.0	159.7
16 x 20	320	120.0	160.0	200.0	240.0	280.0	320.0	12.5	199.7
16 x 24	384	144.0	192.0	240.0	288.0	336.0	384.0	15.0	239.6
16 x 28	448	168.0	224.0	280.0	336.0	392.0	448.0	17.5	279.6
16 x 32	512	192.0	256.0	320.0	384.0	448.0	512.0	20.0	319.5
16 x 36	576	216.0	288.0	360.0	432.0	504.0	576.0	22.5	359.4
16 x 40	640	240.0	320.0	400.0	480.0	560.0	640.0	25.0	399.4
16 x 44	704	264.0	352.0	440.0	528.0	616.0	704.0	27.5	439.3
16 x 48	768	288.0	384.0	480.0	576.0	672.0	768.0	30.0	479.2

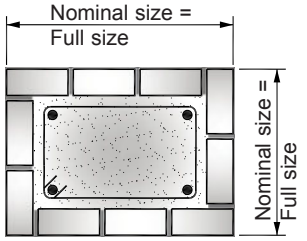
1. Per MSJC Code Section 2.3.3.2.1 Eqs. 2-17 & 2-18, for *R* use Tables ASD-9a, 9b, 9c.
2. Based on MSJC Code Section 2.1.6.4, $\rho_{min.}$ = 0.25%.
3. Based on MSJC Code Section 2.1.6.4, $\rho_{max.}$ = 4.0%.

Table ASD-86b Tied Masonry Compression Capacity for Columns Constructed so that the Nominal Column Dimension Equals the Actual Column Dimension[1]

$$P_a = (P_{masonry} + P_{steel})R = \frac{(0.25f'_m A_n + 0.65F_s A_{st})}{1000} R$$

f_y = 60,000 psi

F_s = 24,000 psi



Nominal Column Size (inches)	Effective Area A_n (sq. in.)	$P_{masonry}$ (kips) = (0.25 f'_m A_n)/1000 f'_m (psi)						P_{steel} (kips) = (0.65 $F_s A_{st}$)/1000	
		1500	2000	2500	3000	3500	4000	Min.[2]	Max.[3]
20 x 20	400.0	150.0	200.0	250.0	300.0	350.0	400.0	15.6	249.6
20 x 24	480.0	180.0	240.0	300.0	360.0	420.0	480.0	18.7	299.5
20 x 28	560.0	210.0	280.0	350.0	420.0	490.0	560.0	21.8	349.4
20 x 32	640.0	240.0	320.0	400.0	480.0	560.0	640.0	25.0	399.4
20 x 36	720.0	270.0	360.0	450.0	540.0	630.0	720.0	28.1	449.3
20 x 40	800.0	300.0	400.0	500.0	600.0	700.0	800.0	31.2	499.2
20 x 44	880.0	330.0	440.0	550.0	660.0	770.0	880.0	34.3	549.1
20 x 48	960.0	360.0	480.0	600.0	720.0	840.0	960.0	37.4	599.0
20 x 52	1040.0	390.0	520.0	650.0	780.0	910.0	1040.0	40.6	649.0
20 x 56	1120.0	420.0	560.0	700.0	840.0	980.0	1120.0	43.7	698.9
20 x 60	1200.0	450.0	600.0	750.0	900.0	1050.0	1200.0	46.8	748.8
24 x 24	576.0	216.0	288.0	360.0	432.0	504.0	576.0	22.5	359.4
24 x 28	672.0	252.0	336.0	420.0	504.0	588.0	672.0	26.2	419.3
24 x 32	768.0	288.0	384.0	480.0	576.0	672.0	768.0	30.0	479.2
24 x 36	864.0	324.0	432.0	540.0	648.0	756.0	864.0	33.7	539.1
24 x 40	960.0	360.0	480.0	600.0	720.0	840.0	960.0	37.4	599.0
24 x 44	1056.0	396.0	528.0	660.0	792.0	924.0	1056.0	41.2	658.9
24 x 48	1152.0	432.0	576.0	720.0	864.0	1008.0	1152.0	44.9	718.8
24 x 52	1248.0	468.0	624.0	780.0	936.0	1092.0	1248.0	48.7	778.8
24 x 56	1344.0	504.0	672.0	840.0	1008.0	1176.0	1344.0	52.4	838.7
24 x 60	1440.0	540.0	720.0	900.0	1080.0	1260.0	1440.0	56.2	898.6
24 x 64	1536.0	576.0	768.0	960.0	1152.0	1344.0	1536.0	59.9	958.5
24 x 68	1632.0	612.0	816.0	1020.0	1224.0	1428.0	1632.0	63.6	1018.0
24 x 72	1728.0	648.0	864.0	1080.0	1296.0	1512.0	1728.0	67.4	1078.0
28 x 28	784.0	294.0	392.0	490.0	588.0	686.0	784.0	30.6	489.2
28 x 32	896.0	336.0	448.0	560.0	672.0	784.0	896.0	34.9	559.1
28 x 36	1008.0	378.0	504.0	630.0	756.0	882.0	1008.0	39.3	629.0
28 x 40	1120.0	420.0	560.0	700.0	840.0	980.0	1120.0	43.7	698.9
28 x 44	1232.0	462.0	616.0	770.0	924.0	1078.0	1232.0	48.0	768.8
28 x 48	1344.0	504.0	672.0	840.0	1008.0	1176.0	1344.0	52.4	838.7
28 x 52	1456.0	546.0	728.0	910.0	1092.0	1274.0	1456.0	56.8	908.5
28 x 56	1568.0	588.0	784.0	980.0	1176.0	1372.0	1568.0	61.2	978.4
28 x 60	1680.0	630.0	840.0	1050.0	1260.0	1470.0	1680.0	65.5	1048.0
28 x 64	1792.0	672.0	896.0	1120.0	1344.0	1568.0	1792.0	69.9	1118.0
28 x 68	1904.0	714.0	952.0	1190.0	1428.0	1666.0	1904.0	74.3	1188.0
28 x 72	2016.0	756.0	1008.0	1260.0	1512.0	1764.0	2016.0	78.6	1258.0
32 x 32	1024.0	384.0	512.0	640.0	768.0	896.0	1024.0	39.9	639.0
32 x 36	1152.0	432.0	576.0	720.0	864.0	1008.0	1152.0	44.9	718.8
32 x 40	1280.0	480.0	640.0	800.0	960.0	1120.0	1280.0	49.9	798.7
32 x 44	1408.0	528.0	704.0	880.0	1056.0	1232.0	1408.0	54.9	878.6
32 x 48	1536.0	576.0	768.0	960.0	1152.0	1344.0	1536.0	59.9	958.5
32 x 52	1664.0	624.0	832.0	1040.0	1248.0	1456.0	1664.0	64.9	1038.0
32 x 56	1792.0	672.0	896.0	1120.0	1344.0	1568.0	1792.0	69.9	1118.0
32 x 60	1920.0	720.0	960.0	1200.0	1440.0	1680.0	1920.0	74.9	1198.0
32 x 64	2048.0	768.0	1024.0	1280.0	1536.0	1792.0	2048.0	79.9	1278.0
32 x 68	2176.0	816.0	1088.0	1360.0	1632.0	1904.0	2176.0	84.9	1358.0
32 x 72	2304.0	864.0	1152.0	1440.0	1728.0	2016.0	2304.0	89.9	1438.0

1. Per MSJC Code Section 2.3.3.2.1 Eqs. 2-17 & 2-18, for *R* use Tables ASD-9a, 9b, 9c.
2. Based on MSJC Code Section 2.1.6.4, $\rho_{min.}$ = 0.25%.
3. Based on MSJC Code Section 2.1.6.4, $\rho_{max.}$ = 4.0%.

Table ASD-87 Capacity of Reinforcing Steel in Tied Masonry Columns (kips)[1]

P_s (kips) = 0.65 F_s A_{st} / 1000 f_y = 60,000 psi

F_s = 24,000 psi

Bar Size[2]	Number of Bars						
	4	6	8	10	12	14	16
#3	6.9	10.3	13.8	17.2	20.7	24.1	27.6
#4	12.3	18.4	24.5	30.6	36.8	42.9	49.0
#5	19.1	28.7	38.3	47.9	57.4	67.0	76.6
#6	27.6	41.4	55.1	68.9	82.7	96.5	110.3
#7	37.5	56.3	75.0	93.8	112.6	131.3	150.1
#8	49.0	73.5	98.0	122.5	147.0	171.5	196.0
#9	62.4	93.5	124.7	155.9	187.1	218.3	249.4
#10	79.0	118.6	158.1	197.6	237.1	276.7	316.2
#11	97.4	146.2	194.9	243.6	292.3	341.0	389.7

1. Note that MSJC Code Table 1.16.1 Footnote 4 limits the maximum area of masonry reinforcement to 6% of the grout space and MSJC Code Section 2.1.6.4 limits masonry reinforcement to a minimum of 0.25% of A_n and a maximum of 4% of A_n
2. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

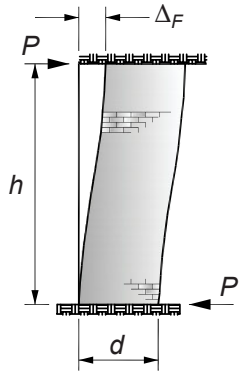
Table ASD-88 Maximum Spacing of Column Ties (inches)[1]

Tie Bar Size	Longitudinal Bar Size[2]								
	#3	#4	#5	#6	#7	#8	#9	#10	#11
1/4"	6.0	8.0	10.0	12.0	12.0	12.0	12.0	12.0	12.0
#3	6.0	8.0	10.0	12.0	14.0	16.0	18.0	18.0	18.0
#4	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.3	22.6
#5	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.3	22.6

1. This table shows the maximum spacing of ties permitted by MSJC Code Section 2.1.6.5 based on 48 times the tie diameter or 16 times the longitudinal bar diameter. The spacing determined from this table may not exceed the least column dimension.
2. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table ASD-89a Coefficients for Deflection and Rigidity of Walls or Piers for Distribution of Horizontal Forces

Fixed Wall or Pier[1]



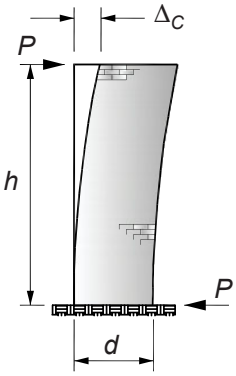
Δ_F = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_F = \frac{1}{\Delta_F}$ Rigidity of fixed wall or pier

Cantilever Wall or Pier[1]



Δ_C = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_C = \frac{1}{\Delta_C}$ Rigidity of cantilever wall or pier

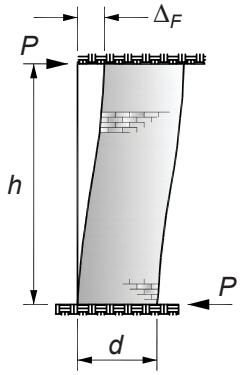
P = 100,000 pounds; t = 1"; E_m = 1,000,000 psi

h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C
0.10	0.030	0.030	33.220	32.900	0.50	0.163	0.200	6.154	5.000	0.90	0.343	0.562	2.916	1.781
0.11	0.033	0.034	30.180	29.820	0.51	0.166	0.206	6.014	4.853	0.91	0.348	0.574	2.871	1.741
0.12	0.036	0.037	27.640	27.250	0.52	0.170	0.212	5.880	4.712	0.92	0.354	0.587	2.826	1.702
0.13	0.039	0.040	25.500	25.080	0.53	0.174	0.219	5.751	4.576	0.93	0.359	0.601	2.782	1.665
0.14	0.042	0.043	23.660	23.200	0.54	0.178	0.225	5.626	4.445	0.94	0.365	0.614	2.739	1.628
0.15	0.045	0.046	22.060	21.580	0.55	0.182	0.232	5.505	4.319	0.95	0.371	0.628	2.697	1.592
0.16	0.048	0.050	20.660	20.150	0.56	0.186	0.238	5.389	4.197	0.96	0.376	0.642	2.656	1.558
0.17	0.051	0.053	19.420	18.880	0.57	0.190	0.245	5.277	4.080	0.97	0.382	0.656	2.616	1.524
0.18	0.055	0.056	18.320	17.750	0.58	0.194	0.252	5.168	3.968	0.98	0.388	0.670	2.577	1.491
0.19	0.058	0.060	17.340	16.740	0.59	0.198	0.259	5.062	3.859	0.99	0.394	0.685	2.538	1.460
0.20	0.061	0.063	16.450	15.820	0.60	0.202	0.266	4.960	3.754	1.00	0.400	0.700	2.500	1.429
0.21	0.064	0.067	15.640	14.990	0.61	0.206	0.274	4.861	3.652	1.01	0.406	0.715	2.463	1.398
0.22	0.067	0.070	14.910	14.230	0.62	0.210	0.281	4.766	3.555	1.02	0.412	0.730	2.426	1.369
0.23	0.070	0.074	14.240	13.540	0.63	0.214	0.289	4.673	3.460	1.03	0.418	0.746	2.391	1.340
0.24	0.073	0.078	13.630	12.900	0.64	0.218	0.297	4.583	3.369	1.04	0.424	0.762	2.356	1.312
0.25	0.077	0.081	13.060	12.310	0.65	0.222	0.305	4.495	3.280	1.05	0.431	0.778	2.321	1.285
0.26	0.080	0.085	12.540	11.760	0.66	0.227	0.313	4.410	3.195	1.06	0.437	0.794	2.288	1.259
0.27	0.083	0.089	12.050	11.250	0.67	0.231	0.321	4.328	3.112	1.07	0.444	0.811	2.255	1.233
0.28	0.086	0.093	11.600	10.780	0.68	0.235	0.330	4.247	3.032	1.08	0.450	0.828	2.222	1.208
0.29	0.089	0.097	11.180	10.340	0.69	0.240	0.338	4.169	2.955	1.09	0.457	0.845	2.191	1.183
0.30	0.093	0.101	10.790	9.921	0.70	0.244	0.347	4.093	2.880	1.10	0.463	0.862	2.159	1.160
0.31	0.096	0.105	10.420	9.531	0.71	0.249	0.356	4.019	2.808	1.11	0.470	0.880	2.129	1.136
0.32	0.099	0.109	10.070	9.165	0.72	0.253	0.365	3.948	2.737	1.12	0.476	0.898	2.099	1.114
0.33	0.103	0.113	9.747	8.820	0.73	0.258	0.375	3.877	2.669	1.13	0.483	0.916	2.069	1.092
0.34	0.106	0.118	9.440	8.495	0.74	0.263	0.384	3.809	2.604	1.14	0.490	0.935	2.040	1.070
0.35	0.109	0.122	9.150	8.187	0.75	0.267	0.394	3.743	2.540	1.15	0.497	0.953	2.012	1.049
0.36	0.113	0.127	8.876	7.895	0.76	0.272	0.404	3.678	2.478	1.16	0.504	0.972	1.984	1.028
0.37	0.116	0.131	8.616	7.618	0.77	0.277	0.414	3.615	2.418	1.17	0.511	0.992	1.956	1.008
0.38	0.119	0.136	8.369	7.356	0.78	0.281	0.424	3.553	2.359	1.18	0.518	1.011	1.929	0.989
0.39	0.123	0.141	8.135	7.106	0.79	0.286	0.434	3.493	2.303	1.19	0.526	1.031	1.903	0.970
0.40	0.126	0.146	7.911	6.868	0.80	0.291	0.445	3.434	2.248	1.20	0.533	1.051	1.877	0.951
0.41	0.130	0.151	7.699	6.641	0.81	0.296	0.456	3.377	2.195	1.21	0.540	1.072	1.851	0.933
0.42	0.133	0.156	7.496	6.425	0.82	0.301	0.467	3.321	2.143	1.22	0.548	1.092	1.826	0.915
0.43	0.137	0.161	7.302	6.219	0.83	0.306	0.478	3.266	2.093	1.23	0.555	1.113	1.802	0.898
0.44	0.141	0.166	7.117	6.021	0.84	0.311	0.489	3.213	2.045	1.24	0.563	1.135	1.777	0.881
0.45	0.144	0.171	6.939	5.833	0.85	0.316	0.501	3.160	1.997	1.25	0.570	1.156	1.753	0.865
0.46	0.148	0.177	6.769	5.652	0.86	0.322	0.512	3.109	1.952	1.26	0.578	1.178	1.730	0.849
0.47	0.151	0.183	6.606	5.479	0.87	0.327	0.524	3.060	1.907	1.27	0.586	1.200	1.707	0.833
0.48	0.155	0.188	6.449	5.312	0.88	0.332	0.537	3.011	1.864	1.28	0.594	1.223	1.684	0.818
0.49	0.159	0.194	6.299	5.153	0.89	0.337	0.549	2.963	1.822	1.29	0.602	1.246	1.662	0.803

1. Piers are limited to an *h/d* ratio of 5 or less.
2. Based on a Shear Modulus $E_v = 0.4\ E_m$.

Table ASD-89b Coefficients for Deflection and Rigidity of Walls or Piers for Distribution of Horizontal Forces

Fixed Wall or Pier[1]



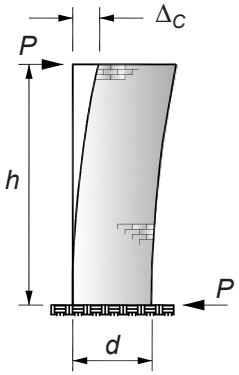
Δ_F = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$$R_F = \frac{1}{\Delta_F}$$ Rigidity of fixed wall or pier

Cantilever Wall or Pier[1]



Δ_C = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$$R_C = \frac{1}{\Delta_C}$$ Rigidity of cantilever wall or pier

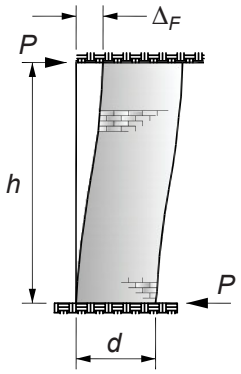
P = 100,000 pounds; t = 1"; E_m = 1,000,000 psi

h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C
1.30	0.610	1.269	1.640	0.788	1.70	1.001	2.475	0.999	0.404	2.10	1.556	4.334	0.643	0.231
1.31	0.618	1.292	1.619	0.774	1.71	1.013	2.513	0.987	0.398	2.11	1.572	4.391	0.636	0.228
1.32	0.626	1.316	1.597	0.760	1.72	1.025	2.551	0.976	0.392	2.12	1.589	4.447	0.629	0.225
1.33	0.634	1.340	1.577	0.746	1.73	1.037	2.590	0.965	0.386	2.13	1.605	4.504	0.623	0.222
1.34	0.643	1.364	1.556	0.733	1.74	1.049	2.629	0.953	0.380	2.14	1.622	4.562	0.617	0.219
1.35	0.651	1.389	1.536	0.720	1.75	1.061	2.669	0.943	0.375	2.15	1.639	4.620	0.610	0.216
1.36	0.660	1.414	1.516	0.707	1.76	1.073	2.709	0.932	0.369	2.16	1.656	4.679	0.604	0.214
1.37	0.668	1.440	1.497	0.695	1.77	1.086	2.749	0.921	0.364	2.17	1.673	4.738	0.598	0.211
1.38	0.677	1.465	1.478	0.682	1.78	1.098	2.790	0.911	0.358	2.18	1.690	4.798	0.592	0.208
1.39	0.686	1.491	1.459	0.671	1.79	1.111	2.831	0.900	0.353	2.19	1.707	4.858	0.586	0.206
1.40	0.694	1.518	1.440	0.659	1.80	1.123	2.873	0.890	0.348	2.20	1.725	4.919	0.580	0.203
1.41	0.703	1.544	1.422	0.648	1.81	1.136	2.915	0.880	0.343	2.21	1.742	4.981	0.574	0.201
1.42	0.712	1.571	1.404	0.636	1.82	1.149	2.957	0.870	0.338	2.22	1.760	5.042	0.568	0.198
1.43	0.721	1.599	1.386	0.626	1.83	1.162	3.000	0.861	0.333	2.23	1.778	5.105	0.562	0.196
1.44	0.731	1.626	1.369	0.615	1.84	1.175	3.044	0.851	0.329	2.24	1.796	5.168	0.557	0.194
1.45	0.740	1.654	1.352	0.604	1.85	1.188	3.088	0.842	0.324	2.25	1.814	5.231	0.551	0.191
1.46	0.749	1.683	1.335	0.594	1.86	1.201	3.132	0.832	0.319	2.26	1.832	5.295	0.546	0.189
1.47	0.759	1.712	1.318	0.584	1.87	1.215	3.177	0.823	0.315	2.27	1.851	5.360	0.540	0.187
1.48	0.768	1.741	1.302	0.574	1.88	1.228	3.222	0.814	0.310	2.28	1.869	5.425	0.535	0.184
1.49	0.778	1.770	1.286	0.565	1.89	1.242	3.268	0.805	0.306	2.29	1.888	5.491	0.530	0.182
1.50	0.788	1.800	1.270	0.556	1.90	1.256	3.314	0.796	0.302	2.30	1.907	5.557	0.524	0.180
1.51	0.797	1.830	1.254	0.546	1.91	1.270	3.360	0.788	0.298	2.31	1.926	5.624	0.519	0.178
1.52	0.807	1.861	1.239	0.537	1.92	1.284	3.407	0.779	0.293	2.32	1.945	5.691	0.514	0.176
1.53	0.817	1.892	1.224	0.529	1.93	1.298	3.455	0.770	0.289	2.33	1.964	5.759	0.509	0.174
1.54	0.827	1.923	1.209	0.520	1.94	1.312	3.503	0.762	0.286	2.34	1.983	5.827	0.504	0.172
1.55	0.837	1.955	1.194	0.512	1.95	1.326	3.551	0.754	0.282	2.35	2.003	5.896	0.499	0.170
1.56	0.848	1.987	1.180	0.503	1.96	1.341	3.600	0.746	0.278	2.36	2.022	5.966	0.494	0.168
1.57	0.858	2.019	1.166	0.495	1.97	1.356	3.649	0.738	0.274	2.37	2.042	6.036	0.490	0.166
1.58	0.868	2.052	1.152	0.487	1.98	1.370	3.699	0.730	0.270	2.38	2.062	6.107	0.485	0.164
1.59	0.879	2.085	1.138	0.480	1.99	1.385	3.749	0.722	0.267	2.39	2.082	6.178	0.480	0.162
1.60	0.890	2.118	1.124	0.472	2.00	1.400	3.800	0.714	0.263	2.40	2.102	6.250	0.476	0.160
1.61	0.900	2.152	1.111	0.465	2.01	1.415	3.851	0.707	0.260	2.41	2.123	6.322	0.471	0.158
1.62	0.911	2.187	1.098	0.457	2.02	1.430	3.903	0.699	0.256	2.42	2.143	6.395	0.467	0.156
1.63	0.922	2.221	1.085	0.450	2.03	1.446	3.955	0.692	0.253	2.43	2.164	6.469	0.462	0.155
1.64	0.933	2.256	1.072	0.443	2.04	1.461	4.008	0.684	0.250	2.44	2.185	6.543	0.458	0.153
1.65	0.944	2.292	1.059	0.436	2.05	1.477	4.061	0.677	0.246	2.45	2.206	6.617	0.453	0.151
1.66	0.955	2.328	1.047	0.430	2.06	1.492	4.115	0.670	0.243	2.46	2.227	6.693	0.449	0.149
1.67	0.967	2.364	1.034	0.423	2.07	1.508	4.169	0.663	0.240	2.47	2.248	6.769	0.445	0.148
1.68	0.978	2.401	1.022	0.417	2.08	1.524	4.224	0.656	0.237	2.48	2.269	6.845	0.441	0.146
1.69	0.990	2.438	1.010	0.410	2.09	1.540	4.279	0.649	0.234	2.49	2.291	6.922	0.437	0.144

1. Piers are limited to an *h/d* ratio of 5 or less
2. Based on a Shear Modulus $E_v = 0.4\ E_m$

Table ASD-89c Coefficients for Deflection and Rigidity of Walls or Piers for Distribution of Horizontal Forces

Fixed Wall or Pier[1]



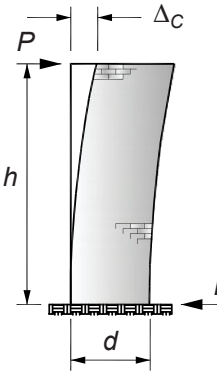
Δ_F = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_F = \frac{1}{\Delta_F}$ Rigidity of fixed wall or pier

Cantilever Wall or Pier[1]



Δ_C = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_C = \frac{1}{\Delta_C}$ Rigidity of cantilever wall or pier

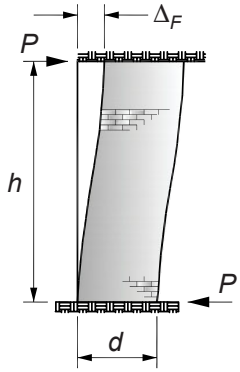
P = 100,000 pounds; t = 1"; E_m = 1,000,000 psi

h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C
2.50	2.313	7.000	0.432	0.143	2.90	3.309	10.63	0.302	0.094	3.30	4.584	15.36	0.218	0.065
2.51	2.334	7.078	0.428	0.141	2.91	3.337	10.73	0.300	0.093	3.31	4.619	15.50	0.216	0.065
2.52	2.356	7.157	0.424	0.140	2.92	3.366	10.84	0.297	0.092	3.32	4.655	15.63	0.215	0.064
2.53	2.378	7.237	0.420	0.138	2.93	3.394	10.94	0.295	0.091	3.33	4.692	15.77	0.213	0.063
2.54	2.401	7.317	0.417	0.137	2.94	3.423	11.05	0.292	0.091	3.34	4.728	15.91	0.212	0.063
2.55	2.423	7.398	0.413	0.135	2.95	3.452	11.15	0.290	0.090	3.35	4.765	16.04	0.210	0.062
2.56	2.446	7.479	0.409	0.134	2.96	3.481	11.26	0.287	0.089	3.36	4.801	16.18	0.208	0.062
2.57	2.468	7.561	0.405	0.132	2.97	3.511	11.37	0.285	0.088	3.37	4.838	16.32	0.207	0.061
2.58	2.491	7.643	0.401	0.131	2.98	3.540	11.48	0.282	0.087	3.38	4.875	16.46	0.205	0.061
2.59	2.514	7.727	0.398	0.129	2.99	3.570	11.59	0.280	0.086	3.39	4.913	16.60	0.204	0.060
2.60	2.538	7.810	0.394	0.128	3.00	3.600	11.70	0.278	0.085	3.40	4.950	16.74	0.202	0.060
2.61	2.561	7.895	0.390	0.127	3.01	3.630	11.81	0.275	0.085	3.41	4.988	16.88	0.200	0.059
2.62	2.584	7.980	0.387	0.125	3.02	3.660	11.92	0.273	0.084	3.42	5.026	17.03	0.199	0.059
2.63	2.608	8.066	0.383	0.124	3.03	3.691	12.04	0.271	0.083	3.43	5.064	17.17	0.197	0.058
2.64	2.632	8.152	0.380	0.123	3.04	3.721	12.15	0.269	0.082	3.44	5.103	17.32	0.196	0.058
2.65	2.656	8.239	0.377	0.121	3.05	3.752	12.26	0.267	0.082	3.45	5.141	17.46	0.195	0.057
2.66	2.680	8.326	0.373	0.120	3.06	3.783	12.38	0.264	0.081	3.46	5.180	17.61	0.193	0.057
2.67	2.704	8.415	0.370	0.119	3.07	3.814	12.50	0.262	0.080	3.47	5.219	17.75	0.192	0.056
2.68	2.729	8.504	0.366	0.118	3.08	3.846	12.61	0.260	0.079	3.48	5.258	17.90	0.190	0.056
2.69	2.754	8.593	0.363	0.116	3.09	3.877	12.73	0.258	0.079	3.49	5.298	18.05	0.189	0.055
2.70	2.778	8.683	0.360	0.115	3.10	3.909	12.85	0.256	0.078	3.50	5.337	18.20	0.187	0.055
2.71	2.803	8.774	0.357	0.114	3.11	3.941	12.96	0.254	0.077	3.51	5.377	18.35	0.186	0.054
2.72	2.828	8.865	0.354	0.113	3.12	3.973	13.08	0.252	0.076	3.52	5.417	18.50	0.185	0.054
2.73	2.854	8.958	0.350	0.112	3.13	4.005	13.20	0.250	0.076	3.53	5.458	18.65	0.183	0.054
2.74	2.879	9.050	0.347	0.110	3.14	4.038	13.33	0.248	0.075	3.54	5.498	18.81	0.182	0.053
2.75	2.905	9.144	0.344	0.109	3.15	4.071	13.45	0.246	0.074	3.55	5.539	18.96	0.181	0.053
2.76	2.930	9.238	0.341	0.108	3.16	4.103	13.57	0.244	0.074	3.56	5.580	19.12	0.179	0.052
2.77	2.956	9.333	0.338	0.107	3.17	4.137	13.69	0.242	0.073	3.57	5.621	19.27	0.178	0.052
2.78	2.982	9.428	0.335	0.106	3.18	4.170	13.82	0.240	0.072	3.58	5.662	19.43	0.177	0.051
2.79	3.009	9.524	0.332	0.105	3.19	4.203	13.94	0.238	0.072	3.59	5.704	19.58	0.175	0.051
2.80	3.035	9.621	0.329	0.104	3.20	4.237	14.07	0.236	0.071	3.60	5.746	19.74	0.174	0.051
2.81	3.062	9.718	0.327	0.103	3.21	4.271	14.19	0.234	0.070	3.61	5.788	19.90	0.173	0.050
2.82	3.089	9.816	0.324	0.102	3.22	4.305	14.32	0.232	0.070	3.62	5.830	20.06	0.172	0.050
2.83	3.116	9.915	0.321	0.101	3.23	4.339	14.45	0.230	0.069	3.63	5.872	20.22	0.170	0.049
2.84	3.143	10.020	0.318	0.100	3.24	4.373	14.58	0.229	0.069	3.64	5.915	20.38	0.169	0.049
2.85	3.170	10.120	0.315	0.099	3.25	4.408	14.71	0.227	0.068	3.65	5.958	20.55	0.168	0.049
2.86	3.197	10.220	0.313	0.098	3.26	4.443	14.84	0.225	0.067	3.66	6.001	20.71	0.167	0.048
2.87	3.225	10.320	0.310	0.097	3.27	4.478	14.97	0.223	0.067	3.67	6.044	20.87	0.165	0.048
2.88	3.253	10.420	0.307	0.096	3.28	4.513	15.10	0.222	0.066	3.68	6.088	21.04	0.164	0.048
2.89	3.281	10.520	0.305	0.095	3.29	4.548	15.23	0.220	0.066	3.69	6.131	21.20	0.163	0.047

1. Piers are limited to an *h/d* ratio of 5 or less
2. Based on a Shear Modulus $E_v = 0.4\ E_m$

Table ASD-89d Coefficients for Deflection and Rigidity of Walls or Piers for Distribution of Horizontal Forces

Fixed Wall or Pier[1]



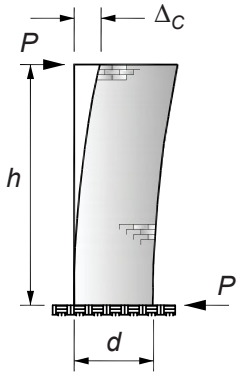
Δ_F = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$$R_F = \frac{1}{\Delta_F}$$ Rigidity of fixed wall or pier

Cantilever Wall or Pier[1]



Δ_C = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$$R_C = \frac{1}{\Delta_C}$$ Rigidity of cantilever wall or pier

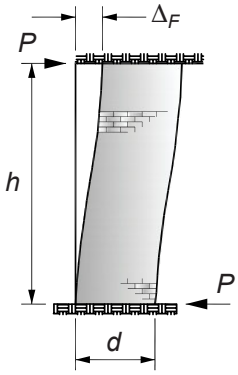
P = 100,000 pounds; t = 1"; E_m = 1,000,000 psi

h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C
3.07	6.175	21.37	0.162	0.047	4.10	8.122	28.80	0.123	0.035	4.50	10.46	37.80	0.096	0.026
3.71	6.219	21.54	0.161	0.046	4.11	8.176	29.00	0.122	0.034	4.51	10.53	38.05	0.095	0.026
3.72	6.264	21.71	0.160	0.046	4.12	8.229	29.21	0.122	0.034	4.52	10.59	38.29	0.094	0.026
3.73	6.309	21.88	0.159	0.046	4.13	8.283	29.42	0.121	0.034	4.53	10.65	38.54	0.094	0.026
3.74	6.353	22.05	0.157	0.045	4.14	8.338	29.63	0.120	0.034	4.54	10.72	38.79	0.093	0.026
3.75	6.398	22.22	0.156	0.045	4.15	8.392	29.83	0.119	0.034	4.55	10.78	39.04	0.093	0.026
3.76	6.444	22.39	0.155	0.045	4.16	8.447	30.04	0.118	0.033	4.56	10.85	39.30	0.092	0.025
3.77	6.489	22.56	0.154	0.044	4.17	8.502	30.26	0.118	0.033	4.57	10.92	39.55	0.092	0.025
3.78	6.535	22.74	0.153	0.044	4.18	8.557	30.47	0.117	0.033	4.58	10.98	39.80	0.091	0.025
3.79	6.581	22.91	0.152	0.044	4.19	8.613	30.68	0.116	0.033	4.59	11.05	40.06	0.091	0.025
3.80	6.627	23.09	0.151	0.043	4.20	8.669	30.90	0.115	0.032	4.60	11.11	40.31	0.090	0.025
3.81	6.674	23.27	0.150	0.043	4.21	8.725	31.11	0.115	0.032	4.61	11.18	40.57	0.089	0.025
3.82	6.720	23.44	0.149	0.043	4.22	8.781	31.33	0.114	0.032	4.62	11.25	40.83	0.089	0.024
3.83	6.767	23.62	0.148	0.042	4.23	8.838	31.54	0.113	0.032	4.63	11.31	41.09	0.088	0.024
3.84	6.814	23.80	0.147	0.042	4.24	8.895	31.76	0.112	0.031	4.64	11.38	41.35	0.088	0.024
3.85	6.862	23.98	0.146	0.042	4.25	8.952	31.98	0.112	0.031	4.65	11.45	41.61	0.087	0.024
3.86	6.909	24.16	0.145	0.041	4.26	9.009	32.20	0.111	0.031	4.66	11.52	41.88	0.087	0.024
3.87	6.957	24.35	0.144	0.041	4.27	9.066	32.42	0.110	0.031	4.67	11.59	42.14	0.086	0.024
3.88	7.005	24.53	0.143	0.041	4.28	9.124	32.65	0.110	0.031	4.68	11.65	42.41	0.086	0.024
3.89	7.053	24.71	0.142	0.040	4.29	9.182	32.87	0.109	0.030	4.69	11.72	42.67	0.085	0.023
3.90	7.102	24.90	0.141	0.040	4.30	9.241	33.09	0.108	0.030	4.70	11.79	42.94	0.085	0.023
3.91	7.151	25.08	0.140	0.040	4.31	9.299	33.32	0.108	0.030	4.71	11.86	43.21	0.084	0.023
3.92	7.200	25.27	0.139	0.040	4.32	9.358	33.54	0.107	0.030	4.72	11.93	43.48	0.084	0.023
3.93	7.249	25.46	0.138	0.039	4.33	9.417	33.77	0.106	0.030	4.73	12.00	43.75	0.083	0.023
3.94	7.298	25.65	0.137	0.039	4.34	9.477	34.00	0.106	0.029	4.74	12.07	44.02	0.083	0.023
3.95	7.348	25.84	0.136	0.039	4.35	9.536	34.23	0.105	0.029	4.75	12.14	44.29	0.082	0.023
3.96	7.398	26.03	0.135	0.038	4.36	9.596	34.46	0.104	0.029	4.76	12.21	44.57	0.082	0.022
3.97	7.448	26.22	0.134	0.038	4.37	9.656	34.69	0.104	0.029	4.77	12.28	44.84	0.081	0.022
3.98	7.498	26.41	0.133	0.038	4.38	9.717	34.93	0.103	0.029	4.78	12.36	45.12	0.081	0.022
3.99	7.549	26.61	0.132	0.038	4.39	9.777	35.16	0.102	0.028	4.79	12.43	45.40	0.080	0.022
4.00	7.600	26.80	0.132	0.037	4.40	9.838	35.39	0.102	0.028	4.80	12.50	45.68	0.080	0.022
4.01	7.651	27.00	0.131	0.037	4.41	9.900	35.63	0.101	0.028	4.81	12.57	45.96	0.080	0.022
4.02	7.702	27.19	0.130	0.037	4.42	9.961	35.87	0.100	0.028	4.82	12.64	46.24	0.079	0.022
4.03	7.754	27.39	0.129	0.037	4.43	10.020	36.10	0.100	0.028	4.83	12.72	46.52	0.079	0.021
4.04	7.806	27.59	0.128	0.036	4.44	10.080	36.34	0.099	0.028	4.84	12.79	46.80	0.078	0.021
4.05	7.858	27.79	0.127	0.036	4.45	10.150	36.58	0.099	0.027	4.85	12.86	47.09	0.078	0.021
4.06	7.910	27.99	0.126	0.036	4.46	10.210	36.82	0.098	0.027	4.86	12.94	47.37	0.077	0.021
4.07	7.963	28.19	0.126	0.035	4.47	10.270	37.07	0.097	0.027	4.87	13.01	47.66	0.077	0.021
4.08	8.016	28.39	0.125	0.035	4.48	10.340	37.31	0.097	0.027	4.88	13.09	47.95	0.076	0.021
4.09	8.069	28.59	0.124	0.035	4.49	10.400	37.55	0.096	0.027	4.89	13.16	48.24	0.076	0.021

1. Piers are limited to an *h/d* ratio of 5 or less
2. Based on a Shear Modulus $E_v = 0.4\ E_m$

Table ASD-89e Coefficients for Deflection and Rigidity of Walls or Piers for Distribution of Horizontal Forces

Fixed Wall or Pier[1]



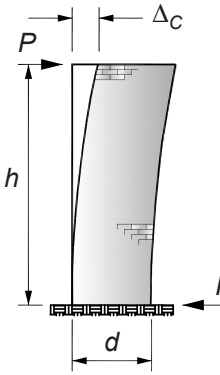
Δ_F = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_F = \frac{1}{\Delta_F}$ Rigidity of fixed wall or pier

Cantilever Wall or Pier[1]



Δ_C = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_C = \frac{1}{\Delta_C}$ Rigidity of cantilever wall or pier

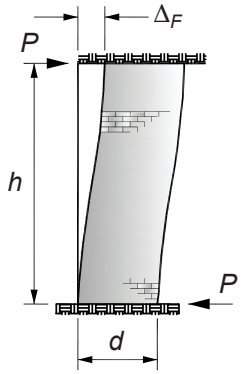
P = 100,000 pounds; t = 1"; E_m = 1,000,000 psi

h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C
4.90	13.23	48.53	0.076	0.021	5.30	16.48	61.14	0.061	0.016	5.70	20.23	75.79	0.049	0.013
4.91	13.31	48.82	0.075	0.020	5.31	16.57	61.48	0.060	0.016	5.71	20.33	76.18	0.049	0.013
4.92	13.39	49.11	0.075	0.020	5.32	16.65	61.82	0.060	0.016	5.72	20.43	76.58	0.049	0.013
4.93	13.46	49.41	0.074	0.020	5.33	16.74	62.17	0.060	0.016	5.73	20.53	76.97	0.049	0.013
4.94	13.54	49.70	0.074	0.020	5.34	16.83	62.51	0.059	0.016	5.74	20.63	77.37	0.048	0.013
4.95	13.61	50.00	0.073	0.020	5.35	16.92	62.86	0.059	0.016	5.75	20.74	77.77	0.048	0.013
4.96	13.69	50.30	0.073	0.020	5.36	17.01	63.20	0.059	0.016	5.76	20.84	78.17	0.048	0.013
4.97	13.77	50.60	0.073	0.020	5.37	17.10	63.55	0.058	0.016	5.77	20.94	78.57	0.048	0.013
4.98	13.84	50.90	0.072	0.020	5.38	17.19	63.90	0.058	0.016	5.78	21.04	78.97	0.048	0.013
4.99	13.92	51.20	0.072	0.020	5.39	17.28	64.25	0.058	0.016	5.79	21.15	79.38	0.047	0.013
5.00	14.00	51.50	0.071	0.019	5.40	17.37	64.61	0.058	0.015	5.80	21.25	79.78	0.047	0.013
5.01	14.08	51.80	0.071	0.019	5.41	17.46	64.96	0.057	0.015	5.81	21.36	80.19	0.047	0.012
5.02	14.16	52.11	0.071	0.019	5.42	17.55	65.31	0.057	0.015	5.82	21.46	80.60	0.047	0.012
5.03	14.24	52.41	0.070	0.019	5.43	17.64	65.67	0.057	0.015	5.83	21.56	81.01	0.046	0.012
5.04	14.31	52.72	0.070	0.019	5.44	17.73	66.03	0.056	0.015	5.84	21.67	81.42	0.046	0.012
5.05	14.39	53.03	0.069	0.019	5.45	17.82	66.39	0.056	0.015	5.85	21.78	81.84	0.046	0.012
5.06	14.47	53.34	0.069	0.019	5.46	17.92	66.75	0.056	0.015	5.86	21.88	82.25	0.046	0.012
5.07	14.55	53.65	0.069	0.019	5.47	18.01	67.11	0.056	0.015	5.87	21.99	82.67	0.045	0.012
5.08	14.63	53.96	0.068	0.019	5.48	18.10	67.47	0.055	0.015	5.88	22.09	83.08	0.045	0.012
5.09	14.71	54.28	0.068	0.018	5.49	18.19	67.83	0.055	0.015	5.89	22.20	83.50	0.045	0.012
5.10	14.80	54.59	0.068	0.018	5.50	18.29	68.20	0.055	0.015	5.90	22.31	83.92	0.045	0.012
5.11	14.88	54.91	0.067	0.018	5.51	18.38	68.57	0.054	0.015	5.91	22.42	84.34	0.045	0.012
5.12	14.96	55.22	0.067	0.018	5.52	18.48	68.93	0.054	0.015	5.92	22.52	84.77	0.044	0.012
5.13	15.04	55.54	0.066	0.018	5.53	18.57	69.30	0.054	0.014	5.93	22.63	85.19	0.044	0.012
5.14	15.12	55.86	0.066	0.018	5.54	18.67	69.67	0.054	0.014	5.94	22.74	85.62	0.044	0.012
5.15	15.20	56.18	0.066	0.018	5.55	18.76	70.05	0.053	0.014	5.95	22.85	86.04	0.044	0.012
5.16	15.29	56.50	0.065	0.018	5.56	18.86	70.42	0.053	0.014	5.96	22.96	86.47	0.044	0.012
5.17	15.37	56.83	0.065	0.018	5.57	18.95	70.79	0.053	0.014	5.97	23.07	86.90	0.043	0.012
5.18	15.45	57.15	0.065	0.017	5.58	19.05	71.17	0.052	0.014	5.98	23.18	87.33	0.043	0.011
5.19	15.54	57.48	0.064	0.017	5.59	19.14	71.55	0.052	0.014	5.99	23.29	87.77	0.043	0.011
5.20	15.62	57.80	0.064	0.017	5.60	19.24	71.93	0.052	0.014	6.00	23.40	88.20	0.043	0.011
5.21	15.71	58.13	0.064	0.017	5.61	19.34	72.31	0.052	0.014	6.01	23.51	88.64	0.043	0.011
5.22	15.79	58.46	0.063	0.017	5.62	19.44	72.69	0.051	0.014	6.02	23.62	89.07	0.042	0.011
5.23	15.87	58.79	0.063	0.017	5.63	19.53	73.07	0.051	0.014	6.03	23.73	89.51	0.042	0.011
5.24	15.96	59.12	0.063	0.017	5.64	19.63	73.45	0.051	0.014	6.04	23.85	89.95	0.042	0.011
5.25	16.05	59.46	0.062	0.017	5.65	19.73	73.84	0.051	0.014	6.05	23.96	90.39	0.042	0.011
5.26	16.13	59.79	0.062	0.017	5.66	19.83	74.23	0.050	0.013	6.06	24.07	90.84	0.042	0.011
5.27	16.22	60.13	0.062	0.017	5.67	19.93	74.61	0.050	0.013	6.07	24.19	91.28	0.041	0.011
5.28	16.30	60.46	0.061	0.017	5.68	20.03	75.00	0.050	0.013	6.08	24.30	91.73	0.041	0.011
5.29	16.39	60.80	0.061	0.016	5.69	20.13	75.40	0.050	0.013	6.09	24.41	92.17	0.041	0.011

1. Piers are limited to an *h/d* ratio of 5 or less
2. Based on a Shear Modulus $E_v = 0.4\ E_m$

Table ASD-89f Coefficients for Deflection and Rigidity of Walls or Piers for Distribution of Horizontal Forces

Fixed Wall or Pier[1]



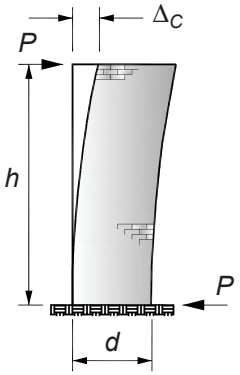
Δ_F = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$$R_F = \frac{1}{\Delta_F}$$ Rigidity of fixed wall or pier

Cantilever Wall or Pier[1]



Δ_C = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$$R_C = \frac{1}{\Delta_C}$$ Rigidity of cantilever wall or pier

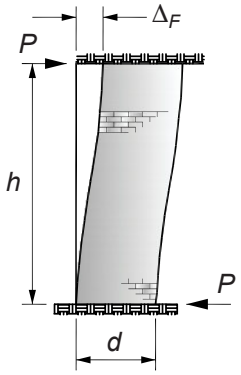
P = 100,000 pounds; t = 1"; E_m = 1,000,000 psi

h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C
6.10	24.53	92.62	0.041	0.011	6.50	29.41	111.8	0.034	0.009	6.90	34.92	133.5	0.029	0.007
6.11	24.64	93.07	0.041	0.011	6.51	29.54	112.3	0.034	0.009	6.91	35.07	134.0	0.029	0.007
6.12	24.76	93.52	0.040	0.011	6.52	29.67	112.8	0.034	0.009	6.92	35.21	134.6	0.028	0.007
6.13	24.87	93.98	0.040	0.011	6.53	29.80	113.3	0.034	0.009	6.93	35.36	135.2	0.028	0.007
6.14	24.99	94.43	0.040	0.011	6.54	29.93	113.9	0.033	0.009	6.94	35.51	135.8	0.028	0.007
6.15	25.11	94.89	0.040	0.011	6.55	30.07	114.4	0.033	0.009	6.95	35.66	136.4	0.028	0.007
6.15	25.22	95.35	0.040	0.0.10	6.56	30.20	114.9	0.033	0.009	6.96	35.80	136.9	0.028	0.007
6.17	25.34	95.81	0.039	0.010	6.57	30.33	115.4	0.033	0.009	6.97	35.95	137.5	0.028	0.007
6.18	25.46	96.27	0.039	0.010	6.58	30.46	115.9	0.033	0.009	6.98	36.10	138.1	0.028	0.007
6.19	25.57	96.73	0.039	0.010	6.59	30.60	116.5	0.033	0.009	6.99	36.25	138.7	0.028	0.007
6.20	25.69	97.19	0.039	0.010	6.60	30.73	117.0	0.033	0.009	7.00	36.40	139.3	0.027	0.007
6.21	25.81	97.66	0.039	0.010	6.61	30.86	117.5	0.032	0.009	7.01	36.55	139.9	0.027	0.007
6.22	25.93	98.12	0.039	0.010	6.62	31.00	118.0	0.032	0.008	7.02	36.70	140.5	0.027	0.007
6.23	26.05	98.59	0.038	0.010	6.63	31.13	118.6	0.032	0.008	7.03	36.85	141.1	0.027	0.007
6.24	26.17	99.06	0.038	0.010	6.64	31.27	119.1	0.032	0.008	7.04	37.00	141.7	0.027	0.007
6.25	26.29	99.53	0.038	0.010	6.65	31.40	119.6	0.032	0.008	7.05	37.16	142.3	0.027	0.007
6.26	26.41	100.00	0.038	0.010	6.66	31.54	120.2	0.032	0.008	7.06	37.31	142.9	0.027	0.007
6.27	26.53	100.50	0.038	0.010	6.67	31.68	120.7	0.032	0.008	7.07	37.46	143.5	0.027	0.007
6.28	26.65	101.00	0.038	0.010	6.68	31.81	121.2	0.031	0.008	7.08	37.61	144.1	0.027	0.007
6.29	26.77	101.40	0.037	0.010	6.69	31.95	121.8	0.031	0.008	7.09	37.77	144.7	0.026	0.007
6.30	26.89	101.90	0.037	0.010	6.70	32.09	122.3	0.031	0.008	7.10	37.92	145.3	0.026	0.007
6.31	27.02	102.40	0.037	0.010	6.71	32.22	122.9	0.031	0.008	7.11	38.08	145.9	0.026	0.007
6.32	27.14	102.90	0.037	0.010	6.72	32.36	123.4	0.031	0.008	7.12	38.23	146.5	0.026	0.007
6.33	27.26	103.40	0.037	0.010	6.73	32.50	123.9	0.031	0.008	7.13	38.39	147.1	0.026	0.007
6.34	27.39	103.80	0.037	0.010	6.74	32.64	124.5	0.031	0.008	7.14	38.54	147.7	0.026	0.007
6.35	27.51	104.30	0.036	0.010	6.75	32.78	125.0	0.031	0.008	7.15	38.70	148.4	0.026	0.007
6.36	27.63	104.80	0.036	0.010	6.76	32.92	125.6	0.030	0.008	7.16	38.85	149.0	0.026	0.007
6.37	27.76	105.30	0.036	0.009	6.77	33.06	126.1	0.030	0.008	7.17	39.01	149.6	0.026	0.007
6.38	27.88	105.80	0.036	0.009	6.78	33.20	126.7	0.030	0.008	7.18	39.17	150.2	0.026	0.007
6.39	28.01	106.30	0.036	0.009	6.79	33.34	127.3	0.030	0.008	7.19	39.33	150.8	0.025	0.007
6.40	28.13	106.80	0.036	0.009	6.80	33.48	127.8	0.030	0.008	7.20	39.48	151.5	0.025	0.007
6.41	28.26	107.30	0.035	0.009	6.81	33.63	128.4	0.030	0.008	7.21	39.64	152.1	0.025	0.007
6.42	28.39	107.80	0.035	0.009	6.82	33.77	128.9	0.030	0.008	7.22	39.80	152.7	0.025	0.007
6.43	28.51	108.30	0.035	0.009	6.83	33.91	129.5	0.029	0.008	7.23	39.96	153.3	0.025	0.007
6.44	28.64	108.80	0.035	0.009	6.84	34.05	130.1	0.029	0.008	7.24	40.12	154.0	0.025	0.006
6.45	28.77	109.30	0.035	0.009	6.85	34.20	130.6	0.029	0.008	7.25	40.28	154.6	0.025	0.006
6.46	28.90	109.80	0.035	0.009	6.86	34.34	131.2	0.029	0.008	7.26	40.44	155.2	0.025	0.006
6.47	29.03	110.30	0.034	0.009	6.87	34.49	131.8	0.029	0.008	7.27	40.61	155.9	0.025	0.006
6.48	29.15	110.80	0.034	0.009	6.88	34.63	132.3	0.029	0.008	7.28	40.77	156.5	0.025	0.006
6.49	29.28	111.30	0.034	0.009	6.89	34.78	132.9	0.029	0.008	7.29	40.93	157.2	0.024	0.006

1. Piers are limited to an *h/d* ratio of 5 or less
2. Based on a Shear Modulus $E_v = 0.4\ E_m$

Table ASD-89g Coefficients for Deflection and Rigidity of Walls or Piers for Distribution of Horizontal Forces

Fixed Wall or Pier[1]



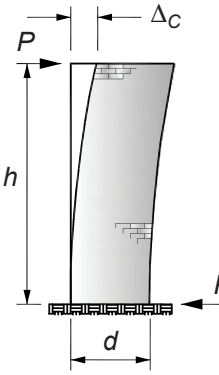
Δ_F = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_F = \frac{P}{E_m t}\left[\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_F = 0.1\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_F = \frac{1}{\Delta_F}$ Rigidity of fixed wall or pier

Cantilever Wall or Pier[1]



Δ_C = Deflection of wall or pier fixed top and bottom[2]

$$\Delta_C = \frac{P}{E_m t}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

$$\Delta_C = 0.4\left(\frac{h}{d}\right)^3 + 0.3\left(\frac{h}{d}\right)$$

$R_C = \frac{1}{\Delta_C}$ Rigidity of cantilever wall or pier

P = 100,000 pounds; t = 1"; E_m = 1,000,000 psi

h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C	h/d	Δ_F	Δ_C	R_F	R_C
7.30	41.09	157.8	0.024	0.006	7.70	47.96	184.9	0.021	0.005	8.10	55.57	215.0	0.018	0.005
7.31	41.25	158.4	0.024	0.006	7.71	48.14	185.6	0.021	0.005	8.11	55.77	215.8	0.018	0.005
7.32	41.42	159.1	0.024	0.006	7.72	48.33	186.4	0.021	0.005	8.12	55.97	216.6	0.018	0.005
7.33	41.58	159.7	0.024	0.006	7.73	48.51	187.1	0.021	0.005	8..13	56.18	217.4	0.018	0.005
7.34	41.75	160.4	0.024	0.006	7.74	48.69	187.8	0.021	0.005	8.14	56.38	218.2	0.018	0.005
7.35	41.91	161.0	0.024	0.006	7.75	48.87	188.5	0.020	0.005	8.15	56.58	219.0	0.018	0.005
7.36	42.08	161.7	0.024	0.006	7.76	49.06	189.2	0.020	0.005	8.16	56.78	219.8	0.018	0.005
7.37	42.24	162.3	0.024	0.006	7.77	49.24	190.0	0.020	0.005	8.17	56.98	220.6	0.018	0.005
7.38	42.41	163.0	0.024	0.006	7.78	49.43	190.7	0.020	0.005	8.18	57.19	221.4	0.017	0.005
7.39	42.58	163.7	0.023	0.006	7.79	49.61	191.4	0.020	0.005	8.19	57.39	222.2	0.017	0.005
7.40	42.74	164.3	0.023	0.006	7.80	49.80	192.2	0.020	0.005	8.20	57.60	223.0	0.017	0.004
7.41	42.91	165.0	0.023	0.006	7.81	49.98	192.9	0.020	0.005	8.21	57.80	223.8	0.017	0.004
7.42	43.08	165.6	0.023	0.006	7.82	50.17	193.6	0.020	0.005	8.22	58.01	224.6	0.017	0.004
7.43	43.25	166.3	0.023	0.006	7.83	50.35	194.4	0.020	0.005	8.23	58.21	225.4	0.017	0.004
7.44	43.42	167.0	0.023	0.006	7.84	50.54	195.1	0.020	0.005	8.24	58.42	226.3	0.017	0.004
7.45	43.58	167.6	0.023	0.006	7.85	50.73	195.8	0.020	0.005	8.25	58.63	227.1	0.017	0.004
7.46	43.75	168.3	0.023	0.006	7.86	50.92	196.6	0.020	0.005	8.26	58.83	227.9	0.017	0.004
7.47	43.92	169.0	0.023	0.006	7.87	51.11	197.3	0.020	0.005	8.27	59.04	228.7	0.017	0.004
7.48	44.09	169.6	0.023	0.006	7.88	51.29	198.1	0.019	0.005	8.28	59.25	229.5	0.017	0.004
7.49	44.27	170.3	0.023	0.006	7.89	51.48	198.8	0.019	0.005	8.29	59.46	230.4	0.017	0.004
7.50	44.44	171.0	0.023	0.006	7.90	51.67	199.6	0.019	0.005	8.30	59.67	231.2	0.017	0.004
7.51	44.61	171.7	0.022	0.006	7.91	51.86	200.3	0.019	0.005	8.31	59.88	232.0	0.017	0.004
7.52	44.78	172.4	0.022	0.006	7.92	52.06	201.1	0.019	0.005	8.32	60.09	232.9	0.017	0.004
7.53	44.95	173.0	0.022	0.006	7.93	52.25	201.8	0.019	0.005	8.33	60.30	233.7	0.017	0.004
7.54	45.13	173.7	0.022	0.006	7.94	52.44	202.6	0.019	0.005	8.34	60.51	234.5	0.017	0.004
7.55	45.30	174.4	0.022	0.006	7.95	52.63	203.4	0.019	0.005	8.35	60.72	235.4	0.016	0.004
7.56	45.48	175.1	0.022	0.006	7.96	52.82	204.1	0.019	0.005	8.36	60.94	236.2	0.016	0.004
7.57	45.65	175.8	0.022	0.006	7.97	53.02	204.9	0.019	0.005	8.37	61.15	237.1	0.016	0.004
7.58	45.83	176.5	0.022	0.006	7.98	53.21	205.7	0.019	0.005	8.38	61.36	237.9	0.016	0.004
7.59	46.00	177.2	0.022	0.006	7.99	53.41	206.4	0.019	0.005	8.39	61.58	238.8	0.016	0.004
7.60	46.18	177.9	0.022	0.006	8.00	53.60	207.2	0.019	0.005	8.40	61.79	239.6	0.016	0.004
7.61	46.35	178.6	0.022	0.006	8.01	53.80	208.0	0.019	0.005	8.41	62.01	240.5	0.016	0.004
7.62	46.53	179.3	0.021	0.006	8.02	53.99	208.7	0.019	0.005	8.42	62.22	241.3	0.016	0.004
7.63	46.71	180.0	0.021	0.006	8.03	54.19	209.5	0.018	0.005	8.43	62.44	242.2	0.016	0.004
7.64	46.89	180.7	0.021	0.006	8.04	54.38	210.3	0.018	0.005	8.44	62.65	243.0	0.016	0.004
7.65	47.06	181.4	0.021	0.006	8.05	54.58	211.1	0.018	0.005	8.45	62.87	243.9	0.016	0.004
7.66	47.24	182.1	0.021	0.005	8.06	54.78	211.9	0.018	0.005	8.46	63.09	244.7	0.016	0.004
7.67	47.42	182.8	0.021	0.005	8.07	54.98	212.6	0.018	0.005	8.47	63.31	245.6	0.016	0.004
7.68	47.60	183.5	0.021	0.005	8.08	55.18	213.4	0.018	0.005	8.48	63.52	246.5	0.016	0.004
7.69	47.78	184.2	0.021	0.005	8.09	55.37	214.2	0.018	0.005	8.49	63.74	247.3	0.016	0.004

1. Piers are limited to an *h/d* ratio of 5 or less
2. Based on a Shear Modulus $E_v = 0.4\ E_m$

Table ASD-91 Allowable Tension B_a (pounds) for Embedded Anchor Bolts in Clay and Concrete Masonry Based on the Masonry Strength[1,2,3]

f'_m (psi)	Embedment Length l_b or Edge Distance l_{be} (inches)						
	2	3	4	5	6	8	10
1000	200	450	790	1240	1790	3180	4970
1500	240	550	970	1520	2190	3890	6080
2000	280	630	1120	1760	2530	4500	7020
2500	310	710	1260	1960	2830	5030	7850
3000	340	770	1380	2150	3100	5510	8600
3500	370	840	1490	2320	3350	5950	9290
4000	400	890	1590	2480	3580	6360	9930
4500	420	950	1690	2630	3790	6740	10540
5000	440	1000	1780	2780	4000	7110	11110
5500	470	1050	1860	2910	4190	7460	11650
6000	490	1100	1950	3040	4380	7790	12170

1. The allowable tension values in Table ASD-91 are based on the compressive strength of masonry assemblages. Where yield strength of anchor bolt steel governs, the allowable tension in pounds is given in Table ASD-92.
2. Values based on MSJC Code Section 2.1.4.2.2 Eq. 2-1.
3. Values may be increased by one-third when considering load combinations including earthquake per MSJC Code Section 2.1.2.3.

Table ASD-92 Allowable Tension B_a (pounds) for Embedded Anchor Bolts in Clay and Concrete Masonry Based on ASTM A307 Anchor Bolts[1,2,3]

Anchor Bolt Diameter (inches)							
1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8
350	800	1410	2210	3180	4330	5650	7160

1. Values are for bolts conforming to ASTM A307. Bolts shall be those specified in MSJC Code Section 2.1.4.2.
2. Values based on MSJC Code Section 2.1.4.2.2 Eq. 2-2.
3. Values may be increased by one-third when considering load combinations including earthquake per MSJC Code Section 2.1.2.3.

Table ASD-93 Allowable Shear B_v (pounds) for Embedded Anchor Bolts in Clay and Concrete Masonry Based on the Masonry Strength and A307 Anchor Bolts[1,2,3,4]

f'_m (psi)	Anchor Bolt Diameter (inches)							
	1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8
1000	210	480	850	1330	1600	1730	1850	1970
1500	210	480	850	1330	1780	1920	2050	2170
2000	210	480	850	1330	1910	2060	2200	2340
2500	210	480	850	1330	1910	2180	2330	2470
3000	210	480	850	1330	1910	2280	2440	2590
3500	210	480	850	1330	1910	2370	2530	2690
4000	210	480	850	1330	1910	2450	2620	2780
4500	210	480	850	1330	1910	2520	2700	2860
5000	210	480	850	1330	1910	2590	2770	2940
5500	210	480	850	1330	1910	2600	2840	3010
6000	210	480	850	1330	1910	2600	2900	3080

1. Values are for bolts conforming to ASTM A307. Bolts shall be those specified in MSJC Code Section 2.1.4.2.
2. The allowable shear values in Table ASD-93 are based on where the compressive strength of masonry assemblages or yield strength of anchor bolt steel governs. Refer to Table ASD-94 for the percent capacity of anchor bolts based on edge distance.
3. Values based on MSJC Code Section 2.1.4.2.3 and MSJC Code Eqs. 2-5 and 2-6. Shaded values are controlled by the capacity of the bolt as given by MSJC Code Eq. 2-6.
4. Values may be increased by one-third when considering load combinations including earthquake per MSJC Code Section 2.1.2.3.

Table ASD-94 Percentage of Shear Capacity of Anchor Bolts Based on Edge Distance l_{be}[1,2,3,4]

Edge Distance	Anchor Bolt Diameter (inches)															
	1/4		3/8		1/2		5/8		3/4		7/8		1		1 1/8	
	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%	l_{be}	%
12d_b	3.0	100	4.5	100	6.0	100	7.5	100	9.0	100	10.5	100	12.0	100	13.5	100
11d_b	2.8	87.5	4.1	89.3	5.5	90.0	6.9	90.4	8.3	90.6	9.6	90.8	11.0	90.9	12.4	91.0
10d_b	2.5	75.0	3.8	78.6	5.0	80.0	6.3	80.8	7.5	81.3	8.8	81.6	10.0	81.8	11.3	82.0
9d_b	2.3	62.5	3.4	67.9	4.5	70.0	5.6	71.2	6.8	71.9	7.9	72.4	9.0	72.7	10.1	73.0
8d_b	2.0	50.0	3.0	57.1	4.0	60.0	5.0	61.5	6.0	62.5	7.0	63.2	8.0	63.6	9.0	64.0
7d_b	1.8	37.5	2.6	46.4	3.5	50.0	4.4	51.9	5.3	53.1	6.1	53.9	7.0	54.5	7.9	55.0
6d_b	1.5	25.0	2.3	35.7	3.0	40.0	3.8	42.3	4.5	43.8	5.3	44.7	6.0	45.5	6.8	46.0
5d_b	1.3	12.5	1.9	25.0	2.5	30.0	3.1	32.7	3.8	34.4	4.4	35.5	5.0	36.4	5.6	37.0
4d_b	1.0	0	1.5	14.3	2.0	20.0	2.5	23.1	3.0	25.0	3.5	26.3	4.0	27.3	4.5	28.0
3d_b			1.1	3.6	1.5	10.0	1.9	13.5	2.3	15.6	2.6	17.1	3.0	18.2	3.4	19.0
2d_b					1.0	0	1.3	3.8	1.5	6.3	1.8	7.9	2.0	9.1	2.3	10.0
1d_b													1.0	0	1.1	1.0
1 in.	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0

1. MSJC Code Section 2.1.4.2.3 requires that the capacity of anchor bolts determined by MSJC Code Eq. 2-5 be reduced when the edge distance is less than 12d_b.
2. d_b = Bar diameters
3. l_{be} = Edge distance in inches
4. % = Percentage capacity of Anchor Bolts

GENERAL NOTES TABLES AND DIAGRAMS

Based on the
International
Building
Code
Requirements

Table GN-1 Weights of Building Materials

	Weight (psf) Pounds per Square Foot
FLOORS:	
Concrete finish, per inch of thickness	12
Light weight concrete fill, per inch of thickness	9
7/8" Hardwood floor on sleepers clipped to concrete without fill	5
1 1/2" Terrazzo floor finish directly on slab	19
1 1/2" Terrazzo floor finish on 1" mortar bed	30
1" Terrazzo finish on 2" concrete bed	38
3/4" Ceramic or quarry tile on 1/2" mortar bed	16
3/4" Ceramic or quarry tile on 1" mortar bed	22
1/4" Linoleum or asphalt tile directly on concrete	1
1/4" Linoleum or asphalt tile on 1" mortar	12
3/4" Mastic floor	9
Hardwood flooring, 7/8" thick	4
Sub-flooring (soft wood), 3/4" thick	2 1/2
Gypsum slab, per inch of thickness	6
Asphalt mastic finish, 1 1/2" thick	18
1/2" Douglas Fir plywood	1 1/2
1" Douglas Fir plywood	3
ROOFS:	
Five-ply felt and gravel (or slag)	6 1/2
Three-ply felt and gravel (or slag)	5 1/2
Five-ply composition roof, no gravel	4
Three-ply felt composition roof, no gravel	3
Asphalt strip shingles	3
Concrete tile	16
Slate, 1/4" thick (laid)	10
Slate, 1/2" thick (laid)	20
Sheathing, 3/4" thick, yellow pine	3 1/2
Sheathing, 3/4" thick, spruce or hemlock	2 1/2
Skylight with galvanized iron frame, 1/4" wire glass	7
Gypsum, per inch of thickness	4
Poured gypsum on steel rails, per inch of thickness	5
Light weight fill or insulation, porous glass, vermiculite, etc, per inch of thickness	1 to 2
Spanish tile (laid)	9 to 12
Shingle-type clay tile	12 to 14
Metal Deck (20 gauge)	2
Metal Deck (18 gauge)	3
Corrugated metal (20 gauge)	2

	Weight (psf) Pounds per Square Foot
CEILINGS:	
3/4" Plaster directly on concrete, blocks or tile	5
3/4" Plaster on metal lath furring	8
3/4" Gypsum plater on metal lath and channel suspended ceiling construction	10
Plaster on rock lath and channel ceiling construction	6
Acoustical fiber tile directly on concrete blocks or tile	1
Acoustical fiber tile on rock lath and channel ceiling construction	5
Acoustical fiber tile on suspended wood furring strips	3
WALLS:[1]	
Windows, Glass, frame and sash	8
Porcelain enamel on sheet steel	3
Structural glass, per inch of thickness	15
Stone 4" thick	55
Glass block 4" thick	18
PARTITIONS:[2]	
3" clay tile	17
4" clay tile	18
6" clay tile	25
8" clay tile	31
10" clay tile	35
3" gypsum block	10
4" gypsum block	13
5" gypsum block	16
6" gypsum block	17
2" solid plaster	20
2 x 4 studs, or metal studs, lath and 3/4" plaster	18
Steel partitions	4
Gypsum plaster per 1/8" thick	1

1. See Tables GN-2 and Tables GN-3a, 3b, 3c for masonry walls.

2. **IBC Section 1607.5** In office buildings and in other buildings where partition locations are subject to change, provisions for partition weight shall be made, whether or not partitions are shown on the construction documents, unless the specified live load exceeds 80 psf (3.83 kN/m^2). The partition load shall not be less than a uniformly distributed live load of 15 psf (0.74 kN/m^2).

Table GN-2 Average Weight[1] of Concrete Masonry Units, Pounds Per Unit (16" Long Units)

Thickness of Units		Lightweight Units: 103 pcf					Medium Weight Units: 115 pcf					Normal Weight Units: 135 pcf				
		4"	6"	8"	10"	12"	4"	6"	8"	10"	12"	4"	6"	8"	10"	12"
Individual Block	4" high units	8	11	13	15	20	9	13	16	17	22	10	16	18	20	26
	8" high units	16	23	27	32	42	18	28	32	36	47	21	33	37	42	55

1. ASTM C90 classifies masonry units as follows: Lightweight: Less than 105 pcf. Medium Weight: 105 pcf to 125 pcf. Normal Weight: 125 pcf or more.

Table GN-3a Average Weight of Completed Walls,[1] Pounds per Square Foot, and Equivalent Solid Thickness, Inches (Weight of Grout = 140 pcf)

Wall Thickness		Hollow Concrete Block												Hollow Clay Block 120 pcf			Equivalent Solid Thickness[2] (inches)			
		Lightweight 103 pcf				Medium Weight 115 pcf				Normal Weight 135 pcf										
		6"	8"	10"	12"	6"	8"	10"	12"	6"	8"	10"	12"	4"	6"	8"	6"	8"	10"	12"
Solid Grouted Wall		52	75	93	118	58	78	98	124	63	84	104	133	38	56	77	5.6	7.6	9.6	11.6
Vertical Cores Grouted at	16" o.c.	41	60	69	88	47	63	80	94	52	66	86	103	33	45	59	4.5	5.8	7.2	8.5
	24" o.c.	37	55	61	79	43	58	72	85	48	61	78	94	31	42	54	4.1	5.2	6.3	7.5
	32" o.c.	36	52	57	74	42	55	68	80	47	58	74	89	30	40	51	4.0	4.9	5.9	7.0
	40" o.c.	35	50	55	71	41	53	66	77	46	56	72	86	29	39	49	3.8	4.7	5.7	6.7
	48" o.c.	34	49	53	69	40	51	64	75	45	55	70	83	28	38	48	3.7	4.6	5.5	6.5
No Grout in Wall		26	33	36	47	32	36	41	53	37	42	47	62	25	30	35	3.4	4.0	4.7	5.5

1. The above table gives the average weight of completed walls of various thicknesses in pounds per square foot of wall face area. An average amount has been added into these values to include the weight of bond beams and reinforcing steel.
2. Equivalent solid thickness means the calculated thickness of the wall if there were no hollow cores, and is obtained by dividing the volume of the solid material in the wall by the face area of the wall. This Equivalent Solid Thickness (*EST*) is for the determination of area for structural design only, e.g., $f_a = P/(EST)b$. A fire rating thickness is based either on equivalent solid thickness of ungrouted units or solid grouted walls (partial grouted walls are considered as ungrouted for fire ratings).

Table GN-3b Average Weight of Completed Walls,[1] Pounds per Square Foot, and Equivalent Solid Thickness, Inches (Weight of Grout = 105 pcf)

Wall Thickness		Hollow Concrete Block												Hollow Clay Block 120 pcf			Equivalent Solid Thickness[2] (inches)			
		Lightweight 103 pcf				Medium Weight 115 pcf				Normal Weight 135 pcf										
		6”	8”	10”	12”	6”	8”	10”	12”	6”	8”	10”	12”	4”	6”	8”	6”	8”	10”	12”
Solid Grouted Wall		45	65	79	100	51	68	84	106	56	74	90	115	35	49	66	5.6	7.6	9.6	11.6
Vertical Cores Grouted at	16” o.c.	37	51	61	78	43	54	66	84	48	60	72	93	32	44	58	4.5	5.8	7.2	8.5
	24” o.c.	35	47	55	71	41	50	60	77	46	56	66	86	30	39	49	4.1	5.2	6.3	7.5
	32” o.c.	33	45	52	67	39	48	57	73	44	54	63	82	29	37	47	4.0	4.9	5.9	7.0
	40” o.c.	32	43	50	65	38	46	55	71	43	52	61	80	28	36	45	3.8	4.7	5.7	6.7
	48” o.c.	31	42	49	63	37	45	54	69	42	51	60	78	27	35	44	3.7	4.6	5.5	6.5
No Grout in Wall		26	33	36	47	32	36	41	53	37	42	47	62	25	30	35	3.4	4.0	4.7	5.5

1. The above table gives the average weight of completed walls of various thicknesses in pounds per square foot of wall face area. An average amount has been added into these values to include the weight of bond beams and reinforcing steel.
2. Equivalent solid thickness means the calculated thickness of the wall if there were no hollow cores, and is obtained by dividing the volume of the solid material in the wall by the face area of the wall. This Equivalent Solid Thickness (*EST*) is for the determination of area for structural design only, e.g., $f_a = P/(EST)b$. A fire rating thickness is based either on equivalent solid thickness of ungrouted units or solid grouted walls (partial grouted walls are considered as ungrouted for fire ratings).

Table GN-3c Average Weight of Reinforced Grouted Brick Walls[1]

Wall Thickness	8”	$8^1/_2$”	9”	$9^1/_2$”	10”	11”	12”	13”
Weight (psf)	80	85	90	95	100	110	120	130

1. Based on an average weight of completed wall of 10 psf per 1” thickness

Diagram GN-4 Wall Section Properties (for Use with Tables GN-4 through GN-12b)

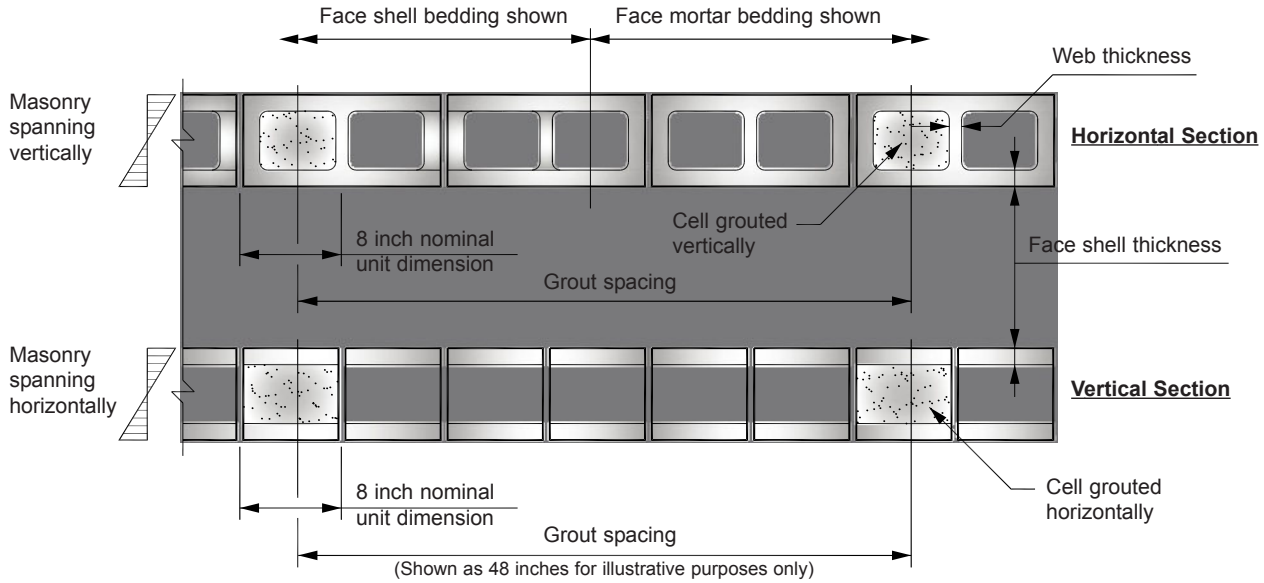


Table GN-4a.4 Wall Section Properties of 4–Inch Clay Masonry, Single Wythe, 4–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1,2]

Grout Spacing (in.)	A_{net} in.² per		I_{net} in.⁴ per		S_{net} in.³ per		r_{net}	A_{avg} in.² per		I_{avg} in.⁴ per		S_{avg} in.³ per		r_{avg}
	spc³	ft	spc³	ft	spc³	ft	(in.)	spc³	ft	spc³	ft	spc³	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	28.5	42.8	30.1	45.2	16.9	25.4	1.03	28.5	42.8	30.1	45.2	16.9	25.4	1.03
16	40.5	30.4	54.4	40.8	30.6	22.9	1.16	44.6	33.5	55.9	41.9	31.4	23.5	1.12
24	52.5	26.3	78.7	39.4	44.2	22.1	1.22	60.8	30.4	81.7	40.8	45.8	22.9	1.16
32	64.5	24.2	103	38.6	57.8	21.7	1.26	76.9	28.8	107	40.3	60.3	22.6	1.18
40	76.5	23.0	127	38.2	71.5	21.4	1.29	93.0	27.9	133	40.0	74.8	22.4	1.20
48	88.5	22.1	152	37.9	85.1	21.3	1.31	109	27.3	159	39.7	89.2	22.3	1.21
56	101	21.5	176	37.7	98.8	21.2	1.32	125	26.8	185	39.6	104	22.2	1.21
64	113	21.1	200	37.5	112	21.1	1.33	141	26.5	210	39.5	118	22.2	1.22
72	125	20.8	224	37.4	126	21.0	1.34	158	26.3	236	39.4	133	22.1	1.22
96	161	20.1	297	37.2	167	20.9	1.36	206	25.7	313	39.2	176	22.0	1.23
120	197	19.7	370	37.0	208	20.8	1.37	254	25.4	391	39.1	219	21.9	1.24
8 Ungrouted	12.0	18.0	24.3	36.4	13.6	20.5	1.42	16.1	24.2	25.8	38.6	14.5	21.7	1.26
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	36.0	18.0	72.9	36.4	40.9	20.5	1.42	54.6	27.3	79.5	39.7	44.6	22.3	1.21
48	72.0	18.0	146	36.4	81.8	20.5	1.42	103	25.7	157	39.2	88.0	22.0	1.23
72	108	18.0	219	36.4	123	20.5	1.42	151	25.2	234	39.0	131	21.9	1.24
96	144	18.0	292	36.4	164	20.5	1.42	200	25.0	311	38.9	175	21.8	1.25
120	180	18.0	364	36.4	205	20.5	1.42	248	24.8	389	38.9	218	21.8	1.25
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	36.0	18.0	72.9	36.4	40.9	20.5	1.42	60.8	30.4	81.7	40.8	45.8	22.9	1.16
48	72.0	18.0	146	36.4	81.8	20.5	1.42	109	27.3	159	39.7	89.2	22.3	1.21
72	108	18.0	219	36.4	123	20.5	1.42	158	26.3	236	39.4	133	22.1	1.22
96	144	18.0	292	36.4	164	20.5	1.42	206	25.7	313	39.2	176	22.0	1.23
120	180	18.0	364	36.4	205	20.5	1.42	254	25.4	391	39.1	219	21.9	1.24
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
4 Solid	14.3	42.8	15.1	45.2	8.5	25.4	1.03	14.3	42.8	15.1	45.2	8.5	25.4	1.03
8	20.3	30.4	27.2	40.8	15.3	22.9	1.16	22.3	33.5	28.0	41.9	15.7	23.5	1.12
16	32.3	24.2	51.5	38.6	28.9	21.7	1.26	38.4	28.8	53.7	40.3	30.2	22.6	1.18
24	44.3	22.1	75.8	37.9	42.6	21.3	1.31	54.6	27.3	79.5	39.7	44.6	22.3	1.21
32	56.3	21.1	100	37.5	56.2	21.1	1.33	70.7	26.5	105	39.5	59.1	22.2	1.22
40	68.3	20.5	124	37.3	69.8	20.9	1.35	86.8	26.1	131	39.3	73.5	22.1	1.23
48	80.3	20.1	149	37.2	83.5	20.9	1.36	103	25.7	157	39.2	88.0	22.0	1.23
56	92.3	19.8	173	37.1	97.1	20.8	1.37	119	25.5	182	39.1	102	22.0	1.24
64	104	19.5	197	37.0	111	20.8	1.38	135	25.4	208	39.0	117	21.9	1.24
72	116	19.4	222	36.9	124	20.7	1.38	151	25.2	234	39.0	131	21.9	1.24
96	152	19.0	294	36.8	165	20.7	1.39	200	25.0	311	38.9	175	21.8	1.25
120	188	18.8	367	36.7	206	20.6	1.40	248	24.8	389	38.9	218	21.8	1.25
4 Ungrouted	6.0	18.0	12.1	36.4	6.8	20.5	1.42	8.1	24.2	12.9	38.6	7.2	21.7	1.26

1. Based on 3/4 in. Face Shells and 7/16 in. joints for 3 9/16 in. actual unit width with 6 cross-webs.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-4a.8 Wall Section Properties of 4–Inch Clay Masonry, Single Wythe, 8–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1,2]

Grout Spacing (in.)	A_{net} in.² per		I_{net} in.⁴ per		S_{net} in.³ per		r_{net}	A_{avg} in.² per		I_{avg} in.⁴ per		S_{avg} in.³ per		r_{avg}
	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	28.5	42.8	30.1	45.2	16.9	25.4	1.03	28.5	42.8	30.1	45.2	16.9	25.4	1.03
16	40.5	30.4	54.4	40.8	30.6	22.9	1.16	44.9	33.7	56.0	42.0	31.4	23.6	1.12
24	52.5	26.3	78.7	39.4	44.2	22.1	1.22	61.3	30.6	81.8	40.9	45.9	23.0	1.16
32	64.5	24.2	103	38.6	57.8	21.7	1.26	77.7	29.1	108	40.4	60.5	22.7	1.18
40	76.5	23.0	127	38.2	71.5	21.4	1.29	94.0	28.2	134	40.1	75.0	22.5	1.19
48	88.5	22.1	152	37.9	85.1	21.3	1.31	110	27.6	159	39.8	89.5	22.4	1.20
56	101	21.5	176	37.7	98.8	21.2	1.32	127	27.2	185	39.7	104	22.3	1.21
64	113	21.1	200	37.5	112	21.1	1.33	143	26.9	211	39.6	119	22.2	1.21
72	125	20.8	224	37.4	126	21.0	1.34	160	26.6	237	39.5	133	22.2	1.22
96	161	20.1	297	37.2	167	20.9	1.36	209	26.1	314	39.3	177	22.1	1.23
120	197	19.7	370	37.0	208	20.8	1.37	258	25.8	392	39.2	220	22.0	1.23
8 Ungrouted	12.0	18.0	24.3	36.4	13.6	20.5	1.42	16.4	24.6	25.8	38.8	14.5	21.8	1.26
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	36.0	18.0	72.9	36.4	40.9	20.5	1.42	61.3	30.6	81.8	40.9	45.9	23.0	1.16
48	72.0	18.0	146	36.4	81.8	20.5	1.42	110	27.6	159	39.8	89.5	22.4	1.20
72	108	18.0	219	36.4	123	20.5	1.42	160	26.6	237	39.5	133	22.2	1.22
96	144	18.0	292	36.4	164	20.5	1.42	209	26.1	314	39.3	177	22.1	1.23
120	180	18.0	364	36.4	205	20.5	1.42	258	25.8	392	39.2	220	22.0	1.23
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	36.0	18.0	72.9	36.4	40.9	20.5	1.42	61.3	30.6	81.8	40.9	45.9	23.0	1.16
48	72.0	18.0	146	36.4	81.8	20.5	1.42	110	27.6	159	39.8	89.5	22.4	1.20
72	108	18.0	219	36.4	123	20.5	1.42	160	26.6	237	39.5	133	22.2	1.22
96	144	18.0	292	36.4	164	20.5	1.42	209	26.1	314	39.3	177	22.1	1.23
120	180	18.0	364	36.4	205	20.5	1.42	258	25.8	392	39.2	220	22.0	1.23
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
8 Solid	28.5	42.8	30.1	45.2	16.9	25.4	1.03	28.5	42.8	30.1	45.2	16.9	25.4	1.03
16	40.5	30.4	54.4	40.8	30.6	22.9	1.16	44.9	33.7	56.0	42.0	31.4	23.6	1.12
24	52.5	26.3	78.7	39.4	44.2	22.1	1.22	61.3	30.6	81.8	40.9	45.9	23.0	1.16
32	64.5	24.2	103	38.6	57.8	21.7	1.26	77.7	29.1	108	40.4	60.5	22.7	1.18
40	76.5	23.0	127	38.2	71.5	21.4	1.29	94.0	28.2	134	40.1	75.0	22.5	1.19
48	88.5	22.1	152	37.9	85.1	21.3	1.31	110	27.6	159	39.8	89.5	22.4	1.20
56	101	21.5	176	37.7	98.8	21.2	1.32	127	27.2	185	39.7	104	22.3	1.21
64	113	21.1	200	37.5	112	21.1	1.33	143	26.9	211	39.6	119	22.2	1.21
72	125	20.8	224	37.4	126	21.0	1.34	160	26.6	237	39.5	133	22.2	1.22
96	161	20.1	297	37.2	167	20.9	1.36	209	26.1	314	39.3	177	22.1	1.23
120	197	19.7	370	37.0	208	20.8	1.37	258	25.8	392	39.2	220	22.0	1.23
8 Ungrouted	12.0	18.0	24.3	36.4	13.6	20.5	1.42	16.4	24.6	25.8	38.8	14.5	21.8	1.26

1. Based on 3/4 in. Face Shells and 7/16 in. joints for 3 9/16 in. actual unit width with 6 cross-webs 3/4 in. thick.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-4b Wall Section Properties of 4–Inch Concrete Masonry, Single Wythe Walls, 8–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1, 2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc³	ft	spc³	ft	spc³	ft	(in.)	spc³	ft	spc³	ft	spc³	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	28.6	42.9	31.6	47.4	17.4	26.2	1.05	28.6	42.9	31.6	47.4	17.4	26.2	1.05
16	41.4	31.0	57.3	43.0	31.6	23.7	1.18	43.7	32.2	57.8	43.4	31.9	23.9	1.16
24	53.4	26.7	82.6	41.3	45.6	22.8	1.24	58.0	28.6	84.0	42.0	46.4	23.2	1.21
32	65.4	24.5	108	40.5	59.6	22.3	1.28	72.2	26.8	110	41.4	60.8	22.8	1.24
40	77.4	23.2	133	40.0	73.6	22.1	1.31	86.5	25.7	137	41.0	75.3	22.6	1.26
48	89.4	22.4	159	39.7	87.6	21.9	1.33	101	25.0	163	40.7	89.8	22.4	1.28
56	101	21.7	184	39.4	102	21.8	1.35	115	24.5	189	40.5	104	22.3	1.29
64	113	21.3	209	39.3	116	21.7	1.36	129	24.1	215	40.3	119	22.3	1.29
72	125	20.9	235	39.1	130	21.6	1.37	144	23.8	241	40.2	134	22.2	1.30
96	161	20.2	311	38.9	172	21.4	1.39	186	23.2	320	40.0	177	22.1	1.31
120	197	19.7	387	38.7	213	21.4	1.40	229	22.8	399	39.9	220	22.0	1.32
8 Ungrouted	12.0	18.0	25.4	38.0	14.0	21.0	1.45	14.3	21.4	26.2	39.3	14.5	21.7	1.36
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	36.0	18.0	76.1	38.0	42.0	21.0	1.45	57.6	28.8	84.2	42.1	46.4	23.2	1.21
48	72.0	18.0	152	38.0	84.0	21.0	1.45	100	25.1	163	40.7	89.8	22.5	1.27
72	108	18.0	228	38.0	126	21.0	1.45	143	23.9	241	40.2	133	22.2	1.30
96	144	18.0	304	38.0	168	21.0	1.45	187	23.3	320	40.0	177	22.1	1.31
120	180	18.0	380	38.0	210	21.0	1.45	229	22.9	399	39.9	220	22.0	1.32
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	36.0	18.0	76.1	38.0	42.0	21.0	1.45	57.1	26.5	84.2	42.1	46.4	23.2	1.26
48	72.0	18.0	152	38.0	84.0	21.0	1.45	99.3	22.2	163	40.7	89.8	22.5	1.35
72	108	18.0	228	38.0	126	21.0	1.45	141	20.8	241	40.2	133	22.2	1.39
96	144	18.0	304	38.0	168	21.0	1.45	184	20.1	320	40.0	177	22.1	1.41
120	180	18.0	380	38.0	210	21.0	1.45	226	19.7	399	39.9	220	22.0	1.42
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
8 Solid	29.0	43.5	31.8	47.6	17.5	26.3	1.05	29.0	43.5	31.8	47.6	17.5	26.3	1.05
16	41.0	30.8	57.1	42.8	31.5	23.6	1.18	43.0	34.0	57.8	43.9	31.9	24.2	1.14
24	53.0	26.5	82.5	41.2	45.5	22.8	1.25	57.1	29.8	83.9	42.3	46.3	23.3	1.19
32	65.0	24.4	108	40.4	59.5	22.3	1.29	71.2	27.6	101	41.5	60.6	22.9	1.22
40	77.0	23.1	133	40.0	73.5	22.0	1.32	85.2	26.4	136	41.0	75.0	22.6	1.25
48	89.0	22.2	159	39.6	87.5	21.9	1.34	99.3	25.5	162	40.7	89.4	22.4	1.26
56	101	21.6	184	39.4	101	21.7	1.35	113	24.9	188	40.4	104	22.3	1.28
64	113	21.2	209	39.2	115	21.6	1.36	127	24.4	214	40.3	118	22.2	1.28
72	125	20.8	235	39.1	129	21.6	1.37	141	24.1	240	40.1	132	22.2	1.29
96	161	20.1	311	38.8	171	21.4	1.39	184	23.4	318	39.9	176	22.0	1.31
120	197	19.7	387	38.7	213	21.3	1.40	226	23.0	396	39.7	219	21.9	1.32
8 Ungrouted	12.0	18.0	25.4	38.0	14.0	21.0	1.45	14.0	21.2	26.0	39.1	14.4	21.6	1.36

1. Based on 3/4 in. Face Shells and in part on NCMA TEK Note 14-1B.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-5a.4 Wall Section Properties of 5–Inch Clay Masonry, Single Wythe, 3 1/8–Inch High, 10–Inch Long Masonry Units, Face Shell Bedding[1,2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc³	ft	spc³	ft	spc³	ft	(in.)	spc³	ft	spc³	ft	spc³	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
5 Solid	23.1	55.5	41.2	98.9	17.8	42.8	1.34	23.1	55.5	41.2	98.9	17.8	42.8	1.34
10	33.1	39.8	74.9	89.9	32.4	38.9	1.50	36.6	43.9	76.9	92.3	33.3	39.9	1.45
20	53.1	31.9	142	85.4	61.5	36.9	1.64	63.5	38.1	148	88.9	64.1	38.5	1.53
30	73.1	29.3	210	83.9	90.7	36.3	1.69	90.5	36.2	220	87.8	95.0	38.0	1.56
40	93.1	27.9	277	83.1	120	35.9	1.72	117	35.2	291	87.3	126	37.7	1.57
50	113	27.2	344	82.7	149	35.7	1.74	144	34.6	362	87.0	157	37.6	1.58
60	133	26.6	412	82.4	178	35.6	1.76	171	34.2	434	86.7	188	37.5	1.59
70	153	26.3	479	82.1	207	35.5	1.77	198	34.0	505	86.6	218	37.4	1.60
80	173	26.0	546	82.0	236	35.4	1.78	225	33.8	576	86.5	249	37.4	1.60
100	213	25.6	681	81.7	295	35.4	1.79	279	33.5	719	86.3	311	37.3	1.61
120	253	25.3	816	81.6	353	35.3	1.80	333	33.3	862	86.2	373	37.3	1.61
5 Ungrouted	10.0	24.0	33.7	80.8	14.6	35.0	1.84	13.5	32.3	35.7	85.6	15.4	37.0	1.63
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
25	50.0	24.0	168	80.8	72.8	35.0	1.84	73.4	35.2	182	87.3	78.6	37.7	1.57
50	100	24.0	337	80.8	146	35.0	1.84	141	33.8	360	86.5	156	37.4	1.60
75	150	24.0	505	80.8	218	35.0	1.84	208	33.3	539	86.2	233	37.3	1.61
100	200	24.0	674	80.8	291	35.0	1.84	275	33.0	717	86.0	310	37.2	1.61
125	250	24.0	842	80.8	364	35.0	1.84	343	32.9	895	86.0	387	37.2	1.62
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
20	40.0	24.0	135	80.8	58.3	35.0	1.84	66.1	39.7	150	89.8	64.8	38.9	1.50
40	80.0	24.0	269	80.8	117	35.0	1.84	123	37.0	294	88.3	127	38.2	1.54
60	120	24.0	404	80.8	175	35.0	1.84	181	36.2	439	87.8	190	38.0	1.56
80	160	24.0	539	80.8	233	35.0	1.84	238	35.7	584	87.6	252	37.9	1.57
100	200	24.0	674	80.8	291	35.0	1.84	295	35.5	728	87.4	315	37.8	1.57
120	240	24.0	808	80.8	350	35.0	1.84	353	35.3	873	87.3	378	37.8	1.57
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
3 1/8 Solid	14.5	55.5	25.8	98.9	11.1	42.8	1.34	14.5	55.5	25.8	98.9	11.1	42.8	1.34
12.5	33.2	31.9	88.9	85.4	38.5	36.9	1.64	39.7	38.1	92.7	88.9	40.1	38.5	1.53
25.0	58.2	27.9	173	83.1	74.9	35.9	1.72	73.4	35.2	182	87.3	78.6	37.7	1.57
37.5	83.2	26.6	257	82.4	111	35.6	1.76	107	34.2	271	86.7	117	37.5	1.59
50.0	108	26.0	342	82.0	148	35.4	1.78	141	33.8	360	86.5	156	37.4	1.60
62.5	133	25.6	426	81.7	184	35.4	1.79	174	33.5	449	86.3	194	37.3	1.61
75.0	158	25.3	510	81.6	221	35.3	1.80	208	33.3	539	86.2	233	37.3	1.61
87.5	183	25.1	594	81.5	257	35.2	1.80	242	33.1	628	86.1	271	37.2	1.61
100.0	208	25.0	678	81.4	293	35.2	1.81	275	33.0	717	86.0	310	37.2	1.61
112.5	233	24.9	763	81.3	330	35.2	1.81	309	33.0	806	86.0	349	37.2	1.62
125.0	258	24.8	847	81.3	366	35.2	1.81	343	32.9	895	86.0	387	37.2	1.62
3 1/8 Ungrouted	6.3	24.0	21.1	80.8	9.1	35.0	1.84	8.4	32.3	22.3	85.6	9.6	37.0	1.63

1. Based on 1 in. Face Shells and 3/8 in. joints for 4 5/8 in. actual unit width with 4 cross-webs.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-6a.4 Wall Section Properties of 6–Inch Clay Masonry, Single Wythe, 4–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1,2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc³	ft	spc³	ft	spc³	ft	(in.)	spc³	ft	spc³	ft	spc³	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	44.5	66.8	115	172	41.3	61.9	1.61	44.5	66.8	115	172	41.3	61.9	1.61
16	64.5	48.4	210	158	75.6	56.7	1.81	69.6	52.2	214	161	77.1	57.8	1.75
24	84.5	42.3	306	153	110	55.0	1.90	94.7	47.4	314	157	113	56.4	1.82
32	105	39.2	402	151	144	54.1	1.96	120	44.9	414	155	149	55.8	1.86
40	125	37.4	497	149	179	53.6	2.00	145	43.5	513	154	184	55.3	1.88
48	145	36.1	593	148	213	53.3	2.03	170	42.5	613	153	220	55.1	1.90
56	165	35.3	688	147	247	53.0	2.05	195	41.8	712	153	256	54.9	1.91
64	185	34.6	784	147	282	52.8	2.06	220	41.3	812	152	292	54.7	1.92
72	205	34.1	879	147	316	52.7	2.07	245	40.9	911	152	328	54.6	1.93
96	265	33.1	1166	146	419	52.4	2.10	321	40.1	1210	151	435	54.4	1.94
120	325	32.5	1453	145	522	52.2	2.12	396	39.6	1509	151	543	54.3	1.95
8 Ungrouted	20.0	30.0	95.6	143	34.4	51.6	2.19	25.1	37.7	99.6	149	35.8	53.7	1.99
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	60.0	30.0	287	143	103	51.6	2.19	85.0	42.5	306	153	110	55.1	1.90
48	120	30.0	574	143	206	51.6	2.19	160	40.1	605	151	218	54.4	1.94
72	180	30.0	860	143	309	51.6	2.19	236	39.3	904	151	325	54.2	1.96
96	240	30.0	1147	143	412	51.6	2.19	311	38.9	1203	150	432	54.1	1.97
120	300	30.0	1434	143	516	51.6	2.19	386	38.6	1501	150	540	54.0	1.97
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	60.0	30.0	287	143	103	51.6	2.19	94.7	47.4	314	157	113	56.4	1.82
48	120	30.0	574	143	206	51.6	2.19	170	42.5	613	153	220	55.1	1.90
72	180	30.0	860	143	309	51.6	2.19	245	40.9	911	152	328	54.6	1.93
96	240	30.0	1147	143	412	51.6	2.19	321	40.1	1210	151	435	54.4	1.94
120	300	30.0	1434	143	516	51.6	2.19	396	39.6	1509	151	543	54.3	1.95
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
4 Solid	22.3	66.8	57.4	172	20.6	61.9	1.61	22.3	66.8	57.4	172	20.6	61.9	1.61
8	32.3	48.4	105	158	37.8	56.7	1.81	34.8	52.2	107	161	38.5	57.8	1.75
16	52.3	39.2	201	151	72.2	54.1	1.96	59.9	44.9	207	155	74.3	55.8	1.86
24	72.3	36.1	296	148	107	53.3	2.03	85.0	42.5	306	153	110	55.1	1.90
32	92.3	34.6	392	147	141	52.8	2.06	110	41.3	406	152	146	54.7	1.92
40	112	33.7	488	146	175	52.6	2.08	135	40.6	506	152	182	54.5	1.93
48	132	33.1	583	146	210	52.4	2.10	160	40.1	605	151	218	54.4	1.94
56	152	32.6	679	145	244	52.3	2.11	185	39.7	705	151	253	54.3	1.95
64	172	32.3	774	145	278	52.2	2.12	211	39.5	804	151	289	54.2	1.95
72	192	32.0	870	145	313	52.1	2.13	236	39.3	904	151	325	54.2	1.96
96	252	31.5	1157	145	416	52.0	2.14	311	38.9	1203	150	432	54.1	1.97
120	312	31.2	1443	144	519	51.9	2.15	386	38.6	1501	150	540	54.0	1.97
4 Ungrouted	10.0	30.0	47.8	143	17.2	51.6	2.19	12.6	37.7	49.8	149	17.9	53.7	1.99

1. Based on $1^1/_4$ in. Face Shells and $^7/_{16}$ in. joints for $5^9/_{16}$ in. actual unit width with 4 cross-webs.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-6a.8 Wall Section Properties of 6–Inch Clay Masonry, Single Wythe, 8–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1,2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	44.5	66.8	115	172	41.3	61.9	1.61	44.5	66.8	115	172	41.3	61.9	1.61
16	64.5	48.4	210	158	75.6	56.7	1.81	69.9	52.4	215	161	77.2	57.9	1.75
24	84.5	42.3	306	153	110	55.0	1.90	95.4	47.7	314	157	113	56.5	1.82
32	105	39.2	402	151	144	54.1	1.96	121	45.3	414	155	149	55.9	1.85
40	125	37.4	497	149	179	53.6	2.00	146	43.9	514	154	185	55.5	1.88
48	145	36.1	593	148	213	53.3	2.03	172	42.9	614	153	221	55.2	1.89
56	165	35.3	688	147	247	53.0	2.05	197	42.2	714	153	257	55.0	1.90
64	185	34.6	784	147	282	52.8	2.06	222	41.7	814	153	293	54.8	1.91
72	205	34.1	879	147	316	52.7	2.07	248	41.3	913	152	328	54.7	1.92
96	265	33.1	1166	146	419	52.4	2.10	324	40.5	1213	152	436	54.5	1.93
120	325	32.5	1453	145	522	52.2	2.12	400	40.0	1512	151	544	54.4	1.94
8 Ungrouted	20.0	30.0	95.6	143	34.4	51.6	2.19	25.4	38.1	99.8	150	35.9	53.8	1.98
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	60.0	30.0	287	143	103	51.6	2.19	95.4	47.7	314	157	113	56.5	1.82
48	120	30.0	574	143	206	51.6	2.19	172	42.9	614	153	221	55.2	1.89
72	180	30.0	860	143	309	51.6	2.19	248	41.3	913	152	328	54.7	1.92
96	240	30.0	1147	143	412	51.6	2.19	324	40.5	1213	152	436	54.5	1.93
120	300	30.0	1434	143	516	51.6	2.19	400	40.0	1512	151	544	54.4	1.94
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	60.0	30.0	287	143	103	51.6	2.19	95.4	47.7	314	157	113	56.5	1.82
48	120	30.0	574	143	206	51.6	2.19	172	42.9	614	153	221	55.2	1.89
72	180	30.0	860	143	309	51.6	2.19	248	41.3	913	152	328	54.7	1.92
96	240	30.0	1147	143	412	51.6	2.19	324	40.5	1213	152	436	54.5	1.93
120	300	30.0	1434	143	516	51.6	2.19	400	40.0	1512	151	544	54.4	1.94
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
8 Solid	44.5	66.8	115	172	41.3	61.9	1.61	44.5	66.8	115	172	41.3	61.9	1.61
16	64.5	48.4	210	158	75.6	56.7	1.81	69.9	52.4	215	161	77.2	57.9	1.75
24	84.5	42.3	306	153	110	55.0	1.90	95.4	47.7	314	157	113	56.5	1.82
32	105	39.2	402	151	144	54.1	1.96	121	45.3	414	155	149	55.9	1.85
40	125	37.4	497	149	179	53.6	2.00	146	43.9	514	154	185	55.5	1.88
48	145	36.1	593	148	213	53.3	2.03	172	42.9	614	153	221	55.2	1.89
56	165	35.3	688	147	247	53.0	2.05	197	42.2	714	153	257	55.0	1.90
64	185	34.6	784	147	282	52.8	2.06	222	41.7	814	153	293	54.8	1.91
72	205	34.1	879	147	316	52.7	2.07	248	41.3	913	152	328	54.7	1.92
96	265	33.1	1166	146	419	52.4	2.10	324	40.5	1213	152	436	54.5	1.93
120	325	32.5	1453	145	522	52.2	2.12	400	40.0	1512	151	544	54.4	1.94
8 Ungrouted	20.0	30.0	95.6	143	34.4	51.6	2.19	25.4	38.1	99.8	150	35.9	53.8	1.98

1. Based on $1\frac{1}{4}$ in. Face Shells and $\frac{7}{16}$ in. joints for $5\frac{9}{16}$ in.actual unit width with 4 cross-webs.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-6b Wall Section Properties of 6–Inch Concrete Masonry, Single Wythe Walls, 8–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1, 2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc³	ft	spc³	ft	spc³	ft	(in.)	spc³	ft	spc³	ft	spc³	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	44.3	66.5	118	177	41.9	62.9	1.63	44.3	66.5	118	177	41.9	62.9	1.63
16	62.1	46.6	207	155	73.5	55.1	1.82	67.3	49.2	211	158	74.9	56.2	1.79
24	78.1	39.1	294	147	104	52.2	1.94	88.5	43.4	303	152	108	53.9	1.87
32	94.1	35.3	381	143	135	50.7	2.01	110	40.5	396	149	141	52.8	1.92
40	110	33.0	467	140	166	49.9	2.06	131	38.7	488	146	174	52.1	1.94
48	126	31.5	554	139	197	49.3	2.10	152	37.6	581	145	207	51.6	1.97
56	142	30.5	641	137	228	48.9	2.12	173	36.8	673	144	239	51.3	1.98
64	158	29.7	728	137	259	48.5	2.15	194	36.1	766	144	272	51.1	1.99
72	174	29.0	815	136	290	48.3	2.16	216	35.6	859	143	305	50.9	2.00
96	222	27.8	1076	134	382	47.8	2.20	279	34.7	1136	142	404	50.5	2.02
120	270	27.0	1336	134	475	47.5	2.22	343	34.1	1414	141	503	50.3	2.04
8 Ungrouted	16.0	24.0	86.9	130	30.9	46.3	2.33	21.2	31.8	92.6	139	32.9	49.4	2.09
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	48.0	24.0	261	130	92.7	46.3	2.33	87.4	43.7	304	152	108	54.0	1.86
48	96.0	24.0	521	130	185	46.3	2.33	151	37.7	582	145	207	51.7	1.96
72	144	24.0	782	130	278	46.3	2.33	214	35.7	859	143	306	50.9	2.00
96	192	24.0	1043	130	371	46.3	2.33	278	34.8	1137	142	404	50.5	2.02
120	240	24.0	1303	130	463	46.3	2.33	342	34.2	1415	141	503	50.3	2.04
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	48.0	24.0	261	130	92.7	46.3	2.33	86.3	38.5	304	152	108	54.0	1.99
48	96.0	24.0	521	130	185	46.3	2.33	148	31.3	582	145	207	51.7	2.16
72	144	24.0	782	130	278	46.3	2.33	210	28.8	859	143	306	50.9	2.23
96	192	24.0	1043	130	371	46.3	2.33	272	27.6	1137	142	404	50.5	2.27
120	240	24.0	1303	130	463	46.3	2.33	334	26.9	1415	142	503	50.3	2.29
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
8 Solid	45.0	67.5	119	178	42.2	63.3	1.62	45.0	67.5	119	178	42.2	63.3	1.62
16	61.0	45.8	206	154	73.1	54.8	1.84	65.7	53.1	210	162	74.8	57.4	1.74
24	77.0	38.5	292	146	104	52.0	1.95	86.3	45.9	302	154	108	54.6	1.83
32	93.0	34.9	379	142	135	50.6	2.02	107	42.3	394	150	140	53.2	1.88
40	109	32.7	466	140	166	49.7	2.07	128	40.1	486	147	173	52.4	1.92
48	125	31.3	553	138	197	49.2	2.10	148	38.6	578	146	205	51.8	1.94
56	141	30.2	640	137	228	48.8	2.13	169	37.6	670	144	238	51.4	1.96
64	157	29.4	727	136	258	48.5	2.15	190	36.8	761	144	271	51.1	1.98
72	173	28.8	814	136	289	48.2	2.17	210	36.2	853	143	303	50.8	1.99
96	221	27.6	1074	134	382	47.8	2.20	272	35.0	1128	142	401	50.4	2.01
120	269	26.9	1335	134	475	47.5	2.23	334	34.3	1404	141	499	50.1	2.03
8 Ungrouted	16.0	24.0	86.9	130	30.9	46.3	2.33	21.7	31.4	91.8	138	32.6	49.0	2.09

1. Based on 1 in. Face Shells and in part on NCMA TEK Note 14-1B.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-8a.4 Wall Section Properties of 8–Inch Clay Masonry, Single Wythe, 4–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1,2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	60.5	90.8	288	433	76.3	114	2.18	60.5	90.8	288	433	76.3	114	2.18
16	80.5	60.4	490	368	130	97.2	2.47	89.0	66.7	508	381	134	101	2.39
24	101	50.3	692	346	183	91.5	2.62	117	58.7	728	364	193	96.3	2.49
32	121	45.2	894	335	236	88.6	2.72	146	54.7	948	356	251	94.0	2.55
40	141	42.2	1096	329	290	86.9	2.79	174	52.3	1168	350	309	92.7	2.59
48	161	40.1	1298	324	343	85.8	2.84	203	50.7	1388	347	367	91.8	2.62
56	181	38.7	1499	321	397	85.0	2.88	231	49.5	1608	345	425	91.1	2.64
64	201	37.6	1701	319	450	84.4	2.91	260	48.7	1828	343	483	90.6	2.65
72	221	36.8	1903	317	503	83.9	2.94	288	48.0	2048	341	541	90.2	2.67
96	281	35.1	2509	314	663	82.9	2.99	373	46.7	2707	338	716	89.5	2.69
120	341	34.1	3114	311	824	82.4	3.02	459	45.9	3367	337	890	89.0	2.71
8 Ungrouted	20.0	30.0	202	303	53.4	80.1	3.18	28.5	42.7	220	330	58.2	87.2	2.78
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	60.0	30.0	606	303	160	80.1	3.18	101	50.7	694	347	184	91.8	2.62
48	120	30.0	1211	303	320	80.1	3.18	187	46.7	1354	338	358	89.5	2.69
72	180	30.0	1817	303	480	80.1	3.18	272	45.4	2013	336	532	88.7	2.72
96	240	30.0	2422	303	641	80.1	3.18	357	44.7	2673	334	707	88.4	2.73
120	300	30.0	3028	303	801	80.1	3.18	443	44.3	3333	333	881	88.1	2.74
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	60.0	30.0	606	303	160	80.1	3.18	117	58.7	728	364	193	96.3	2.49
48	120	30.0	1211	303	320	80.1	3.18	203	50.7	1388	347	367	91.8	2.62
72	180	30.0	1817	303	480	80.1	3.18	288	48.0	2048	341	541	90.2	2.67
96	240	30.0	2422	303	641	80.1	3.18	373	46.7	2707	338	716	89.5	2.69
120	300	30.0	3028	303	801	80.1	3.18	459	45.9	3367	337	890	89.0	2.71
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
4 Solid	30.3	90.8	144	433	38.1	114	2.18	30.3	90.8	144	433	38.1	114	2.18
8	40.3	60.4	245	368	64.8	97.2	2.47	44.5	66.7	254	381	67.2	101	2.39
16	60.3	45.2	447	335	118	88.6	2.72	72.9	54.7	474	356	125	94.0	2.55
24	80.3	40.1	649	324	172	85.8	2.84	101	50.7	694	347	184	91.8	2.62
32	100	37.6	851	319	225	84.4	2.91	130	48.7	914	343	242	90.6	2.65
40	120	36.1	1052	316	278	83.5	2.96	158	47.5	1134	340	300	89.9	2.68
48	140	35.1	1254	314	332	82.9	2.99	187	46.7	1354	338	358	89.5	2.69
56	160	34.3	1456	312	385	82.5	3.01	215	46.1	1574	337	416	89.2	2.70
64	180	33.8	1658	311	438	82.2	3.03	244	45.7	1793	336	474	88.9	2.71
72	200	33.4	1860	310	492	82.0	3.05	272	45.4	2013	336	532	88.7	2.72
96	260	32.5	2465	308	652	81.5	3.08	357	44.7	2673	334	707	88.4	2.73
120	320	32.0	3071	307	812	81.2	3.10	443	44.3	3333	333	881	88.1	2.74
4 Ungrouted	10.0	30.0	101	303	26.7	80.1	3.18	14.2	42.7	110	330	29.1	87.2	2.78

1. Based on $1\frac{1}{4}$ in. Face Shells and $\frac{7}{16}$ in. joints for $7\frac{9}{16}$ in. actual unit width with 4 cross-webs.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-8a.8 Wall Section Properties of 8–Inch Clay Masonry, Single Wythe, 8–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1,2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	60.5	90.8	288	433	76.3	114	2.18	60.5	90.8	288	433	76.3	114	2.18
16	80.5	60.4	490	368	130	97.2	2.47	89.5	67.1	509	382	135	101	2.39
24	101	50.3	692	346	183	91.5	2.62	118	59.2	730	365	193	96.6	2.48
32	121	45.2	894	335	236	88.6	2.72	147	55.3	951	357	252	94.3	2.54
40	141	42.2	1096	329	290	86.9	2.79	176	52.9	1172	352	310	93.0	2.58
48	161	40.1	1298	324	343	85.8	2.84	205	51.3	1393	348	368	92.1	2.60
56	181	38.7	1499	321	397	85.0	2.88	234	50.2	1614	346	427	91.5	2.62
64	201	37.6	1701	319	450	84.4	2.91	263	49.4	1835	344	485	91.0	2.64
72	221	36.8	1903	317	503	83.9	2.94	292	48.7	2056	343	544	90.6	2.65
96	281	35.1	2509	314	663	82.9	2.99	379	47.4	2719	340	719	89.9	2.68
120	341	34.1	3114	311	824	82.4	3.02	466	46.6	3382	338	895	89.5	2.69
8 Ungrouted	20.0	30.0	202	303	53.4	80.1	3.18	29.0	43.5	221	332	58.4	87.7	2.76
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	60.0	30.0	606	303	160	80.1	3.18	118	59.2	730	365	193	96.6	2.48
48	120	30.0	1211	303	320	80.1	3.18	205	51.3	1393	348	368	92.1	2.60
72	180	30.0	1817	303	480	80.1	3.18	292	48.7	2056	343	544	90.6	2.65
96	240	30.0	2422	303	641	80.1	3.18	379	47.4	2719	340	719	89.9	2.68
120	300	30.0	3028	303	801	80.1	3.18	466	46.6	3382	338	895	89.5	2.69
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	60.0	30.0	606	303	160	80.1	3.18	118	59.2	730	365	193	96.6	2.48
48	120	30.0	1211	303	320	80.1	3.18	205	51.3	1393	348	368	92.1	2.60
72	180	30.0	1817	303	480	80.1	3.18	292	48.7	2056	343	544	90.6	2.65
96	240	30.0	2422	303	641	80.1	3.18	379	47.4	2719	340	719	89.9	2.68
120	300	30.0	3028	303	801	80.1	3.18	466	46.6	3382	338	895	89.5	2.69
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
8 Solid	60.5	90.8	288	433	76.3	114	2.18	60.5	90.8	288	433	76.3	114	2.18
16	80.5	60.4	490	368	130	97.2	2.47	89.5	67.1	509	382	135	101	2.39
24	101	50.3	692	346	183	91.5	2.62	118	59.2	730	365	193	96.6	2.48
32	121	45.2	894	335	236	88.6	2.72	147	55.3	951	357	252	94.3	2.54
40	141	42.2	1096	329	290	86.9	2.79	176	52.9	1172	352	310	93.0	2.58
48	161	40.1	1298	324	343	85.8	2.84	205	51.3	1393	348	368	92.1	2.60
56	181	38.7	1499	321	397	85.0	2.88	234	50.2	1614	346	427	91.5	2.62
64	201	37.6	1701	319	450	84.4	2.91	263	49.4	1835	344	485	91.0	2.64
72	221	36.8	1903	317	503	83.9	2.94	292	48.7	2056	343	544	90.6	2.65
96	281	35.1	2509	314	663	82.9	2.99	379	47.4	2719	340	719	89.9	2.68
120	341	34.1	3114	311	824	82.4	3.02	466	46.6	3382	338	895	89.5	2.69
8 Ungrouted	20.0	30.0	202	303	53.4	80.1	3.18	29.0	43.5	221	332	58.4	87.7	2.76

1. Based on $1^1/_4$ in. Face Shells and $^7/_{16}$ in. joints for $7^9/_{16}$ in. actual unit width with 4 cross-webs.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-8b Wall Section Properties of 8–Inch Concrete Masonry, Single Wythe Walls, 8–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1, 2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	60.0	90.1	293	440	77.0	115	2.21	60.0	90.1	293	440	77.0	115	2.21
16	82.6	62.0	505	379	132	99.3	2.47	89.9	65.6	516	387	135	101	2.43
24	103	51.3	711	355	186	93.2	2.63	117	57.4	737	369	193	96.7	2.53
32	123	46.0	916	344	240	90.2	2.73	145	53.3	959	360	252	94.4	2.60
40	143	42.8	1122	337	294	88.3	2.81	172	50.8	1182	354	310	92.9	2.64
48	163	40.6	1328	332	348	87.1	2.86	199	49.2	1403	351	368	92.0	2.67
56	183	39.1	1534	329	402	86.2	2.90	227	48.0	1625	348	426	91.3	2.69
64	203	38.0	1740	326	456	85.6	2.93	254	47.2	1847	346	484	90.8	2.71
72	223	37.1	1946	324	510	85.0	2.96	281	46.5	2068	345	543	90.4	2.72
96	283	35.3	2563	320	672	84.0	3.01	363	45.1	2734	342	717	89.6	2.75
120	343	34.3	3180	318	834	83.4	3.05	445	44.3	3400	340	892	89.2	2.77
8 Ungrouted	20.0	30.0	206	309	54.0	81.0	3.21	27.3	41.0	222	333	58.2	87.3	2.85
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	60.0	30.0	617	309	162	81.0	3.21	116	57.8	739	370	194	97.0	2.53
48	120	30.0	1235	309	324	81.0	3.21	198	49.4	1405	351	368	92.1	2.67
72	180	30.0	1852	309	486	81.0	3.21	280	46.6	2070	345	543	90.5	2.72
96	240	30.0	2470	309	648	81.0	3.21	362	45.2	2736	342	718	89.7	2.75
120	300	30.0	3087	309	810	81.0	3.21	444	44.4	3401	340	892	89.2	2.77
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	60.0	30.0	617	309	162	81.0	3.21	114	50.5	739	370	194	96.9	2.71
48	120	30.0	1235	309	324	81.0	3.21	194	40.3	1405	351	368	92.1	2.95
72	180	30.0	1852	309	486	81.0	3.21	274	36.8	2070	345	543	90.5	3.06
96	240	30.0	2470	309	648	81.0	3.21	354	35.1	2736	342	718	89.7	3.12
120	300	30.0	3087	309	810	81.0	3.21	434	34.1	3401	340	892	89.2	3.16
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
8 Solid	61.0	91.5	296	443	77.5	116	2.20	61.0	91.5	296	443	77.5	116	2.20
16	81.0	60.8	501	376	132	98.6	2.49	87.6	71.2	516	397	135	104	2.36
24	101	50.5	707	354	186	92.7	2.65	114	61.0	736	375	193	98.3	2.48
32	121	45.4	913	342	240	89.8	2.75	141	55.8	956	364	251	95.4	2.55
40	141	42.3	1118	336	293	88.0	2.82	167	52.8	1176	357	308	93.6	2.60
48	161	40.3	1324	331	347	86.9	2.87	194	50.7	1396	352	366	92.5	2.64
56	181	38.8	1530	328	401	86.0	2.91	221	49.2	1616	349	424	91.6	2.66
64	201	37.7	1736	326	455	85.4	2.94	247	48.1	1836	347	482	91.0	2.68
72	221	36.8	1942	324	509	84.9	2.96	274	47.3	2056	345	539	90.5	2.70
96	281	35.1	2559	320	671	83.9	3.02	354	45.6	2716	341	712	89.5	2.74
120	341	34.1	3177	318	833	83.3	3.05	434	44.6	3376	339	886	88.9	2.76
8 Ungrouted	20.0	30.0	206	309	54.0	81.0	3.21	26.6	40.5	220	330	57.7	86.6	2.86

1. Based on 1.25 in. Face Shells and in part on NCMA TEK Note 14-1B.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-10b Wall Section Properties of 10–Inch Concrete Masonry, Single Wythe Walls, 8–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1, 2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc³	ft	spc³	ft	spc³	ft	(in.)	spc³	ft	spc³	ft	spc³	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	75.7	114	589	883	122	184	2.79	75.7	114	589	883	122	184	2.79
16	99.7	74.8	959	719	199	149	3.10	111	80.5	991	744	206	154	3.04
24	120	59.8	1312	656	273	136	3.31	143	69.4	1393	697	290	145	3.17
32	140	52.4	1666	625	346	130	3.45	174	63.8	1795	673	373	140	3.25
40	160	47.9	2019	606	420	126	3.56	206	60.5	2197	659	457	137	3.30
48	180	44.9	2372	593	493	123	3.63	237	58.3	2599	650	540	135	3.34
56	200	42.8	2726	584	566	121	3.69	268	56.7	3000	643	623	134	3.37
64	220	41.2	3079	577	640	120	3.74	300	55.5	3402	638	707	133	3.39
72	240	40.0	3432	572	713	119	3.78	331	54.6	3804	634	790	132	3.41
96	300	37.5	4492	562	933	117	3.87	426	52.7	5009	626	1041	130	3.45
120	360	36.0	5552	555	1154	115	3.93	520	51.6	6215	621	1291	129	3.47
8 Ungrouted	20.0	30.0	353	530	73.4	110	4.20	31.5	47.2	402	603	83.5	125	3.57
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	60.0	30.0	1060	530	220	110	4.20	140	70.0	1398	699	290	146	3.16
48	120	30.0	2120	530	440	110	4.20	234	58.6	2603	651	541	135	3.33
72	180	30.0	3180	530	661	110	4.20	329	54.8	3809	635	791	132	3.40
96	240	30.0	4240	530	881	110	4.20	423	52.9	5014	627	1042	130	3.44
120	300	30.0	5300	530	1101	110	4.20	517	51.7	6219	622	1292	129	3.47
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	60.0	30.0	1060	530	220	110	4.20	138	58.5	1398	699	290	145	3.46
48	120	30.0	2120	530	440	110	4.20	229	44.3	2603	651	541	135	3.84
72	180	30.0	3180	530	661	110	4.20	320	39.5	3809	635	791	132	4.01
96	240	30.0	4240	530	881	110	4.20	411	37.1	5014	627	1042	130	4.11
120	300	30.0	5300	530	1101	110	4.20	502	35.7	6219	622	1292	129	4.17
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
8 Solid	77.0	116	594	892	124	185	2.78	77.0	116	594	892	124	185	2.78
16	97.0	72.8	948	711	197	148	3.13	107	89.1	993	778	206	162	2.96
24	117	58.5	1301	650	270	135	3.33	138	74.8	1391	718	289	149	3.10
32	137	51.4	1654	620	344	129	3.48	168	67.7	1789	688	372	143	3.19
40	157	47.1	2008	602	417	125	3.58	198	63.4	2188	670	455	139	3.25
48	177	44.3	2361	590	491	123	3.65	229	60.6	2586	658	537	137	3.29
56	197	42.2	2714	582	564	121	3.71	259	58.6	2984	649	620	135	3.33
64	217	40.7	3068	575	637	120	3.76	289	57.0	3382	643	703	134	3.36
72	237	39.5	3421	570	711	118	3.80	320	55.8	3781	638	786	132	3.38
96	297	37.1	4481	560	931	116	3.88	411	53.5	4976	628	1034	130	3.43
120	357	35.7	5541	554	1151	115	3.94	502	52.0	6171	622	1282	129	3.46
8 Ungrouted	20.0	30.0	353	530	73.4	110	4.20	30.3	46.3	398	597	82.8	124	3.59

1. Based on 1.25 in. Face Shells and in part on NCMA TEK Note 14-1B.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-12a.4 Wall Section Properties of 12–Inch Clay Masonry, Single Wythe, 4–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1,2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)	spc[3]	ft	spc[3]	ft	spc[3]	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	92.0	138	1014	1521	176	265	3.32	92.0	138	1014	1521	176	265	3.32
16	116	87.0	1618	1214	281	211	3.74	135	101	1730	1298	301	226	3.59
24	140	70.0	2223	1111	387	193	3.98	177	88.6	2447	1223	426	213	3.72
32	164	61.5	2827	1060	492	184	4.15	220	82.4	3163	1186	550	206	3.79
40	188	56.4	3432	1030	597	179	4.27	262	78.7	3880	1164	675	202	3.85
48	212	53.0	4036	1009	702	175	4.36	305	76.2	4596	1149	799	200	3.88
56	236	50.6	4641	994	807	173	4.43	348	74.5	5313	1138	924	198	3.91
64	260	48.8	5245	984	912	171	4.49	390	73.2	6029	1130	1049	197	3.93
72	284	47.3	5850	975	1017	170	4.54	433	72.1	6746	1124	1173	196	3.95
96	356	44.5	7663	958	1333	167	4.64	561	70.1	8895	1112	1547	193	3.98
120	428	42.8	9477	948	1648	165	4.71	688	68.8	11044	1104	1921	192	4.01
8 Ungrouted	24.0	36.0	605	907	105	158	5.02	42.6	63.9	716	1075	125	187	4.10
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	72.0	36.0	1814	907	315	158	5.02	152	76.2	2298	1149	400	200	3.88
48	144	36.0	3627	907	631	158	5.02	280	70.1	4447	1112	773	193	3.98
72	216	36.0	5441	907	946	158	5.02	408	68.0	6597	1099	1147	191	4.02
96	288	36.0	7254	907	1262	158	5.02	536	67.0	8746	1093	1521	190	4.04
120	360	36.0	9068	907	1577	158	5.02	664	66.4	10895	1090	1895	189	4.05
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	72.0	36.0	1814	907	315	158	5.02	177	88.6	2447	1223	426	216	3.72
48	144	36.0	3627	907	631	158	5.02	305	76.2	4596	1149	799	200	3.88
72	216	36.0	5441	907	946	158	5.02	433	72.1	6746	1124	1173	196	3.95
96	288	36.0	7254	907	1262	158	5.02	561	70.1	8895	1112	1547	193	3.98
120	360	36.0	9068	907	1577	158	5.02	688	68.8	11044	1104	1921	192	4.01
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
4 Solid	46.0	138	507	1521	88.2	265	3.32	46.0	138	507	1521	88.2	265	3.32
8	58.0	87.0	809	1214	141	211	3.74	67.3	101	865	1298	150	226	3.59
16	82.0	61.5	1414	1060	246	184	4.15	110	82.4	1582	1186	275	206	3.79
24	106	53.0	2018	1009	351	175	4.36	152	76.2	2298	1149	400	200	3.88
32	130	48.8	2623	984	456	171	4.49	195	73.2	3015	1130	524	197	3.93
40	154	46.2	3227	968	561	168	4.58	238	71.3	3731	1119	649	195	3.96
48	178	44.5	3832	958	666	167	4.64	280	70.1	4447	1112	773	193	3.98
56	202	43.3	4436	951	772	165	4.69	323	69.2	5164	1107	898	192	4.00
64	226	42.4	5041	945	877	164	4.72	365	68.5	5880	1103	1023	192	4.01
72	250	41.7	5645	941	982	164	4.75	408	68.0	6597	1099	1147	191	4.02
96	322	40.3	7459	932	1297	162	4.81	536	67.0	8746	1093	1521	190	4.04
120	394	39.4	9272	927	1613	161	4.85	664	66.4	10895	1090	1895	189	4.05
4 Ungrouted	12.0	36.0	302	907	52.6	158	5.02	21.3	63.9	358	1075	62.3	187	4.10

1. Based on 11 1/2 in. Face Shells and 1/2 in. joints for 11 1/2 in. actual unit width with 4 cross-webs and 1 long center web.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-12a.8 Wall Section Properties of 12–Inch Clay Masonry, Single Wythe, 8–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1,2]

Grout Spacing (in.)	A_{net}		I_{net}		S_{net}		r_{net}	A_{avg}		I_{avg}		S_{avg}		r_{avg}
	in.² per		in.⁴ per		in.³ per			in.² per		in.⁴ per		in.³ per		
	spc³	ft	spc³	ft	spc³	ft	(in.)	spc³	ft	spc³	ft	spc³	ft	(in.)
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	92.0	138	1014	1521	176	265	3.32	92.0	138	1014	1521	176	265	3.32
16	116	87.0	1618	1214	281	211	3.74	136	102	1738	1304	302	227	3.58
24	140	70.0	2223	1111	387	193	3.98	180	89.9	2463	1231	428	214	3.70
32	164	61.5	2827	1060	492	184	4.15	224	83.9	3187	1195	554	208	3.77
40	188	56.4	3432	1030	597	179	4.27	268	80.3	3912	1174	680	204	3.82
48	212	53.0	4036	1009	702	175	4.36	312	77.9	4636	1159	806	202	3.86
56	236	50.6	4641	994	807	173	4.43	356	76.2	5361	1149	932	200	3.88
64	260	48.8	5245	984	912	171	4.49	399	74.9	6085	1141	1058	198	3.90
72	284	47.3	5850	975	1017	170	4.54	443	73.9	6809	1135	1184	197	3.92
96	356	44.5	7663	958	1333	167	4.64	575	71.9	8983	1123	1562	195	3.95
120	428	42.8	9477	948	1648	165	4.71	707	70.7	11156	1116	1940	194	3.97
8 Ungrouted	24.0	36.0	605	907	105	158	5.02	43.9	65.9	724	1087	126	189	4.06
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	72.0	36.0	1814	907	315	158	5.02	180	89.9	2463	1231	428	214	3.70
48	144	36.0	3627	907	631	158	5.02	312	77.9	4636	1159	806	202	3.86
72	216	36.0	5441	907	946	158	5.02	443	73.9	6809	1135	1184	197	3.92
96	288	36.0	7254	907	1262	158	5.02	575	71.9	8983	1123	1562	195	3.95
120	360	36.0	9068	907	1577	158	5.02	707	70.7	11156	1116	1940	194	3.97
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	72.0	36.0	1814	907	315	158	5.02	180	89.9	2463	1231	428	214	3.70
48	144	36.0	3627	907	631	158	5.02	312	77.9	4636	1159	806	202	3.86
72	216	36.0	5441	907	946	158	5.02	443	73.9	6809	1135	1184	197	3.92
96	288	36.0	7254	907	1262	158	5.02	575	71.9	8983	1123	1562	195	3.95
120	360	36.0	9068	907	1577	158	5.02	707	70.7	11156	1116	1940	194	3.97
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
8 Solid	92.0	138	1014	1521	176	265	3.32	92.0	138	1014	1521	176	265	3.32
16	116	87.0	1618	1214	281	211	3.74	136	102	1738	1304	302	227	3.58
24	140	70.0	2223	1111	387	193	3.98	180	89.9	2463	1231	428	214	3.70
32	164	61.5	2827	1060	492	184	4.15	224	83.9	3187	1195	554	208	3.77
40	188	56.4	3432	1030	597	179	4.27	268	80.3	3912	1174	680	204	3.82
48	212	53.0	4036	1009	702	175	4.36	312	77.9	4636	1159	806	202	3.86
56	236	50.6	4641	994	807	173	4.43	356	76.2	5361	1149	932	200	3.88
64	260	48.8	5245	984	912	171	4.49	399	74.9	6085	1141	1058	198	3.90
72	284	47.3	5850	975	1017	170	4.54	443	73.9	6809	1135	1184	197	3.92
96	356	44.5	7663	958	1333	167	4.64	575	71.9	8983	1123	1562	195	3.95
120	428	42.8	9477	948	1648	165	4.71	707	70.7	11156	1116	1940	194	3.97
8 Ungrouted	24.0	36.0	605	907	105	158	5.02	43.9	65.9	724	1087	126	189	4.06

1. Based on 1 1/2 in. Face Shells and 1/2 in. joints for 11 1/2 in. actual unit width with 4 cross-webs.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-12b Wall Section Properties of 12–Inch Concrete Masonry, Single Wythe Walls, 8–Inch High, 16–Inch Long Masonry Units, Face Shell Bedding[1, 2]

Grout Spacing (in.)	A_{net} in.² per		I_{net} in.⁴ per		S_{net} in.³ per		r_{net} (in.)	A_{avg} in.² per		I_{avg} in.⁴ per		S_{avg} in.³ per		r_{avg} (in.)
	spc³	ft	spc³	ft	spc³	ft		spc³	ft	spc³	ft	spc³	ft	
	Horizontal Section Properties (Grouted Vertically, Masonry Spanning Vertically)													
8 Solid	91.3	137	1035	1553	178	267	3.37	91.3	137	1035	1553	178	267	3.37
16	116	87.3	1612	1209	277	208	3.72	131	94.6	1680	1260	289	217	3.65
24	136	68.2	2153	1076	370	185	3.97	166	80.4	2322	1161	400	200	3.80
32	156	58.7	2694	1010	463	174	4.15	200	73.3	2965	1112	510	191	3.89
40	176	52.9	3234	970	556	167	4.28	235	69.1	3608	1082	621	186	3.96
48	196	49.1	3775	944	649	162	4.38	270	66.2	4250	1063	731	183	4.01
56	216	46.4	4316	925	743	159	4.47	304	64.2	4893	1048	842	180	4.04
64	236	44.3	4857	911	836	157	4.53	339	62.7	5536	1038	952	179	4.07
72	256	42.7	5398	900	929	155	4.59	374	61.5	6178	1030	1063	177	4.09
96	316	39.6	7020	878	1208	151	4.71	478	59.1	8106	1013	1395	174	4.14
120	376	37.6	8642	864	1487	149	4.79	582	57.7	10034	1003	1726	173	4.17
8 Ungrouted	20.0	30.0	541	811	93.0	140	5.20	34.7	52.0	643	964	111	166	4.30
	Horizontal Section Properties (Grouted Horizontally, Masonry Spanning Vertically)													
24	60.0	30.0	1622	811	279	140	5.20	162	81.2	2333	1166	401	201	3.79
48	120	30.0	3245	811	558	140	5.20	266	66.6	4261	1065	733	183	4.00
72	180	30.0	4867	811	837	140	5.20	370	61.7	6188	1031	1065	177	4.09
96	240	30.0	6490	811	1117	140	5.20	474	59.3	8116	1015	1396	175	4.14
120	300	30.0	8112	811	1396	140	5.20	578	57.8	10044	1004	1728	173	4.17
	Vertical Section Properties (Grouted Vertically, Masonry Spanning Horizontally)													
24	60.0	30.0	1622	811	279	140	5.20	160	66.5	2333	1166	401	201	4.19
48	120	30.0	3245	811	558	140	5.20	259	48.3	4261	1065	733	183	4.70
72	180	30.0	4867	811	837	140	5.20	359	42.2	6188	1031	1065	177	4.95
96	240	30.0	6490	811	1116	140	5.20	459	39.1	8116	1015	1396	174	5.09
120	300	30.0	8112	811	1396	140	5.20	558	37.3	10044	1004	1728	173	5.19
	Vertical Section Properties (Grouted Horizontally, Masonry Spanning Horizontally)													
8 Solid	93.0	140	1047	1571	180	270	3.36	93.0	140	1047	1571	180	270	3.36
16	113	84.8	1588	1191	273	205	3.75	126	106	1685	1336	290	230	3.56
24	133	66.5	2129	1064	366	183	4.00	160	87.4	2322	1209	400	208	3.72
32	153	57.4	2670	1001	459	172	4.18	193	78.3	2959	1146	509	197	3.83
40	173	51.9	3211	963	552	166	4.31	226	72.8	3596	1108	619	191	3.90
48	193	48.3	3751	938	645	161	4.41	259	69.2	4234	1083	728	186	3.96
56	213	45.6	4292	920	738	158	4.49	292	66.6	4871	1064	838	183	4.00
64	233	43.7	4833	906	832	156	4.55	326	64.6	5508	1051	948	181	4.03
72	253	42.2	5374	896	924	154	4.61	359	63.1	6146	1040	1057	179	4.06
96	313	39.1	6996	874	1204	150	4.73	459	60.1	8058	1019	1386	175	4.12
120	373	37.3	8619	862	1483	148	4.81	558	58.2	9970	1007	1715	173	4.16
8 Ungrouted	20.0	30.0	541	811	93.0	140	5.20	33.2	50.9	637	956	110	164	4.33

1. Based on 1.25 in. Face Shells and in part on NCMA TEK Note 14-1B.
2. Net section properties are averaged over the grout spacing in the plane of the section. Average section properties are averaged over the span and are intended for stiffness or displacement computations only.
3. Properties in noted columns are based on a length of wall section equal to the grout spacing. (spc = spacing)

Table GN-17 Approximate Measurements of Masonry Materials

Item	Weight (lbs)	Volume (cu. ft)
Portland Cement		
1 Bag of Portland Cement	94	1.0
1, 12-Quart Bucket of Portland Cement	38	0.4
Lime		
1 Cubic Foot of Lime Putty[1]	80	1.0
1, 12-Quart Bucket of Lime Putty	30	0.37
6.5 Full No. 2 Shovels of Lime Putty	80	1.0
1 Cubic Foot of Hydrated Lime	40	1.0
100 Pounds of Hydrated Lime make the following volume of Lime Putty	100	2.18
1 Cubic Foot of Quicklime	60	1.0
100 Pounds of Quicklime makes the following volume of Lime Putty	100	3.69
Sand[2]		
1 Cubic Yard of Sand	2700	1.0
1 Ton of Sand	200	0.75
1 Cubic Foot of Sand	100	1.0
1, 12-Quart Bucket of Sand	40	0.4

1. Made from approximately 45.8 lbs of hydrated lime or 27.3 lbs of quicklime.
2. Weight varies with moisture content.

Table GN-18a Approximate Grout Quantities in Clay Masonry Walls[1, 2]

Nominal Thickness of Clay Masonry Units[3] (in.)	Spacing of Grouted Cells and Vertical Reinforcing Bars	4 inch High Units			8 inch High Units		
		Cubic Yards of Grout[2]		Number of Units[3] Filled	Cubic Yards of Grout[2]		Number of Units[3] Filled
		per 100 Square Feet of Wall	per 100 Units[3]	per Cubic Yard of Grout	per 100 Square Feet of Wall	per 100 Units[3]	per Cubic Yard of Grout
4[4]	Solid	0.31	0.14	731	0.31	0.27	366
	16”	0.15	0.07	1463	0.15	0.14	731
	24”	0.12	0.05	1925	0.12	0.10	962
	32”	0.10	0.04	2286	0.10	0.09	1143
	40”	0.09	0.04	2575	0.09	0.08	1288
	48”	0.08	0.04	2813	0.08	0.07	1407
5[5]	Solid	0.50	0.11	897	0.50	0.22	449
	10”	0.25	0.06	1794	0.25	0.11	897
	20”	0.18	0.04	2473	0.18	0.08	1237
	30”	0.15	0.03	3050	0.15	0.07	1525
	40”	0.13	0.03	3547	0.13	0.06	1773
	48”	0.11	0.03	3979	0.11	0.05	1989
6[4]	Solid	0.72	0.32	313	0.72	0.64	156
	16”	0.36	0.16	626	0.36	0.32	313
	24”	0.26	0.12	853	0.26	0.23	426
	32”	0.22	0.10	1043	0.22	0.19	521
	40”	0.19	0.08	1203	0.19	0.17	601
	48”	0.17	0.07	1340	0.17	0.15	670
8[4]	Solid	1.19	0.53	189	1.19	1.06	95
	16”	0.59	0.26	378	0.59	0.53	189
	24”	0.44	0.19	516	0.44	0.39	258
	32”	0.36	0.16	631	0.36	0.32	315
	40”	0.31	0.14	728	0.31	0.27	364
	48”	0.28	0.12	811	0.28	0.25	405
12[4]	Solid	1.76	0.78	128	1.76	1.57	64
	16”	0.88	0.39	255	0.88	0.78	128
	24”	0.64	0.28	352	0.64	0.57	176
	32”	0.52	0.23	433	0.52	0.46	217
	40”	0.45	0.20	504	0.45	0.40	252
	48”	0.40	0.18	565	0.40	0.35	282

1. Table is based on running bond (no grout in middle cells) and horizontal beams at 4 foot on center.
2. Table includes a 3 percent allowance for grout loss and various job conditions.
3. For open end units increase the approximate quantities of grout required by about 10 percent. For slumped block reduce the above grout quantities by 5 percent.
4. Based on 16” long clay masonry units.
5. Based on 10” long clay masonry units.

Table GN-18b Approximate Grout Quantities in Concrete Masonry Walls[1]

Thickness of Standard Two Cell Concrete Masonry Units[3] (in.)	Spacing of Grouted Cells and Vertical Reinforcing Bars	Cubic Yards of Grout[2] per 100 Square Feet of Wall	Cubic Yards of Grout[2] per 100 Block Units[4]	Number of Block Units[4] Filled per Cubic Yard of Grout
6	Solid	0.93	0.83	120
	16”	0.55	0.49	205
	24”	0.42	0.37	270
	32”	0.35	0.31	320
	40”	0.31	0.28	360
	48”	0.28	0.25	396
8	Solid	1.12	1.00	100
	16”	0.65	0.58	171
	24”	0.50	0.44	225
	32”	0.43	0.38	267
	40”	0.37	0.33	300
	48”	0.34	0.30	330
10	Solid	1.38	1.23	80
	16”	0.82	0.73	137
	24”	0.63	0.56	180
	32”	0.53	0.47	214
	40”	0.47	0.42	240
	48”	0.43	0.38	264
12	Solid	1.73	1.54	65
	16”	1.01	0.90	111
	24”	0.76	0.68	146
	32”	0.64	0.57	174
	40”	0.57	0.51	195
	48”	0.53	0.47	215

1. Table includes horizontal bond beams at 4 foot on center.
2. Table includes a 3 percent allowance for grout loss and various job conditions.
3. For open end block increase the approximate quantities of grout required by about 10 percent. For slumped block reduce the above grout quantities by 5 percent.
4. Based on standard 8” high by 16” long concrete masonry units.

Table GN-18c Approximate Grout Quantities Needed in 2 Wythe Brick Wall Construction

Width of Grout Space (inches)	Cubic Yards of Grout[1] per 100 Square Feet of Wall	Square Feet of Wall Filled per Cubic Yard of Grout[1]
2.0	0.64	157
2.5	0.79	126
3.0	0.96	105
3.5	1.11	90
4.0	1.27	79
4.5	1.43	70
5.0[2]	1.59	63
5.5[2]	1.75	57
6.0[2]	1.91	52
6.5[2]	2.07	48
7.0[2]	2.23	45
8.0[2]	2.54	39

1. Table includes a 3 percent allowance for grout loss and various job conditions.
2. When the width of the grout space is 5” or more, it is advisable to use floaters during low lift grouting.

Table GN-19a Properties of Standard Steel Reinforcing Bars

Bar Designation Number[2]	Nominal Weight (lbs/ft)	Nominal Dimensions[1]			Deformation Requirements (inches)		
		Diameter (inches)	Cross-Sectional Area (sq. in.)	Perimeter (inches)	Maximum Average Spacing	Minimum Average Height	Maximum Gap (chord of 12 1/2% of nominal perimeter)
#3	0.376	0.375	0.11	1.178	0.262	0.015	0.143
#4	0.668	0.500	0.20	1.571	0.350	0.020	0.191
#5	1.043	0.625	0.31	1.963	0.437	0.028	0.239
#6	1.502	0.750	0.44	2.356	0.525	0.038	0.286
#7	2.044	0.875	0.60	2.749	0.612	0.044	0.334
#8	2.670	1.000	0.79	3.142	0.700	0.050	0.383
#9	3.400	1.128	1.00	3.544	0.790	0.056	0.431
#10	4.303	1.270	1.27	3.990	0.889	0.064	0.487
#11	5.313	1.410	1.56	4.430	0.987	0.071	0.540
#14	7.650	1.693	2.25	5.319	1.185	0.085	0.648
#18	13.600	2.257	4.00	7.091	1.580	0.102	0.864

1. The nominal dimension of a deformed bar is equivalent to that of a plain bar having the same weight per foot as the deformed bar.
2. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-19b SI Properties of Standard Steel Reinforcing Bars[1,3] (Soft Metric Bar Properties)

Bar Size[2]	Weight (kg·m)	Diameter (mm)	Area (mm^2)	Perimeter (mm)
#10 (#3)	0.560	9.5	71	29.9
#13 (#4)	0.994	12.7	129	39.9
#16 (#5)	1.552	15.9	199	49.9
#19 (#6)	2.235	19.1	284	59.8
#22 (#7)	3.042	22.2	387	69.8
#25 (#8)	3.973	25.4	510	79.8
#29 (#9)	5.060	28.7	645	90.0
#32 (#10)	6.404	32.3	819	101.3
#36 (#11)	7.907	35.8	1006	112.5
#43 (#14)	11.38	43.0	1452	135.1
#57 (#18)	20.24	57.3	2581	180.1

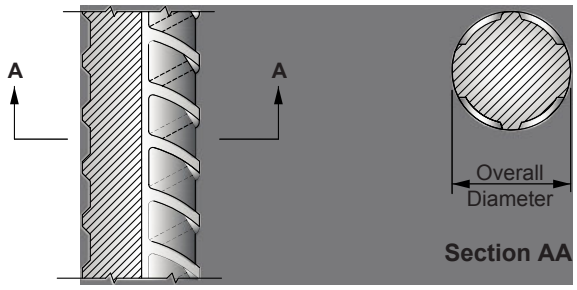
1. Based on ASTM A615/A615M Table 1.
2. MSJC Code limits the maximum bar size to #36 (#11) (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #29 (#9) (MSJC Code Section 3.3.3.1).
3. Equivalent soft metric identification and properties for non-metric bars.

Table GN-19c SI Properties of Standard Metric Steel Reinforcing Bars[1,3]

Bar Size	Weight (kg·m)	Diameter (mm)	Area (mm^2)	Perimeter (mm)
#10	0.785	11.3	100	35.5
#15	1.570	16.0	200	50.3
#20	2.355	19.5	300	61.3
#25	3.925	25.2	500	79.2
#30	5.495	29.9	700	93.9
#35	7.850	35.7	1000	112.2
#45	11.775	43.7	1500	137.3
#55	19.625	56.4	2500	177.2

1. Based on ASTM A615/A615M Table 1.
2. MSJC Code limits the maximum bar size to #36 (#11) (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #29 (#9) (MSJC Code Section 3.3.3.1).
3. Actual metric reinforcement - See Table GN-19b for soft metric properties of non-metric reinforcement.

Table GN-19d Overall Diameter of Bars



Bar Size	Approx. Dia. to Outside of Deformations[1] (inches)	Bar Size[2]	Approx. Dia. to Outside of Deformations[1] (inches)
#3	7/16	**#8**	1 1/8
#4	9/16	**#9**	1 1/4
#5	11/16	**#10**	1 7/16
#6	7/8	**#11**	1 5/8
#7	1	**#14**	1 7/8
		#18	2 1/2

1. Diameters tabulated are the approximate dimension to the outside of the deformations.
2. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-19e Properties of Steel Reinforcing Wire[1]

Steel Wire Gage	Diameter (inches)	Perimeter (inches)	Area (square inches)	Ultimate Strength (lbs)	Weight (plf)	Allowable Tension Load @ 30,000 psi (lbs)	Allowable Tension Load @ 40,000 psi[2] (lbs)
14	0.080	0.251	0.005	375	0.017	150	200
13	0.092	0.287	0.007	525	0.022	198	264
12	0.106	0.330	0.009	675	0.030	261	348
11	0.120	0.377	0.011	825	0.039	342	456
10	0.135	0.424	0.014	1050	0.049	429	572
9[3]	0.148	0.465	0.017	1275	0.059	519	692
8[3]	0.162	0.509	0.021	1575	0.070	618	824
7	0.177	0.556	0.025	1875	0.084	738	984
3/16[3]	0.188	0.587	0.028	2100	0.094	828	1104
6	0.192	0.603	0.029	2175	0.098	867	1156
5	0.207	0.650	0.034	2550	0.114	1008	1344
4	0.225	0.707	0.040	3000	0.135	1197	1596
3	0.244	0.763	0.047	3525	0.158	1398	1864
1/4[3]	0.250	0.785	0.049	3675	0.167	1473	1964
2	0.262	0.823	0.054	4050	0.184	1623	2163
1	0.283	0.889	0.063	4725	0.214	1887	2515
1-0	0.306	0.961	0.074	5550	0.251	2214	2952
5/16[3]	0.313	0.980	0.077	5775	0.261	2301	3068
2-0	0.331	1.040	0.086	6450	0.292	2580	3440
3-0	0.362	1.137	0.103	7725	0.351	3096	4128

1. Based on the United States Steel Wire Gage and ASTM A951 with F_{su} = 75,000 psi min., F_y = 60,000 psi min. and F_s allowable = 30,000 psi.
2. Allowable tension loads increased 1/3 for wind and seismic loads.
3. Used for joint reinforcement.

Table GN-20a Areas of Various Combinations of Bars[1]

Areas, A_s (or A'_s) (top) sq. in.

Columns headed 0 5 contain data for bars of one size in groups of one to ten.

Columns headed 1 2 3 4 5 contain data for bars of two sizes with one to five bars of each size.

			0	5		1	2	3	4	5
	1		0.20	1.20		0.31	0.42	0.53	0.64	0.75
	2		0.40	1.40		0.51	0.62	0.73	0.84	0.95
#4	3	#4	0.60	1.60	#3	0.71	0.82	0.93	1.04	1.15
	4		0.80	1.80		0.91	1.02	1.13	1.24	1.35
	5		1.00	2.00		1.11	1.22	1.33	1.44	1.55

			0	5		1	2	3	4	5		1	2	3	4	5
	1		0.31	1.86		0.51	0.71	0.91	1.11	1.31		0.42	0.53	0.64	0.75	0.86
	2		0.62	2.17		0.82	1.02	1.22	1.42	1.62		0.73	0.84	0.95	1.06	1.17
#5	3	#5	0.93	2.48	#4	1.13	1.33	1.53	1.73	1.93	#3	1.04	1.15	1.26	1.37	1.48
	4		1.24	2.79		1.44	1.64	1.84	2.04	2.24		1.35	1.46	1.57	1.68	1.79
	5		1.55	3.10		1.75	1.95	2.15	2.35	2.55		1.66	1.77	1.88	1.99	2.10

			0	5		1	2	3	4	5		1	2	3	4	5		1	2	3	4	5
	1		0.44	2.64		0.75	1.06	1.37	1.68	1.99		0.64	0.84	1.04	1.24	1.44						
	2		0.88	3.08		1.19	1.50	1.81	2.12	2.43		1.08	1.28	1.48	1.68	1.88						
#6	3	#6	1.32	3.52	#5	1.63	1.94	2.25	2.56	2.87	#4	1.52	1.72	1.92	2.12	2.32						
	4		1.76	3.96		2.07	2.38	2.69	3.00	3.31		1.96	2.16	2.36	2.56	2.76						
	5		2.20	4.40		2.51	2.82	3.13	3.44	3.75		2.40	2.60	2.80	3.00	3.20						

			0	5		1	2	3	4	5		1	2	3	4	5		1	2	3	4	5
	1		0.60	3.60		1.04	1.48	1.92	2.36	2.80		0.91	1.22	1.53	1.84	2.15		0.80	1.00	1.20	1.40	1.60
	2		1.20	4.20		1.64	2.08	2.52	2.96	3.40		1.51	1.82	2.13	2.44	2.75		1.40	1.60	1.80	2.00	2.20
#7	3	#7	1.80	4.80	#6	2.24	2.68	3.12	3.56	4.00	#5	2.11	2.42	2.73	3.04	3.35	#4	2.00	2.20	2.40	2.60	2.80
	4		2.40	5.40		2.84	3.28	3.72	4.16	4.60		2.71	3.02	3.33	3.64	3.95		2.60	2.80	3.00	3.20	3.40
	5		3.00	6.00		3.44	3.88	4.32	4.76	5.20		3.31	3.62	3.93	4.24	4.55		3.20	3.40	3.60	3.80	4.00

			0	5		1	2	3	4	5		1	2	3	4	5		1	2	3	4	5
	1		0.79	4.74		1.39	1.99	2.59	3.19	3.79		1.23	1.67	2.11	2.55	2.99		1.10	1.41	1.72	2.03	2.34
	2		1.58	5.53		2.18	2.78	3.38	3.98	4.58		2.02	2.46	2.90	3.34	3.78		1.89	2.20	2.51	2.82	3.13
#8	3	#8	2.37	6.32	#7	2.97	3.57	4.17	4.77	5.37	#6	2.81	3.25	3.69	4.13	4.57	#5	2.68	2.99	3.30	3.61	3.92
	4		3.16	7.11		3.76	4.36	4.96	5.56	6.16		3.60	4.04	4.48	4.92	5.36		3.47	3.78	4.09	4.40	4.71
	5		3.95	7.90		4.55	5.15	5.75	6.35	6.95		4.39	4.83	5.27	5.71	6.15		4.26	4.57	4.88	5.19	5.50

			0	5		1	2	3	4	5		1	2	3	4	5		1	2	3	4	5
	1		1.00	6.00		1.79	2.58	3.37	4.16	4.95		1.60	2.20	2.80	3.40	4.00		1.44	1.88	2.32	2.76	3.20
	2		2.00	7.00		2.79	3.58	4.37	5.16	5.95		2.60	3.20	3.80	4.40	5.00		2.44	2.88	3.32	3.76	4.20
#9	3	#9	3.00	8.00	#8	3.79	4.58	5.37	6.16	6.95	#7	3.60	4.20	4.80	5.40	6.00	#6	3.44	3.88	4.32	4.76	5.20
	4		4.00	9.00		4.79	5.58	6.37	7.16	7.95		4.60	5.20	5.80	6.40	7.00		4.44	4.88	5.32	5.76	6.20
	5		5.00	10.00		5.79	6.58	7.37	8.16	8.95		5.60	6.20	6.80	7.40	8.00		5.44	5.88	6.32	6.76	7.20

			0	5		1	2	3	4	5		1	2	3	4	5		1	2	3	4	5
	1		1.27	7.62		2.27	3.27	4.27	5.27	6.27		2.06	2.85	3.64	4.43	5.22		1.87	2.47	3.07	3.67	4.27
	2		2.54	8.89		3.54	4.54	5.54	6.54	7.54		3.33	4.12	4.91	5.70	6.49		3.14	3.74	4.34	4.94	5.54
#10	3	#10	3.81	10.16	#9	4.81	5.81	6.81	7.81	8.81	#8	4.00	5.39	6.18	6.97	7.76	#7	4.41	5.01	5.61	6.21	6.81
	4		5.08	11.43		6.08	7.08	8.08	9.08	10.08		5.87	6.66	7.45	8.24	9.03		5.68	6.28	6.88	7.48	8.08
	5		6.35	12.70		7.35	8.35	9.35	10.35	11.35		7.14	7.93	8.72	9.51	10.30		6.95	7.55	8.15	8.75	9.35

			0	5		1	2	3	4	5		1	2	3	4	5		1	2	3	4	5
	1		1.56	9.36		2.83	4.10	5.37	6.64	7.91		2.56	3.56	4.56	5.56	6.56		2.35	3.14	3.93	4.72	5.51
	2		3.12	10.92		4.39	5.66	6.93	8.20	9.47		4.12	5.12	6.12	7.12	8.12		3.91	4.70	5.49	6.28	7.07
#11	3	#11	4.68	12.48	#10	5.95	7.22	8.49	9.76	11.03	#9	5.68	6.68	7.68	8.68	9.68	#8	5.47	6.26	7.05	7.84	8.63
	4		6.24	14.04		7.51	8.78	10.05	11.32	12.59		7.24	8.24	9.24	10.24	11.24		7.03	7.82	8.61	9.40	10.19
	5		7.80	15.60		9.07	10.34	11.61	12.88	14.15		8.80	9.80	10.80	11.80	12.80		8.59	9.38	10.17	10.96	11.75

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-20b Areas of Reinforcing Steel Per Foot for Various Spacing

Spacing			Bar Size[1]									Spacing
(in.)	(ft)	8" cells	#3	#4	#5	#6	#7	#8	#9	#10	#11	(in.)
2.00	0.167		0.66	1.20	1.86	2.64	3.60	4.74				2.00
2.25	0.188		0.59	1.07	1.65	2.35	3.20	4.21		Note[2]		2.25
2.50	0.208		0.53	0.96	1.49	2.11	2.88	3.79	4.80			2.50
2.75	0.229		0.48	0.87	1.35	1.92	2.62	3.45	4.36	5.54		2.75
3.00	0.250		0.44	0.80	1.24	1.76	2.40	3.16	4.00	5.08	6.24	3.00
3.25	0.271		0.41	0.74	1.14	1.62	2.22	2.92	3.69	4.69	5.76	3.25
3.50	0.292		0.38	0.69	1.06	1.51	2.06	2.71	3.43	4.35	5.35	3.50
3.75	0.313		0.35	0.64	0.99	1.41	1.92	2.53	3.20	4.06	4.99	3.75
4.00	0.333		0.33	0.60	0.93	1.32	1.80	2.37	3.00	3.81	4.68	4.00
4.25	0.354		0.31	0.56	0.88	1.24	1.69	2.23	2.82	3.59	4.40	4.25
4.50	0.375		0.29	0.53	0.83	1.17	1.60	2.11	2.67	3.39	4.16	4.50
4.75	0.396		0.28	0.51	0.78	1.11	1.52	2.00	2.53	3.21	3.94	4.75
5.00	0.417		0.26	0.48	0.74	1.06	1.44	1.90	2.40	3.05	3.74	5.00
5.25	0.438		0.25	0.46	0.71	1.01	1.37	1.81	2.29	2.90	3.57	5.25
5.50	0.458		0.24	0.44	0.68	0.96	1.31	1.72	2.18	2.77	3.40	5.50
5.75	0.479		0.23	0.42	0.65	0.92	1.25	1.65	2.09	2.65	3.26	5.75
6.00	0.500		0.22	0.40	0.62	0.88	1.20	1.58	2.00	2.54	3.12	6.00
6.50	0.542		0.20	0.37	0.57	0.81	1.11	1.46	1.85	2.34	2.88	6.50
7.00	0.583		0.19	0.34	0.53	0.75	1.03	1.35	1.71	2.18	2.67	7.00
7.50	0.625		0.18	0.32	0.50	0.70	0.96	1.26	1.60	2.03	2.50	7.50
8.00	0.667	1	0.17	0.30	0.47	0.66	0.90	1.19	1.50	1.91	2.34	8.00
8.50	0.708		0.16	0.28	0.44	0.62	0.85	1.12	1.41	1.79	2.20	8.50
9.00	0.750		0.15	0.27	0.41	0.59	0.80	1.05	1.33	1.69	2.08	9.00
9.50	0.792		0.14	0.25	0.39	0.56	0.76	1.00	1.26	1.60	1.97	9.50
10.0	0.833		0.13	0.24	0.37	0.53	0.72	0.95	1.20	1.52	1.87	10.0
10.5	0.875		0.13	0.23	0.35	0.50	0.69	0.90	1.14	1.45	1.78	10.5
11.0	0.917		0.12	0.22	0.34	0.48	0.65	0.86	1.09	1.39	1.70	11.0
11.5	0.958		0.11	0.21	0.32	0.46	0.63	0.82	1.04	1.33	1.63	11.5
12.0	1.000		0.11	0.20	0.31	0.44	0.60	0.79	1.00	1.27	1.56	12.0
13.0	1.083		0.10	0.18	0.29	0.41	0.55	0.73	0.92	1.17	1.44	13.0
14.0	1.167		0.09	0.17	0.27	0.38	0.51	0.68	0.86	1.09	1.34	14.0
15.0	1.250		0.09	0.16	0.25	0.35	0.48	0.63	0.80	1.02	1.25	15.0
16.0	1.333	2	0.08	0.15	0.23	0.33	0.45	0.59	0.75	0.95	1.17	16.0
18.0	1.500		0.07	0.13	0.21	0.29	0.40	0.53	0.67	0.85	1.04	18.0
20.0	1.667		0.07	0.12	0.19	0.26	0.36	0.47	0.60	0.76	0.94	20.0
22.0	1.833		0.06	0.11	0.17	0.24	0.33	0.43	0.55	0.69	0.85	22.0
24.0	2.000	3	0.06	0.10	0.16	0.22	0.30	0.40	0.50	0.64	0.78	24.0
30.0	2.500		0.04	0.08	0.12	0.18	0.24	0.32	0.40	0.51	0.62	30.0
32.0	2.667	4	0.04	0.08	0.12	0.17	0.23	0.30	0.38	0.48	0.59	32.0
36.0	3.000		0.04	0.07	0.10	0.15	0.20	0.26	0.33	0.42	0.52	36.0
40.0	3.333	5	0.03	0.06	0.09	0.13	0.18	0.24	0.30	0.38	0.47	40.0
42.0	3.500		0.03	0.06	0.09	0.13	0.17	0.23	0.29	0.36	0.45	42.0
48.0	4.000	6	0.03	0.05	0.08	0.11	0.15	0.20	0.25	0.32	0.39	48.0
54.0	4.500		0.02	0.04	0.07	0.10	0.13	0.18	0.22	0.28	0.35	54.0
56.0	4.667	7	0.02	0.04	0.07	0.09	0.13	0.17	0.21	0.27	0.33	56.0
60.0	5.000		0.02	0.04	0.06	0.09	0.12	0.16	0.20	0.25	0.31	60.0
64.0	5.333	8	0.02	0.04	0.06	0.08	0.11	0.15	0.19	0.24	0.29	64.0
66.0	5.500		0.02	0.04	0.06	0.08	0.11	0.14	0.18	0.23	0.28	66.0
72.0	6.000	9	0.02	0.03	0.05	0.07	0.10	0.13	0.17	0.21	0.26	72.0
78.0	6.500		0.02	0.03	0.05	0.07	0.09	0.12	0.15	0.20	0.24	78.0
80.0	6.667	10	0.02	0.03	0.05	0.07	0.09	0.12	0.15	0.19	0.23	80.0
84.0	7.000		0.02	0.03	0.04	0.06	0.09	0.11	0.14	0.18	0.22	84.0
88.0	7.333	11	0.02	0.03	0.04	0.06	0.08	0.11	0.14	0.17	0.21	88.0
90.0	7.500		0.01	0.03	0.04	0.06	0.08	0.11	0.13	0.17	0.21	90.0
96.0	8.000	12	0.01	0.03	0.04	0.06	0.08	0.10	0.13	0.16	0.20	96.0

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).
2. Limited by the one inch or one diameter clear spacing minimum MSJC Code Section 1.13.3.1.

Table GN-20c Areas of Reinforcing Steel per Foot (square inches)

Reinforcement			Reinforcement Spacing (inches)								
Size[1]	Diameter (inches)	Area (sq. in.)	8	10	12	14	16	18	20	22	24
			0’ - 8”	0’ - 10”	1’ - 0”	1’ - 2”	1’ - 4”	1’ - 6”	1’ - 8”	1’ - 10”	2’ - 0”
2 - #9 wires	0.1480	0.035	0.052	0.041	0.035	0.030	0.026	0.023	0.021	0.019	0.017
2 - #8 wires	0.1620	0.041	0.062	0.049	0.041	0.035	0.031	0.027	0.025	0.022	0.021
2 - 3/16” wires	0.1875	0.055	0.083	0.066	0.055	0.047	0.041	0.037	0.033	0.030	0.028
2 - 1/4” wires	0.2500	0.098	0.147	0.118	0.098	0.084	0.074	0.065	0.059	0.053	0.049
2 - 5/16” wires	0.3125	0.152	0.228	0.182	0.152	0.130	0.114	0.101	0.091	0.083	0.076
#2	1/4	0.049	0.074	0.059	0.049	0.042	0.037	0.033	0.029	0.027	0.025
#3	3/8	0.110	0.165	0.132	0.110	0.094	0.083	0.073	0.066	0.060	0.055
#4	1/2	0.200	0.294	0.235	0.196	0.168	0.147	0.131	0.118	0.107	0.098
#5	5/8	0.310	0.461	0.368	0.307	0.263	0.230	0.205	0.184	0.167	0.154
#6	3/4	0.440	0.663	0.530	0.442	0.379	0.332	0.295	0.265	0.241	0.221
#7	7/8	0.600	0.902	0.721	0.601	0.515	0.451	0.401	0.361	0.328	0.301
#8	1.0	0.790	1.179	0.943	0.786	0.674	0.590	0.524	0.472	0.429	0.393
#9	1 1/8	1.000	1.500	1.200	1.000	0.857	0.750	0.667	0.600	0.545	0.500
#10	1 1/4	1.270	1.905	1.524	1.270	1.089	0.953	0.847	0.762	0.693	0.635
#11	1 3/8	1.560	2.340	1.872	1.560	1.337	1.170	1.040	0.936	0.851	0.780

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-20d Areas of Reinforcing Steel per Foot (square inches)

Reinforcement			Reinforcement Spacing (inches)								
Size[1]	Diameter (inches)	Area (sq. in.)	26	28	30	32	34	36	40	44	48
			2'- 2"	2' - 4"	2" - 6"	2' - 8"	2' - 10"	3' - 0"	3' - 4"	3' - 8"	4' - 0
2 - #9 wires	0.1480	0.035	0.016	0.015	0.014	0.013	0.012	0.012	0.010	0.009	0.009
2 - #8 wires	0.1620	0.041	0.019	0.018	0.016	0.015	0.015	0.014	0.012	0.011	0.010
2 - 3/16" wires	0.1875	0.055	0.025	0.024	0.022	0.021	0.019	0.018	0.017	0.015	0.014
2 - 1/4" wires	0.2500	0.098	0.045	0.042	0.039	0.037	0.035	0.033	0.029	0.027	0.025
2 - 5/16" wires	0.3125	0.152	0.070	0.065	0.061	0.057	0.054	0.051	0.046	0.041	0.038
#2	1/4	0.049	0.023	0.021	0.020	0.018	0.017	0.016	0.015	0.013	0.012
#3	3/8	0.110	0.051	0.047	0.044	0.041	0.039	0.037	0.033	0.030	0.028
#4	1/2	0.200	0.090	0.084	0.078	0.074	0.069	0.065	0.059	0.053	0.049
#5	5/8	0.310	0.142	0.132	0.123	0.115	0.108	0.102	0.092	0.084	0.077
#6	3/4	0.440	0.204	0.189	0.177	0.166	0.156	0.147	0.133	0.121	0.111
#7	7/8	0.600	0.277	0.258	0.240	0.225	0.212	0.200	0.180	0.164	0.150
#8	1.0	0.790	0.363	0.337	0.314	0.295	0.277	0.262	0.236	0.214	0.197
#9	1 1/8	1.000	0.462	0.429	0.400	0.375	0.353	0.333	0.300	0.273	0.250
#10	1 1/4	1.270	0.586	0.544	0.508	0.476	0.448	0.423	0.381	0.346	0.318
#11	1 3/8	1.560	0.720	0.669	0.624	0.585	0.551	0.520	0.468	0.425	0.390

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-21a Maximum Spacing (inches) of Minimum Reinforcing Steel, $A_s = 0.0007bt$[1,2]

Actual Wall Thickness, *t* (inches)	Min. Area[1,2] (sq. in./ft)	Reinforcing Bar Size						
		#3	#4	#5	#6	#7	#8	#9
3.50	0.029	45	48					
3.63	0.030	43	48					
3.75	0.032	42	48					
4.00	0.034	39	48					
4.50	0.038	35	48					
5.00	0.042	31	48					
5.25	0.044	30	48					
5.50	0.046	29	48				See Footnote 3	
5.63	0.047	28	48					
5.75	0.048	27	48					
6.00	0.050	26	48					
6.25	0.053	25	46	48				
6.50	0.055	24	44	48				
6.75	0.057	23	42	48				
7.00	0.059	22	41	48				
7.25	0.061	21	39	48				
7.50	0.063	20	38	48				
7.63	0.064	20	37	48				
7.75	0.065	20	36	48				
8.00	0.067	20	36	48				
8.50	0.071	18	34	48				
8.75	0.074	18	33	48				
9.00	0.076	17	32	48	48			
9.50	0.080	17	30	47	48			
9.63	0.081	16	30	46	48			
10.00	0.084	16	29	44	48			
10.50	0.088	15	27	42	48			
11.00	0.092	14	26	40	48			
11.50	0.097	14	25	39	48			
11.63	0.098	13	24	38	48			
12.00	0.101	13	24	37	48			
12.50	0.105	13	23	35	48			
13.00	0.109	12	22	34	48			
13.50	0.113	12	21	33	47	48		
14.00	0.118	11	20	32	45	48		
14.50	0.122	11	20	31	43	48		
15.00	0.126	10	19	30	42	48		
15.50	0.130	10	18	29	41	48		
15.63	0.131	10	18	28	40	48		
16.00	0.134	10	18	28	39	48		

1. Reinforcing steel spacing shown will provide required area of steel based on ρ_{min} = 0.0007. To be conservative, all spacing values shown were truncated to the nearest lower inch.
2. Minimum area of principal reinforcement may be less than 0.0013*bt* but may not be less than 0.0007*bt* per MSJC Code Section 1.14.6.3 The sum of the horizontal and vertical reinforcement must be at least 0.002*bt* which is 0.002 times the gross sectional area.
3. Values shown to the right of the heavy zigzag line are limited to a maximum spacing of 4 ft (48 in.) o. c. per MSJC Code Section 1.14.6.3. Values to the right of the zigzag line which are less than 48 in. are limited to 6 times the thickness per MSJC Code Section 2.3.3.3.1.

Table GN-21b Maximum Spacing (inches) Based on Reinforcing Steel, $A_s = 0.0013bt$[1,2]

Actual Wall Thickness, *t* (inches)	Area A_s[1,2] (sq. in./ft)	Reinforcing Bar Size						
		#3	#4	#5	#6	#7	#8	#9
3.50	0.055	21						
3.63	0.057	22						
3.75	0.059	23	23					
4.00	0.062	21	24					
4.50	0.070	19	27					
5.00	0.078	17	30	30				
5.25	0.082	16	29	31				
5.50	0.086	15	28	33			See Footnote 3	
5.63	0.088	15	27	34				
5.75	0.090	14	27	35				
6.00	0.094	14	26	36				
6.25	0.098	14	25	37				
6.50	0.101	13	24	37	39			
6.75	0.105	13	23	35	40			
7.00	0.109	12	22	34	42			
7.25	0.113	11	21	33	44			
7.50	0.117	11	21	32	45	48		
7.63	0.119	11	20	31	44	48		
7.75	0.121	11	20	31	44	48		
8.00	0.125	11	19	30	42	48		
8.50	0.133	10	18	28	40	48		
8.75	0.137	9	18	27	39	48		
9.00	0.140	9	17	26	38	48		
9.50	0.148	9	16	25	36	48		
9.63	0.150	8	15	24	35	48	48	
10.00	0.156	8	15	24	34	46	48	
10.50	0.164	8	15	23	32	44	48	
11.00	0.172	8	14	22	31	42	48	
11.50	0.179	7	13	21	29	40	48	
11.63	0.181	7	13	20	29	39	48	
12.00	0.187	7	13	20	28	38	48	
12.50	0.195	7	12	19	27	37	48	
13.00	0.203	7	12	18	26	36	47	48
13.50	0.211	6	11	18	25	34	45	48
14.00	0.218	6	11	17	24	33	43	48
14.50	0.226	6	11	16	23	32	42	48
15.00	0.234	6	10	16	23	31	41	48
15.50	0.242	5	10	15	22	30	39	48
15.63	0.244	5	10	15	22	30	38	48
16.00	0.250	5	10	15	21	29	38	48

1. Reinforcing steel spacing shown will provide required area of steel based on $\rho = 0.0013$. To be conservative, all spacing values shown were truncated to the nearest lower inch.
2. Minimum area of principal reinforcement may be less than $0.0013bt$ but may not be less than $0.0007bt$ per MSJC Code Section 1.14.6.3 The sum of the horizontal and vertical reinforcement must be at least $0.002bt$ which is 0.002 times the gross sectional area.
3. Values shown to the right of the heavy zigzag line are limited to a maximum spacing of 4 ft (48 in.) o. c. per MSJC Code Section 1.14.6.3. Values to the right of the zigzag line which are less than 48 in. are limited to 6 times the thickness per MSJC Code Section 2.3.3.3.1.

Table GN-21c Maximum Spacing (inches) Based on Reinforcing Steel, $A_s = 0.001bt$[1,2]

Actual Wall Thickness, *t* (inches)	Area A_s[1,2] (sq. in./ft)	Reinforcing Bar Size						
		#3	#4	#5	#6	#7	#8	#9
3.50	0.042	21						
3.63	0.044	22						
3.75	0.045	23						
4.00	0.048	24						
4.50	0.054	24	27					
5.00	0.060	22	30					
5.25	0.063	21	31					
5.50	0.066	20	33				See Footnote 3	
5.63	0.068	19	34					
5.75	0.069	19	35					
6.00	0.072	18	33	36				
6.25	0.075	18	32	37				
6.50	0.078	17	31	39				
6.75	0.081	16	30	40				
7.00	0.084	16	29	42				
7.25	0.087	15	28	43	44			
7.50	0.090	15	27	41	45			
7.63	0.092	14	26	40	46			
7.75	0.093	14	26	40	47			
8.00	0.096	14	25	39	48			
8.50	0.102	13	24	36	48			
8.75	0.105	13	23	35	48			
9.00	0.108	12	22	34	48	48		
9.50	0.114	12	21	33	46	48		
9.63	0.116	11	21	32	46	48		
10.00	0.120	11	20	31	44	48		
10.50	0.126	10	19	30	42	48		
11.00	0.132	10	18	28	40	48		
11.50	0.138	10	17	27	38	48		
11.63	0.140	9	17	27	38	48		
12.00	0.144	9	17	26	37	48		
12.50	0.150	9	16	25	35	48	48	
13.00	0.156	8	15	24	34	46	48	
13.50	0.162	8	15	23	33	44	48	
14.00	0.168	8	14	22	31	43	48	
14.50	0.174	8	14	21	30	41	48	
15.00	0.180	7	13	21	29	40	48	
15.50	0.186	7	13	20	28	39	48	
15.63	0.188	7	13	20	28	38	48	
16.00	0.192	7	13	19	28	38	47	

1. Reinforcing steel spacing shown will provide required area of steel based on $\rho = 0.001$. To be conservative, all spacing values shown were truncated to the nearest lower inch.
2. Minimum area of principal reinforcement may be less than $0.0013bt$ but may not be less than $0.0007bt$ per MSJC Code Section 1.14.6.3 The sum of the horizontal and vertical reinforcement must be at least $0.002bt$ which is 0.002 times the gross sectional area.
3. Values shown to the right of the heavy zigzag line are limited to a maximum spacing of 4 ft (48 in.) o. c. per MSJC Code Section 1.14.6.3. Values to the right of the zigzag line which are less than 48 in. are limited to 6 times the thickness per MSJC Code Section 2.3.3.3.1.

Table GN-22a Basic Development Length (inches) for Tension and Compression Bars[1,2]

BAR	F_y (ksi)	f'_m (psi)										
		1000	1500	2000	2500	3000	3500	4000	4500	5000	5500	6000
#3	40	12.33	12	12	12	12	12	12	12	12	12	12
	50	15.42	12.59	12	12	12	12	12	12	12	12	12
	60	18.50	15.10	13.08	12	12	12	12	12	12	12	12
#4	40	16.44	13.43	12	12	12	12	12	12	12	12	12
	50	20.56	16.78	14.53	13.00	12	12	12	12	12	12	12
	60	24.67	20.14	17.44	15.60	14.24	13.18	12.33	12	12	12	12
#5	40	20.56	16.78	14.53	13.00	12	12	12	12	12	12	12
	50	25.69	20.98	18.17	16.25	14.83	13.73	12.85	12.11	12	12	12
	60	30.83	25.17	21.80	19.50	17.80	16.48	15.42	14.53	13.79	13.15	12.59
#6	40	32.06	26.18	22.67	20.28	18.51	17.14	16.03	15.12	14.34	13.67	13.09
	50	40.08	32.73	28.34	25.35	23.14	21.42	20.04	18.89	17.93	17.09	16.36
	60	48.10	39.27	34.01	30.42	27.77	25.71	24.05	22.67	21.51	20.51	19.64
#7	40	37.41	30.54	26.45	23.66	21.60	20.00	18.70	17.64	16.73	15.95	15.27
	50	46.76	38.18	33.07	29.58	27.00	25.00	23.38	22.04	20.91	19.94	19.09
	60	56.12	45.82	39.68	35.49	32.40	29.99	28.06	26.45	25.10	23.93	22.91
#8	40	49.33	40.28	34.88	31.20	28.48	26.37	24.67	23.26	22.06	21.04	20.14
	50	61.66	50.35	43.60	39.00	35.60	32.96	30.83	29.07	27.58	26.30	25.17
	60	74.00	60.42	52.32	46.80	42.72	39.55	37.00	34.88	33.09	31.55	30.21
#9	40	55.65	45.43	39.35	35.19	32.13	29.74	27.82	26.23	24.89	23.73	22.72
	50	69.56	56.79	49.18	43.99	40.16	37.18	34.78	32.79	31.11	29.66	28.40
	60	83.47	68.15	59.02	52.79	48.19	44.62	41.73	39.35	37.33	35.59	34.08
#10[3]	40	62.65	51.15	44.30	39.62	36.17	33.49	31.33	29.53	28.02	26.71	25.58
	50	78.31	63.94	55.38	49.53	45.21	41.86	39.16	36.92	35.02	33.39	31.97
	60	93.98	76.73	66.45	59.44	54.26	50.23	46.99	44.30	42.03	40.07	38.37
#11[3]	40	69.56	56.79	49.18	43.99	40.16	37.18	34.78	32.79	31.11	29.66	28.40
	50	86.95	70.99	61.48	54.99	50.20	46.48	43.47	40.99	38.88	37.07	35.50
	60	104.34	85.19	73.78	65.99	60.24	55.77	52.17	49.18	46.66	44.49	42.60

1. Based on MSJC Code Eq. 2-9 and 3-15. Bar cover and bar clear spacing must be less than or equal to $5d_b$.
2. When using epoxy coated bars, increase development length by 50%.
3. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-22b Basic Development Length (inches) for Standard Hooks in Tension

Bar Size	Bar Diameter (in.)	Allowable Stress Design[1]	Strength Design[2]
#3	0.375	4.22	4.88
#4	0.500	5.63	6.50
#5	0.625	7.03	8.13
#6	0.750	8.44	9.75
#7	0.875	9.84	11.38
#8	1.000	11.25	13.00
#9	1.128	12.69	14.66
#10[3]	1.270	14.29	–
#11[3]	1.410	15.86	–

1. MSJC Code Section 2.1.10.5.1.
2. MSJC Code Section 3.3.3.2 Eq. 3-14.
3. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23a Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
2.50	#3	0.11	0.0055	0.0028	0.0018	0.0014	0.0011	0.0009	2.50
	#4	0.20	0.0100	0.0050	0.0033	0.0025	0.0020	0.0017	
	#5	0.31	0.0155	0.0078	0.0052	0.0039	0.0031	0.0026	
	#6	0.44	0.0220	0.0110	0.0073	0.0055	0.0044	0.0037	
	#7	0.60	0.0300	0.0150	0.0100	0.0075	0.0060	0.0050	
	#8	0.79	0.0395	0.0198	0.0132	0.0099	0.0079	0.0066	
	#9	1.00	0.0500	0.0250	0.0167	0.0125	0.0100	0.0083	
	#10	1.27	0.0635	0.0318	0.0212	0.0159	0.0127	0.0106	
	#11	1.56	0.0780	0.0390	0.0260	0.0195	0.0156	0.0130	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
2.75	#3	0.11	0.0050	0.0025	0.0017	0.0013	0.0010	0.0008	2.75
	#4	0.20	0.0091	0.0045	0.0030	0.0023	0.0018	0.0015	
	#5	0.31	0.0141	0.0070	0.0047	0.0035	0.0028	0.0023	
	#6	0.44	0.0200	0.0100	0.0067	0.0050	0.0040	0.0033	
	#7	0.60	0.0273	0.0136	0.0091	0.0068	0.0055	0.0045	
	#8	0.79	0.0359	0.0180	0.0120	0.0090	0.0072	0.0060	
	#9	1.00	0.0455	0.0227	0.0152	0.0114	0.0091	0.0076	
	#10	1.27	0.0577	0.0289	0.0192	0.0144	0.0115	0.0096	
	#11	1.56	0.0709	0.0355	0.0236	0.0177	0.0142	0.0118	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
3.00	#3	0.11	0.0046	0.0023	0.0015	0.0011	0.0009	0.0008	3.00
	#4	0.20	0.0083	0.0042	0.0028	0.0021	0.0017	0.0014	
	#5	0.31	0.0129	0.0065	0.0043	0.0032	0.0026	0.0022	
	#6	0.44	0.0183	0.0092	0.0061	0.0046	0.0037	0.0031	
	#7	0.60	0.0250	0.0125	0.0083	0.0063	0.0050	0.0042	
	#8	0.79	0.0329	0.0165	0.0110	0.0082	0.0066	0.0055	
	#9	1.00	0.0417	0.0208	0.0139	0.0104	0.0083	0.0069	
	#10	1.27	0.0529	0.0265	0.0176	0.0132	0.0106	0.0088	
	#11	1.56	0.0650	0.0325	0.0217	0.0163	0.0130	0.0108	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
3.25	#3	0.11	0.0042	0.0021	0.0014	0.0011	0.0008	0.0007	3.25
	#4	0.20	0.0077	0.0038	0.0026	0.0019	0.0015	0.0013	
	#5	0.31	0.0119	0.0060	0.0040	0.0030	0.0024	0.0020	
	#6	0.44	0.0169	0.0085	0.0056	0.0042	0.0034	0.0028	
	#7	0.60	0.0231	0.0115	0.0077	0.0058	0.0046	0.0038	
	#8	0.79	0.0304	0.0152	0.0101	0.0076	0.0061	0.0051	
	#9	1.00	0.0385	0.0192	0.0128	0.0096	0.0077	0.0064	
	#10	1.27	0.0488	0.0244	0.0163	0.0122	0.0098	0.0081	
	#11	1.56	0.0600	0.0300	0.0200	0.0150	0.0120	0.0100	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Section 3.3.3.1).

Table GN-23b Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; b and d in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						d (inches)
			8	16	24	32	40	48	
3.50	#3	0.11	0.0039	0.0020	0.0013	0.0010	0.0008	0.0007	3.50
	#4	0.20	0.0071	0.0036	0.0024	0.0018	0.0014	0.0012	
	#5	0.31	0.0111	0.0055	0.0037	0.0028	0.0022	0.0018	
	#6	0.44	0.0157	0.0079	0.0052	0.0039	0.0031	0.0026	
	#7	0.60	0.0214	0.0107	0.0071	0.0054	0.0043	0.0036	
	#8	0.79	0.0282	0.0141	0.0094	0.0071	0.0056	0.0047	
	#9	1.00	0.0357	0.0179	0.0119	0.0089	0.0071	0.0060	
	#10	1.27	0.0454	0.0227	0.0151	0.0113	0.0091	0.0076	
	#11	1.56	0.0557	0.0279	0.0186	0.0139	0.0111	0.0093	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						d (inches)
			8	16	24	32	40	48	
3.75	#3	0.11	0.0037	0.0018	0.0012	0.0009	0.0007	0.0006	3.75
	#4	0.20	0.0067	0.0033	0.0022	0.0017	0.0013	0.0011	
	#5	0.31	0.0103	0.0052	0.0034	0.0026	0.0021	0.0017	
	#6	0.44	0.0147	0.0073	0.0049	0.0037	0.0029	0.0024	
	#7	0.60	0.0200	0.0100	0.0067	0.0050	0.0040	0.0033	
	#8	0.79	0.0263	0.0132	0.0088	0.0066	0.0053	0.0044	
	#9	1.00	0.0333	0.0167	0.0111	0.0083	0.0067	0.0056	
	#10	1.27	0.0423	0.0212	0.0141	0.0106	0.0085	0.0071	
	#11	1.56	0.0520	0.0260	0.0173	0.0130	0.0104	0.0087	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						d (inches)
			8	16	24	32	40	48	
4.00	#3	0.11	0.0034	0.0017	0.0011	0.0009	0.0007	0.0006	4.00
	#4	0.20	0.0063	0.0031	0.0021	0.0016	0.0013	0.0010	
	#5	0.31	0.0097	0.0048	0.0032	0.0024	0.0019	0.0016	
	#6	0.44	0.0138	0.0069	0.0046	0.0034	0.0028	0.0023	
	#7	0.60	0.0188	0.0094	0.0063	0.0047	0.0038	0.0031	
	#8	0.79	0.0247	0.0123	0.0082	0.0062	0.0049	0.0041	
	#9	1.00	0.0313	0.0156	0.0104	0.0078	0.0063	0.0052	
	#10	1.27	0.0397	0.0198	0.0132	0.0099	0.0079	0.0066	
	#11	1.56	0.0488	0.0244	0.0163	0.0122	0.0098	0.0081	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						d (inches)
			8	16	24	32	40	48	
4.25	#3	0.11	0.0032	0.0016	0.0011	0.0008	0.0006	0.0005	4.25
	#4	0.20	0.0059	0.0029	0.0020	0.0015	0.0012	0.0010	
	#5	0.31	0.0091	0.0046	0.0030	0.0023	0.0018	0.0015	
	#6	0.44	0.0129	0.0065	0.0043	0.0032	0.0026	0.0022	
	#7	0.60	0.0176	0.0088	0.0059	0.0044	0.0035	0.0029	
	#8	0.79	0.0232	0.0116	0.0077	0.0058	0.0046	0.0039	
	#9	1.00	0.0294	0.0147	0.0098	0.0074	0.0059	0.0049	
	#10	1.27	0.0374	0.0187	0.0125	0.0093	0.0075	0.0062	
	#11	1.56	0.0459	0.0229	0.0153	0.0115	0.0092	0.0076	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23c Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
4.50	#3	0.11	0.0031	0.0015	0.0010	0.0008	0.0006	0.0005	4.50
	#4	0.20	0.0056	0.0028	0.0019	0.0014	0.0011	0.0009	
	#5	0.31	0.0086	0.0043	0.0029	0.0022	0.0017	0.0014	
	#6	0.44	0.0122	0.0061	0.0041	0.0031	0.0024	0.0020	
	#7	0.60	0.0167	0.0083	0.0056	0.0042	0.0033	0.0028	
	#8	0.79	0.0219	0.0110	0.0073	0.0055	0.0044	0.0037	
	#9	1.00	0.0278	0.0139	0.0093	0.0069	0.0056	0.0046	
	#10	1.27	0.0353	0.0176	0.0118	0.0088	0.0071	0.0059	
	#11	1.56	0.0433	0.0217	0.0144	0.0108	0.0087	0.0072	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
4.75	#3	0.11	0.0029	0.0014	0.0010	0.0007	0.0006	0.0005	4.75
	#4	0.20	0.0053	0.0026	0.0018	0.0013	0.0011	0.0009	
	#5	0.31	0.0082	0.0041	0.0027	0.0020	0.0016	0.0014	
	#6	0.44	0.0116	0.0058	0.0039	0.0029	0.0023	0.0019	
	#7	0.60	0.0158	0.0079	0.0053	0.0039	0.0032	0.0026	
	#8	0.79	0.0208	0.0104	0.0069	0.0052	0.0042	0.0035	
	#9	1.00	0.0263	0.0132	0.0088	0.0066	0.0053	0.0044	
	#10	1.27	0.0334	0.0167	0.0111	0.0084	0.0067	0.0056	
	#11	1.56	0.0411	0.0205	0.0137	0.0103	0.0082	0.0068	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
5.00	#3	0.11	0.0028	0.0014	0.0009	0.0007	0.0006	0.0005	5.00
	#4	0.20	0.0050	0.0025	0.0017	0.0013	0.0010	0.0008	
	#5	0.31	0.0078	0.0039	0.0026	0.0019	0.0016	0.0013	
	#6	0.44	0.0110	0.0055	0.0037	0.0028	0.0022	0.0018	
	#7	0.60	0.0150	0.0075	0.0050	0.0038	0.0030	0.0025	
	#8	0.79	0.0198	0.0099	0.0066	0.0049	0.0040	0.0033	
	#9	1.00	0.0250	0.0125	0.0083	0.0063	0.0050	0.0042	
	#10	1.27	0.0318	0.0159	0.0106	0.0079	0.0064	0.0053	
	#11	1.56	0.0390	0.0195	0.0130	0.0098	0.0078	0.0065	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
5.25	#3	0.11	0.0026	0.0013	0.0009	0.0007	0.0005	0.0004	5.25
	#4	0.20	0.0048	0.0024	0.0016	0.0012	0.0010	0.0008	
	#5	0.31	0.0074	0.0037	0.0025	0.0018	0.0015	0.0012	
	#6	0.44	0.0105	0.0052	0.0035	0.0026	0.0021	0.0017	
	#7	0.60	0.0143	0.0071	0.0048	0.0036	0.0029	0.0024	
	#8	0.79	0.0188	0.0094	0.0063	0.0047	0.0038	0.0031	
	#9	1.00	0.0238	0.0119	0.0079	0.0060	0.0048	0.0040	
	#10	1.27	0.0302	0.0151	0.0101	0.0076	0.0060	0.0050	
	#11	1.56	0.0371	0.0186	0.0124	0.0093	0.0074	0.0062	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23d Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
5.50	#3	0.11	0.0025	0.0013	0.0008	0.0006	0.0005	0.0004	5.50
	#4	0.20	0.0045	0.0023	0.0015	0.0011	0.0009	0.0008	
	#5	0.31	0.0070	0.0035	0.0023	0.0018	0.0014	0.0012	
	#6	0.44	0.0100	0.0050	0.0033	0.0025	0.0020	0.0017	
	#7	0.60	0.0136	0.0068	0.0045	0.0034	0.0027	0.0023	
	#8	0.79	0.0180	0.0090	0.0060	0.0045	0.0036	0.0030	
	#9	1.00	0.0227	0.0114	0.0076	0.0057	0.0045	0.0038	
	#10	1.27	0.0289	0.0144	0.0096	0.0072	0.0058	0.0048	
	#11	1.56	0.0355	0.0177	0.0118	0.0089	0.0071	0.0059	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
5.75	#3	0.11	0.0024	0.0012	0.0008	0.0006	0.0005	0.0004	5.75
	#4	0.20	0.0043	0.0022	0.0014	0.0011	0.0009	0.0007	
	#5	0.31	0.0067	0.0034	0.0022	0.0017	0.0013	0.0011	
	#6	0.44	0.0096	0.0048	0.0032	0.0024	0.0019	0.0016	
	#7	0.60	0.0130	0.0065	0.0043	0.0033	0.0026	0.0022	
	#8	0.79	0.0172	0.0086	0.0057	0.0043	0.0034	0.0029	
	#9	1.00	0.0217	0.0109	0.0072	0.0054	0.0043	0.0036	
	#10	1.27	0.0276	0.0138	0.0092	0.0069	0.0055	0.0046	
	#11	1.56	0.0339	0.0170	0.0113	0.0085	0.0068	0.0057	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
6.00	#3	0.11	0.0023	0.0011	0.0008	0.0006	0.0005	0.0004	6.00
	#4	0.20	0.0042	0.0021	0.0014	0.0010	0.0008	0.0007	
	#5	0.31	0.0065	0.0032	0.0022	0.0016	0.0013	0.0011	
	#6	0.44	0.0092	0.0046	0.0031	0.0023	0.0018	0.0015	
	#7	0.60	0.0125	0.0063	0.0042	0.0031	0.0025	0.0021	
	#8	0.79	0.0165	0.0082	0.0055	0.0041	0.0033	0.0027	
	#9	1.00	0.0208	0.0104	0.0069	0.0052	0.0042	0.0035	
	#10	1.27	0.0265	0.0132	0.0088	0.0066	0.0053	0.0044	
	#11	1.56	0.0325	0.0163	0.0108	0.0081	0.0065	0.0054	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
6.25	#3	0.11	0.0022	0.0011	0.0007	0.0006	0.0004	0.0004	6.25
	#4	0.20	0.0040	0.0020	0.0013	0.0010	0.0008	0.0007	
	#5	0.31	0.0062	0.0031	0.0021	0.0016	0.0012	0.0010	
	#6	0.44	0.0088	0.0044	0.0029	0.0022	0.0018	0.0015	
	#7	0.60	0.0120	0.0060	0.0040	0.0030	0.0024	0.0020	
	#8	0.79	0.0158	0.0079	0.0053	0.0040	0.0032	0.0026	
	#9	1.00	0.0200	0.0100	0.0067	0.0050	0.0040	0.0033	
	#10	1.27	0.0254	0.0127	0.0085	0.0064	0.0051	0.0042	
	#11	1.56	0.0312	0.0156	0.0104	0.0078	0.0062	0.0052	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23e Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
6.50	#3	0.11	0.0021	0.0011	0.0007	0.0005	0.0004	0.0004	6.50
	#4	0.20	0.0038	0.0019	0.0013	0.0010	0.0008	0.0006	
	#5	0.31	0.0060	0.0030	0.0020	0.0015	0.0012	0.0010	
	#6	0.44	0.0085	0.0042	0.0028	0.0021	0.0017	0.0014	
	#7	0.60	0.0115	0.0058	0.0038	0.0029	0.0023	0.0019	
	#8	0.79	0.0152	0.0076	0.0051	0.0038	0.0030	0.0025	
	#9	1.00	0.0192	0.0096	0.0064	0.0048	0.0038	0.0032	
	#10	1.27	0.0244	0.0122	0.0081	0.0061	0.0049	0.0041	
	#11	1.56	0.0300	0.0150	0.0100	0.0075	0.0060	0.0050	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
6.75	#3	0.11	0.0020	0.0010	0.0007	0.0005	0.0004	0.0003	6.75
	#4	0.20	0.0037	0.0019	0.0012	0.0009	0.0007	0.0006	
	#5	0.31	0.0057	0.0029	0.0019	0.0014	0.0011	0.0010	
	#6	0.44	0.0081	0.0041	0.0027	0.0020	0.0016	0.0014	
	#7	0.60	0.0111	0.0056	0.0037	0.0028	0.0022	0.0019	
	#8	0.79	0.0146	0.0073	0.0049	0.0037	0.0029	0.0024	
	#9	1.00	0.0185	0.0093	0.0062	0.0046	0.0037	0.0031	
	#10	1.27	0.0235	0.0118	0.0078	0.0059	0.0047	0.0039	
	#11	1.56	0.0289	0.0144	0.0096	0.0072	0.0058	0.0048	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
7.00	#3	0.11	0.0020	0.0010	0.0007	0.0005	0.0004	0.0003	7.00
	#4	0.20	0.0036	0.0018	0.0012	0.0009	0.0007	0.0006	
	#5	0.31	0.0055	0.0028	0.0018	0.0014	0.0011	0.0009	
	#6	0.44	0.0079	0.0039	0.0026	0.0020	0.0016	0.0013	
	#7	0.60	0.0107	0.0054	0.0036	0.0027	0.0021	0.0018	
	#8	0.79	0.0141	0.0071	0.0047	0.0035	0.0028	0.0024	
	#9	1.00	0.0179	0.0089	0.0060	0.0045	0.0036	0.0030	
	#10	1.27	0.0227	0.0113	0.0076	0.0057	0.0045	0.0038	
	#11	1.56	0.0279	0.0139	0.0093	0.0070	0.0056	0.0046	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
7.25	#3	0.11	0.0019	0.0009	0.0006	0.0005	0.0004	0.0003	7.25
	#4	0.20	0.0034	0.0017	0.0011	0.0009	0.0007	0.0006	
	#5	0.31	0.0053	0.0027	0.0018	0.0013	0.0011	0.0009	
	#6	0.44	0.0076	0.0038	0.0025	0.0019	0.0015	0.0013	
	#7	0.60	0.0103	0.0052	0.0034	0.0026	0.0021	0.0017	
	#8	0.79	0.0136	0.0068	0.0045	0.0034	0.0027	0.0023	
	#9	1.00	0.0172	0.0086	0.0057	0.0043	0.0034	0.0029	
	#10	1.27	0.0219	0.0109	0.0073	0.0055	0.0044	0.0036	
	#11	1.56	0.0269	0.0134	0.0090	0.0067	0.0054	0.0045	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23f Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
7.50	#3	0.11	0.0018	0.0009	0.0006	0.0005	0.0004	0.0003	7.50
	#4	0.20	0.0033	0.0017	0.0011	0.0008	0.0007	0.0006	
	#5	0.31	0.0052	0.0026	0.0017	0.0013	0.0010	0.0009	
	#6	0.44	0.0073	0.0037	0.0024	0.0018	0.0015	0.0012	
	#7	0.60	0.0100	0.0050	0.0033	0.0025	0.0020	0.0017	
	#8	0.79	0.0132	0.0066	0.0044	0.0033	0.0026	0.0022	
	#9	1.00	0.0167	0.0083	0.0056	0.0042	0.0033	0.0028	
	#10	1.27	0.0212	0.0106	0.0071	0.0053	0.0042	0.0035	
	#11	1.56	0.0260	0.0130	0.0087	0.0065	0.0052	0.0043	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
7.75	#3	0.11	0.0018	0.0009	0.0006	0.0004	0.0004	0.0003	7.75
	#4	0.20	0.0032	0.0016	0.0011	0.0008	0.0006	0.0005	
	#5	0.31	0.0050	0.0025	0.0017	0.0013	0.0010	0.0008	
	#6	0.44	0.0071	0.0035	0.0024	0.0018	0.0014	0.0012	
	#7	0.60	0.0097	0.0048	0.0032	0.0024	0.0019	0.0016	
	#8	0.79	0.0127	0.0064	0.0042	0.0032	0.0025	0.0021	
	#9	1.00	0.0161	0.0081	0.0054	0.0040	0.0032	0.0027	
	#10	1.27	0.0205	0.0102	0.0068	0.0051	0.0041	0.0034	
	#11	1.56	0.0252	0.0126	0.0084	0.0063	0.0050	0.0042	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
8.00	#3	0.11	0.0017	0.0009	0.0006	0.0004	0.0003	0.0003	8.00
	#4	0.20	0.0031	0.0016	0.0010	0.0008	0.0006	0.0005	
	#5	0.31	0.0048	0.0024	0.0016	0.0012	0.0010	0.0008	
	#6	0.44	0.0069	0.0034	0.0023	0.0017	0.0014	0.0011	
	#7	0.60	0.0094	0.0047	0.0031	0.0023	0.0019	0.0016	
	#8	0.79	0.0123	0.0062	0.0041	0.0031	0.0025	0.0021	
	#9	1.00	0.0156	0.0078	0.0052	0.0039	0.0031	0.0026	
	#10	1.27	0.0198	0.0099	0.0066	0.0050	0.0040	0.0033	
	#11	1.56	0.0244	0.0122	0.0081	0.0061	0.0049	0.0041	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
8.25	#3	0.11	0.0017	0.0008	0.0006	0.0004	0.0003	0.0003	8.25
	#4	0.20	0.0030	0.0015	0.0010	0.0008	0.0006	0.0005	
	#5	0.31	0.0047	0.0023	0.0016	0.0012	0.0009	0.0008	
	#6	0.44	0.0067	0.0033	0.0022	0.0017	0.0013	0.0011	
	#7	0.60	0.0091	0.0045	0.0030	0.0023	0.0018	0.0015	
	#8	0.79	0.0120	0.0060	0.0040	0.0030	0.0024	0.0020	
	#9	1.00	0.0152	0.0076	0.0051	0.0038	0.0030	0.0025	
	#10	1.27	0.0192	0.0096	0.0064	0.0048	0.0038	0.0032	
	#11	1.56	0.0236	0.0118	0.0079	0.0059	0.0047	0.0039	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23g Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
8.50	#3	0.11	0.0016	0.0008	0.0005	0.0004	0.0003	0.0003	8.50
	#4	0.20	0.0029	0.0015	0.0010	0.0007	0.0006	0.0005	
	#5	0.31	0.0046	0.0023	0.0015	0.0011	0.0009	0.0008	
	#6	0.44	0.0065	0.0032	0.0022	0.0016	0.0013	0.0011	
	#7	0.60	0.0088	0.0044	0.0029	0.0022	0.0018	0.0015	
	#8	0.79	0.0116	0.0058	0.0039	0.0029	0.0023	0.0019	
	#9	1.00	0.0147	0.0074	0.0049	0.0037	0.0029	0.0025	
	#10	1.27	0.0187	0.0093	0.0062	0.0047	0.0037	0.0031	
	#11	1.56	0.0229	0.0115	0.0076	0.0057	0.0046	0.0038	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
8.75	#3	0.11	0.0016	0.0008	0.0005	0.0004	0.0003	0.0003	8.75
	#4	0.20	0.0029	0.0014	0.0010	0.0007	0.0006	0.0005	
	#5	0.31	0.0044	0.0022	0.0015	0.0011	0.0009	0.0007	
	#6	0.44	0.0063	0.0031	0.0021	0.0016	0.0013	0.0010	
	#7	0.60	0.0086	0.0043	0.0029	0.0021	0.0017	0.0014	
	#8	0.79	0.0113	0.0056	0.0038	0.0028	0.0023	0.0019	
	#9	1.00	0.0143	0.0071	0.0048	0.0036	0.0029	0.0024	
	#10	1.27	0.0181	0.0091	0.0060	0.0045	0.0036	0.0030	
	#11	1.56	0.0223	0.0111	0.0074	0.0056	0.0045	0.0037	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
9.00	#3	0.11	0.0015	0.0008	0.0005	0.0004	0.0003	0.0003	9.00
	#4	0.20	0.0028	0.0014	0.0009	0.0007	0.0006	0.0005	
	#5	0.31	0.0043	0.0022	0.0014	0.0011	0.0009	0.0007	
	#6	0.44	0.0061	0.0031	0.0020	0.0015	0.0012	0.0010	
	#7	0.60	0.0083	0.0042	0.0028	0.0021	0.0017	0.0014	
	#8	0.79	0.0110	0.0055	0.0037	0.0027	0.0022	0.0018	
	#9	1.00	0.0139	0.0069	0.0046	0.0035	0.0028	0.0023	
	#10	1.27	0.0176	0.0088	0.0059	0.0044	0.0035	0.0029	
	#11	1.56	0.0217	0.0108	0.0072	0.0054	0.0043	0.0036	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
9.25	#3	0.11	0.0015	0.0007	0.0005	0.0004	0.0003	0.0002	9.25
	#4	0.20	0.0027	0.0014	0.0009	0.0007	0.0005	0.0005	
	#5	0.31	0.0042	0.0021	0.0014	0.0010	0.0008	0.0007	
	#6	0.44	0.0059	0.0030	0.0020	0.0015	0.0012	0.0010	
	#7	0.60	0.0081	0.0041	0.0027	0.0020	0.0016	0.0014	
	#8	0.79	0.0107	0.0053	0.0036	0.0027	0.0021	0.0018	
	#9	1.00	0.0135	0.0068	0.0045	0.0034	0.0027	0.0023	
	#10	1.27	0.0172	0.0086	0.0057	0.0043	0.0034	0.0029	
	#11	1.56	0.0211	0.0105	0.0070	0.0053	0.0042	0.0035	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23h Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
9.50	#3	0.11	0.0014	0.0007	0.0005	0.0004	0.0003	0.0002	9.50
	#4	0.20	0.0026	0.0013	0.0009	0.0007	0.0005	0.0004	
	#5	0.31	0.0041	0.0020	0.0014	0.0010	0.0008	0.0007	
	#6	0.44	0.0058	0.0029	0.0019	0.0014	0.0012	0.0010	
	#7	0.60	0.0079	0.0039	0.0026	0.0020	0.0016	0.0013	
	#8	0.79	0.0104	0.0052	0.0035	0.0026	0.0021	0.0017	
	#9	1.00	0.0132	0.0066	0.0044	0.0033	0.0026	0.0022	
	#10	1.27	0.0167	0.0084	0.0056	0.0042	0.0033	0.0028	
	#11	1.56	0.0205	0.0103	0.0068	0.0051	0.0041	0.0034	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
9.75	#3	0.11	0.0014	0.0007	0.0005	0.0004	0.0003	0.0002	9.75
	#4	0.20	0.0026	0.0013	0.0009	0.0006	0.0005	0.0004	
	#5	0.31	0.0040	0.0020	0.0013	0.0010	0.0008	0.0007	
	#6	0.44	0.0056	0.0028	0.0019	0.0014	0.0011	0.0009	
	#7	0.60	0.0077	0.0038	0.0026	0.0019	0.0015	0.0013	
	#8	0.79	0.0101	0.0051	0.0034	0.0025	0.0020	0.0017	
	#9	1.00	0.0128	0.0064	0.0043	0.0032	0.0026	0.0021	
	#10	1.27	0.0163	0.0081	0.0054	0.0041	0.0033	0.0027	
	#11	1.56	0.0200	0.0100	0.0067	0.0050	0.0040	0.0033	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
10.00	#3	0.11	0.0014	0.0007	0.0005	0.0003	0.0003	0.0002	10.00
	#4	0.20	0.0025	0.0013	0.0008	0.0006	0.0005	0.0004	
	#5	0.31	0.0039	0.0019	0.0013	0.0010	0.0008	0.0006	
	#6	0.44	0.0055	0.0028	0.0018	0.0014	0.0011	0.0009	
	#7	0.60	0.0075	0.0038	0.0025	0.0019	0.0015	0.0013	
	#8	0.79	0.0099	0.0049	0.0033	0.0025	0.0020	0.0016	
	#9	1.00	0.0125	0.0063	0.0042	0.0031	0.0025	0.0021	
	#10	1.27	0.0159	0.0079	0.0053	0.0040	0.0032	0.0026	
	#11	1.56	0.0195	0.0098	0.0065	0.0049	0.0039	0.0033	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
10.25	#3	0.11	0.0013	0.0007	0.0004	0.0003	0.0003	0.0002	10.25
	#4	0.20	0.0024	0.0012	0.0008	0.0006	0.0005	0.0004	
	#5	0.31	0.0038	0.0019	0.0013	0.0009	0.0008	0.0006	
	#6	0.44	0.0054	0.0027	0.0018	0.0013	0.0011	0.0009	
	#7	0.60	0.0073	0.0037	0.0024	0.0018	0.0015	0.0012	
	#8	0.79	0.0096	0.0048	0.0032	0.0024	0.0019	0.0016	
	#9	1.00	0.0122	0.0061	0.0041	0.0030	0.0024	0.0020	
	#10	1.27	0.0155	0.0077	0.0052	0.0039	0.0031	0.0026	
	#11	1.56	0.0190	0.0095	0.0063	0.0048	0.0038	0.0032	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23i Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
10.50	#3	0.11	0.0013	0.0007	0.0004	0.0003	0.0003	0.0002	10.50
	#4	0.20	0.0024	0.0012	0.0008	0.0006	0.0005	0.0004	
	#5	0.31	0.0037	0.0018	0.0012	0.0009	0.0007	0.0006	
	#6	0.44	0.0052	0.0026	0.0017	0.0013	0.0010	0.0009	
	#7	0.60	0.0071	0.0036	0.0024	0.0018	0.0014	0.0012	
	#8	0.79	0.0094	0.0047	0.0031	0.0024	0.0019	0.0016	
	#9	1.00	0.0119	0.0060	0.0040	0.0030	0.0024	0.0020	
	#10	1.27	0.0151	0.0076	0.0050	0.0038	0.0030	0.0025	
	#11	1.56	0.0186	0.0093	0.0062	0.0046	0.0037	0.0031	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
10.75	#3	0.11	0.0013	0.0006	0.0004	0.0003	0.0003	0.0002	10.75
	#4	0.20	0.0023	0.0012	0.0008	0.0006	0.0005	0.0004	
	#5	0.31	0.0036	0.0018	0.0012	0.0009	0.0007	0.0006	
	#6	0.44	0.0051	0.0026	0.0017	0.0013	0.0010	0.0009	
	#7	0.60	0.0070	0.0035	0.0023	0.0017	0.0014	0.0012	
	#8	0.79	0.0092	0.0046	0.0031	0.0023	0.0018	0.0015	
	#9	1.00	0.0116	0.0058	0.0039	0.0029	0.0023	0.0019	
	#10	1.27	0.0148	0.0074	0.0049	0.0037	0.0030	0.0025	
	#11	1.56	0.0181	0.0091	0.0060	0.0045	0.0036	0.0030	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
11.00	#3	0.11	0.0013	0.0006	0.0004	0.0003	0.0003	0.0002	11.00
	#4	0.20	0.0023	0.0011	0.0008	0.0006	0.0005	0.0004	
	#5	0.31	0.0035	0.0018	0.0012	0.0009	0.0007	0.0006	
	#6	0.44	0.0050	0.0025	0.0017	0.0013	0.0010	0.0008	
	#7	0.60	0.0068	0.0034	0.0023	0.0017	0.0014	0.0011	
	#8	0.79	0.0090	0.0045	0.0030	0.0022	0.0018	0.0015	
	#9	1.00	0.0114	0.0057	0.0038	0.0028	0.0023	0.0019	
	#10	1.27	0.0144	0.0072	0.0048	0.0036	0.0029	0.0024	
	#11	1.56	0.0177	0.0089	0.0059	0.0044	0.0035	0.0030	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
11.25	#3	0.11	0.0012	0.0006	0.0004	0.0003	0.0002	0.0002	11.25
	#4	0.20	0.0022	0.0011	0.0007	0.0006	0.0004	0.0004	
	#5	0.31	0.0034	0.0017	0.0011	0.0009	0.0007	0.0006	
	#6	0.44	0.0049	0.0024	0.0016	0.0012	0.0010	0.0008	
	#7	0.60	0.0067	0.0033	0.0022	0.0017	0.0013	0.0011	
	#8	0.79	0.0088	0.0044	0.0029	0.0022	0.0018	0.0015	
	#9	1.00	0.0111	0.0056	0.0037	0.0028	0.0022	0.0019	
	#10	1.27	0.0141	0.0071	0.0047	0.0035	0.0028	0.0024	
	#11	1.56	0.0173	0.0087	0.0058	0.0043	0.0035	0.0029	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23j Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
11.50	#3	0.11	0.0012	0.0006	0.0004	0.0003	0.0002	0.0002	11.50
	#4	0.20	0.0022	0.0011	0.0007	0.0005	0.0004	0.0004	
	#5	0.31	0.0034	0.0017	0.0011	0.0008	0.0007	0.0006	
	#6	0.44	0.0048	0.0024	0.0016	0.0012	0.0010	0.0008	
	#7	0.60	0.0065	0.0033	0.0022	0.0016	0.0013	0.0011	
	#8	0.79	0.0086	0.0043	0.0029	0.0021	0.0017	0.0014	
	#9	1.00	0.0109	0.0054	0.0036	0.0027	0.0022	0.0018	
	#10	1.27	0.0138	0.0069	0.0046	0.0035	0.0028	0.0023	
	#11	1.56	0.0170	0.0085	0.0057	0.0042	0.0034	0.0028	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
11.75	#3	0.11	0.0012	0.0006	0.0004	0.0003	0.0002	0.0002	11.75
	#4	0.20	0.0021	0.0011	0.0007	0.0005	0.0004	0.0004	
	#5	0.31	0.0033	0.0016	0.0011	0.0008	0.0007	0.0005	
	#6	0.44	0.0047	0.0023	0.0016	0.0012	0.0009	0.0008	
	#7	0.60	0.0064	0.0032	0.0021	0.0016	0.0013	0.0011	
	#8	0.79	0.0084	0.0042	0.0028	0.0021	0.0017	0.0014	
	#9	1.00	0.0106	0.0053	0.0035	0.0027	0.0021	0.0018	
	#10	1.27	0.0135	0.0068	0.0045	0.0034	0.0027	0.0023	
	#11	1.56	0.0166	0.0083	0.0055	0.0041	0.0033	0.0028	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
12.00	#3	0.11	0.0011	0.0006	0.0004	0.0003	0.0002	0.0002	12.00
	#4	0.20	0.0021	0.0010	0.0007	0.0005	0.0004	0.0003	
	#5	0.31	0.0032	0.0016	0.0011	0.0008	0.0006	0.0005	
	#6	0.44	0.0046	0.0023	0.0015	0.0011	0.0009	0.0008	
	#7	0.60	0.0063	0.0031	0.0021	0.0016	0.0013	0.0010	
	#8	0.79	0.0082	0.0041	0.0027	0.0021	0.0016	0.0014	
	#9	1.00	0.0104	0.0052	0.0035	0.0026	0.0021	0.0017	
	#10	1.27	0.0132	0.0066	0.0044	0.0033	0.0026	0.0022	
	#11	1.56	0.0163	0.0081	0.0054	0.0041	0.0033	0.0027	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
12.25	#3	0.11	0.0011	0.0006	0.0004	0.0003	0.0002	0.0002	12.25
	#4	0.20	0.0020	0.0010	0.0007	0.0005	0.0004	0.0003	
	#5	0.31	0.0032	0.0016	0.0011	0.0008	0.0006	0.0005	
	#6	0.44	0.0045	0.0022	0.0015	0.0011	0.0009	0.0007	
	#7	0.60	0.0061	0.0031	0.0020	0.0015	0.0012	0.0010	
	#8	0.79	0.0081	0.0040	0.0027	0.0020	0.0016	0.0013	
	#9	1.00	0.0102	0.0051	0.0034	0.0026	0.0020	0.0017	
	#10	1.27	0.0130	0.0065	0.0043	0.0032	0.0026	0.0022	
	#11	1.56	0.0159	0.0080	0.0053	0.0040	0.0032	0.0027	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23k Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
12.50	#3	0.11	0.0011	0.0006	0.0004	0.0003	0.0002	0.0002	12.50
	#4	0.20	0.0020	0.0010	0.0007	0.0005	0.0004	0.0003	
	#5	0.31	0.0031	0.0016	0.0010	0.0008	0.0006	0.0005	
	#6	0.44	0.0044	0.0022	0.0015	0.0011	0.0009	0.0007	
	#7	0.60	0.0060	0.0030	0.0020	0.0015	0.0012	0.0010	
	#8	0.79	0.0079	0.0040	0.0026	0.0020	0.0016	0.0013	
	#9	1.00	0.0100	0.0050	0.0033	0.0025	0.0020	0.0017	
	#10	1.27	0.0127	0.0064	0.0042	0.0032	0.0025	0.0021	
	#11	1.56	0.0156	0.0078	0.0052	0.0039	0.0031	0.0026	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
12.75	#3	0.11	0.0011	0.0005	0.0004	0.0003	0.0002	0.0002	12.75
	#4	0.20	0.0020	0.0010	0.0007	0.0005	0.0004	0.0003	
	#5	0.31	0.0030	0.0015	0.0010	0.0008	0.0006	0.0005	
	#6	0.44	0.0043	0.0022	0.0014	0.0011	0.0009	0.0007	
	#7	0.60	0.0059	0.0029	0.0020	0.0015	0.0012	0.0010	
	#8	0.79	0.0077	0.0039	0.0026	0.0019	0.0015	0.0013	
	#9	1.00	0.0098	0.0049	0.0033	0.0025	0.0020	0.0016	
	#10	1.27	0.0125	0.0062	0.0042	0.0031	0.0025	0.0021	
	#11	1.56	0.0153	0.0076	0.0051	0.0038	0.0031	0.0025	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
13.00	#3	0.11	0.0011	0.0005	0.0004	0.0003	0.0002	0.0002	13.00
	#4	0.20	0.0019	0.0010	0.0006	0.0005	0.0004	0.0003	
	#5	0.31	0.0030	0.0015	0.0010	0.0007	0.0006	0.0005	
	#6	0.44	0.0042	0.0021	0.0014	0.0011	0.0008	0.0007	
	#7	0.60	0.0058	0.0029	0.0019	0.0014	0.0012	0.0010	
	#8	0.79	0.0076	0.0038	0.0025	0.0019	0.0015	0.0013	
	#9	1.00	0.0096	0.0048	0.0032	0.0024	0.0019	0.0016	
	#10	1.27	0.0122	0.0061	0.0041	0.0031	0.0024	0.0020	
	#11	1.56	0.0150	0.0075	0.0050	0.0038	0.0030	0.0025	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
13.25	#3	0.11	0.0010	0.0005	0.0003	0.0003	0.0002	0.0002	13.25
	#4	0.20	0.0019	0.0009	0.0006	0.0005	0.0004	0.0003	
	#5	0.31	0.0029	0.0015	0.0010	0.0007	0.0006	0.0005	
	#6	0.44	0.0042	0.0021	0.0014	0.0010	0.0008	0.0007	
	#7	0.60	0.0057	0.0028	0.0019	0.0014	0.0011	0.0009	
	#8	0.79	0.0075	0.0037	0.0025	0.0019	0.0015	0.0012	
	#9	1.00	0.0094	0.0047	0.0031	0.0024	0.0019	0.0016	
	#10	1.27	0.0120	0.0060	0.0040	0.0030	0.0024	0.0020	
	#11	1.56	0.0147	0.0074	0.0049	0.0037	0.0029	0.0025	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23I Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; *b* and *d* in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
13.50	#3	0.11	0.0010	0.0005	0.0003	0.0003	0.0002	0.0002	13.50
	#4	0.20	0.0019	0.0009	0.0006	0.0005	0.0004	0.0003	
	#5	0.31	0.0029	0.0014	0.0010	0.0007	0.0006	0.0005	
	#6	0.44	0.0041	0.0020	0.0014	0.0010	0.0008	0.0007	
	#7	0.60	0.0056	0.0028	0.0019	0.0014	0.0011	0.0009	
	#8	0.79	0.0073	0.0037	0.0024	0.0018	0.0015	0.0012	
	#9	1.00	0.0093	0.0046	0.0031	0.0023	0.0019	0.0015	
	#10	1.27	0.0118	0.0059	0.0039	0.0029	0.0024	0.0020	
	#11	1.56	0.0144	0.0072	0.0048	0.0036	0.0029	0.0024	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
13.75	#3	0.11	0.0010	0.0005	0.0003	0.0003	0.0002	0.0002	13.75
	#4	0.20	0.0018	0.0009	0.0006	0.0005	0.0004	0.0003	
	#5	0.31	0.0028	0.0014	0.0009	0.0007	0.0006	0.0005	
	#6	0.44	0.0040	0.0020	0.0013	0.0010	0.0008	0.0007	
	#7	0.60	0.0055	0.0027	0.0018	0.0014	0.0011	0.0009	
	#8	0.79	0.0072	0.0036	0.0024	0.0018	0.0014	0.0012	
	#9	1.00	0.0091	0.0045	0.0030	0.0023	0.0018	0.0015	
	#10	1.27	0.0115	0.0058	0.0038	0.0029	0.0023	0.0019	
	#11	1.56	0.0142	0.0071	0.0047	0.0035	0.0028	0.0024	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
14.00	#3	0.11	0.0010	0.0005	0.0003	0.0002	0.0002	0.0002	14.00
	#4	0.20	0.0018	0.0009	0.0006	0.0004	0.0004	0.0003	
	#5	0.31	0.0028	0.0014	0.0009	0.0007	0.0006	0.0005	
	#6	0.44	0.0039	0.0020	0.0013	0.0010	0.0008	0.0007	
	#7	0.60	0.0054	0.0027	0.0018	0.0013	0.0011	0.0009	
	#8	0.79	0.0071	0.0035	0.0024	0.0018	0.0014	0.0012	
	#9	1.00	0.0089	0.0045	0.0030	0.0022	0.0018	0.0015	
	#10	1.27	0.0113	0.0057	0.0038	0.0028	0.0023	0.0019	
	#11	1.56	0.0139	0.0070	0.0046	0.0035	0.0028	0.0023	

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						*d* (inches)
			8	16	24	32	40	48	
14.25	#3	0.11	0.0010	0.0005	0.0003	0.0002	0.0002	0.0002	14.25
	#4	0.20	0.0018	0.0009	0.0006	0.0004	0.0004	0.0003	
	#5	0.31	0.0027	0.0014	0.0009	0.0007	0.0005	0.0005	
	#6	0.44	0.0039	0.0019	0.0013	0.0010	0.0008	0.0006	
	#7	0.60	0.0053	0.0026	0.0018	0.0013	0.0011	0.0009	
	#8	0.79	0.0069	0.0035	0.0023	0.0017	0.0014	0.0012	
	#9	1.00	0.0088	0.0044	0.0029	0.0022	0.0018	0.0015	
	#10	1.27	0.0111	0.0056	0.0037	0.0028	0.0022	0.0019	
	#11	1.56	0.0137	0.0068	0.0046	0.0034	0.0027	0.0023	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-23m Steel Ratio $\rho = \frac{A_s}{bd}$ A_s in Square Inches; b and d in Inches

d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						d (inches)
			8	16	24	32	40	48	
14.50	#3	0.11	0.0009	0.0005	0.0003	0.0002	0.0002	0.0002	14.50
	#4	0.20	0.0017	0.0009	0.0006	0.0004	0.0003	0.0003	
	#5	0.31	0.0027	0.0013	0.0009	0.0007	0.0005	0.0004	
	#6	0.44	0.0038	0.0019	0.0013	0.0009	0.0008	0.0006	
	#7	0.60	0.0052	0.0026	0.0017	0.0013	0.0010	0.0009	
	#8	0.79	0.0068	0.0034	0.0023	0.0017	0.0014	0.0011	
	#9	1.00	0.0086	0.0043	0.0029	0.0022	0.0017	0.0014	
	#10	1.27	0.0109	0.0055	0.0036	0.0027	0.0022	0.0018	
	#11	1.56	0.0134	0.0067	0.0045	0.0034	0.0027	0.0022	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						d (inches)
			8	16	24	32	40	48	
14.75	#3	0.11	0.0009	0.0005	0.0003	0.0002	0.0002	0.0002	14.75
	#4	0.20	0.0017	0.0008	0.0006	0.0004	0.0003	0.0003	
	#5	0.31	0.0026	0.0013	0.0009	0.0007	0.0005	0.0004	
	#6	0.44	0.0037	0.0019	0.0012	0.0009	0.0007	0.0006	
	#7	0.60	0.0051	0.0025	0.0017	0.0013	0.0010	0.0008	
	#8	0.79	0.0067	0.0033	0.0022	0.0017	0.0013	0.0011	
	#9	1.00	0.0085	0.0042	0.0028	0.0021	0.0017	0.0014	
	#10	1.27	0.0108	0.0054	0.0036	0.0027	0.0022	0.0018	
	#11	1.56	0.0132	0.0066	0.0044	0.0033	0.0026	0.0022	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						d (inches)
			8	16	24	32	40	48	
15.00	#3	0.11	0.0009	0.0005	0.0003	0.0002	0.0002	0.0002	15.00
	#4	0.20	0.0017	0.0008	0.0006	0.0004	0.0003	0.0003	
	#5	0.31	0.0026	0.0013	0.0009	0.0006	0.0005	0.0004	
	#6	0.44	0.0037	0.0018	0.0012	0.0009	0.0007	0.0006	
	#7	0.60	0.0050	0.0025	0.0017	0.0013	0.0010	0.0008	
	#8	0.79	0.0066	0.0033	0.0022	0.0016	0.0013	0.0011	
	#9	1.00	0.0083	0.0042	0.0028	0.0021	0.0017	0.0014	
	#10	1.27	0.0106	0.0053	0.0035	0.0026	0.0021	0.0018	
	#11	1.56	0.0130	0.0065	0.0043	0.0033	0.0026	0.0022	
d (inches)	Bar Size[1]	Steel Area (sq. in.)	Steel Spacing (inches)						d (inches)
			8	16	24	32	40	48	
15.25	#3	0.11	0.0009	0.0005	0.0003	0.0002	0.0002	0.0002	15.25
	#4	0.20	0.0016	0.0008	0.0005	0.0004	0.0003	0.0003	
	#5	0.31	0.0025	0.0013	0.0008	0.0006	0.0005	0.0004	
	#6	0.44	0.0036	0.0018	0.0012	0.0009	0.0007	0.0006	
	#7	0.60	0.0049	0.0025	0.0016	0.0012	0.0010	0.0008	
	#8	0.79	0.0065	0.0032	0.0022	0.0016	0.0013	0.0011	
	#9	1.00	0.0082	0.0041	0.0027	0.0020	0.0016	0.0014	
	#10	1.27	0.0104	0.0052	0.0035	0.0026	0.0021	0.0017	
	#11	1.56	0.0128	0.0064	0.0043	0.0032	0.0026	0.0021	

1. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-24a Ratio of Steel Area to Gross Cross-Sectional Area[1]

Wall Thickness (inches)	Bar Size[2]	Area (sq. in.)	Spacing of Steel Reinforcement (inches)					
			8	16	24	32	40	48
5.625 (6" Nominal)	#3	0.11	0.0024	0.0012	0.0008	0.0006	0.0005	0.0004
	#4	0.20	0.0044	0.0022	0.0015	0.0011	0.0009	0.0007
	#5	0.31	0.0069	0.0034	0.0023	0.0017	0.0014	0.0011
7.625 (8" Nominal)	#3	0.11	0.0018	0.0009	0.0006	0.0005	0.0004	0.0003
	#4	0.20	0.0033	0.0016	0.0011	0.0008	0.0007	0.0005
	#5	0.31	0.0051	0.0025	0.0017	0.0013	0.0010	0.0008
	#6	0.44	0.0072	0.0036	0.0024	0.0018	0.0014	0.0012
	#7	0.60	0.0098	0.0049	0.0033	0.0025	0.0020	0.0016
	#8	0.79	0.0130	0.0065	0.0043	0.0032	0.0026	0.0022
	#9	1.00	0.0164	0.0082	0.0055	0.0041	0.0033	0.0027
	#10	1.27	0.0208	0.0104	0.0069	0.0052	0.0042	0.0035
	#11	1.56	0.0256	0.0128	0.0085	0.0064	0.0051	0.0043
9.625 (10" Nominal)	#4	0.20	0.0026	0.0013	0.0009	0.0006	0.0005	0.0004
	#5	0.31	0.0040	0.0020	0.0013	0.0010	0.0008	0.0007
	#6	0.44	0.0057	0.0029	0.0019	0.0014	0.0011	0.0010
	#7	0.60	0.0078	0.0039	0.0026	0.0019	0.0016	0.0013
	#8	0.79	0.0103	0.0051	0.0034	0.0026	0.0021	0.0017
	#9	1.00	0.0130	0.0065	0.0043	0.0032	0.0026	0.0022
	#10	1.27	0.0165	0.0082	0.0055	0.0041	0.0033	0.0027
	#11	1.56	0.0203	0.0101	0.0068	0.0051	0.0041	0.0034
11.625 (12" Nominal)	#4	0.20	0.0022	0.0011	0.0007	0.0005	0.0004	0.0004
	#5	0.31	0.0033	0.0017	0.0011	0.0008	0.0007	0.0006
	#6	0.44	0.0047	0.0024	0.0016	0.0012	0.0009	0.0008
	#7	0.60	0.0065	0.0032	0.0022	0.0016	0.0013	0.0011
	#8	0.79	0.0085	0.0042	0.0028	0.0021	0.0017	0.0014
	#9	1.00	0.0108	0.0054	0.0036	0.0027	0.0022	0.0018
	#10	1.27	0.0137	0.0068	0.0046	0.0034	0.0027	0.0023
	#11	1.56	0.0168	0.0084	0.0056	0.0042	0.0034	0.0028
15.625 (16" Nominal)	#4	0.20	0.0016	0.0008	0.0005	0.0004	0.0003	0.0003
	#5	0.31	0.0025	0.0012	0.0008	0.0006	0.0005	0.0004
	#6	0.44	0.0035	0.0018	0.0012	0.0009	0.0007	0.0006
	#7	0.60	0.0048	0.0024	0.0016	0.0012	0.0010	0.0008
	#8	0.79	0.0063	0.0032	0.0021	0.0016	0.0013	0.0011
	#9	1.00	0.0080	0.0040	0.0027	0.0020	0.0016	0.0013
	#10	1.27	0.0102	0.0051	0.0034	0.0025	0.0020	0.0017
	#11	1.56	0.0125	0.0062	0.0042	0.0031	0.0025	0.0021

1. MSJC Code Minimum = 0.0007*bt* for Seismic Design Category D and E.
2. MSJC Code limits the maximum bar size to #11 (MSJC Code Section 1.12.2.1) and Strength Design limits the maximum bar size to #9 (MSJC Code Section 3.3.3.1).

Table GN-24b Maximum Area of Steel per CMU Cell

Nominal Thickness (inches)	Actual Thickness (inches)	Approximate Cell Area (sq. in.)	6% Working Stress Design Limit on Steel Area (sq. in.)[1]	4% Strength Design Limit on Steel Area (sq. in.)[2]
4	3 5/8	12.6	0.76	0.50
6	5 5/8	21.0	1.26	0.84
8	7 5/8	30.0	1.80	1.20
10	9 5/8	42.0	2.52	1.68
12	11 5/8	54.0	3.24	2.16

1. Based on MSJC Code Table 1.16.1 Footnote 4.
2. Based on MSJC Code Section 3.3.3.1.

Table GN-24c Maximum Number of Reinforcing Bars per Cell

Nominal Thickness (inches)		Area of Steel (sq. in.)[1]	Bar Size and Area per Bar							
			#4 0.20	#5 0.31	#6 0.44	#7 0.60	#8 0.79	#9 1.00	#10[3] 1.27	#11[3] 1.56
4% Code Limit on Steel Area[2]	4	0.50	2	1	1	–	–	–	–	–
	6	0.84	4	2	2	1	1	–	–	–
	8	1.20	6	4	2	2	1	1	–	–
	10	1.68	8	5	3	2	2	1	–	–
	12	2.16	10	7	5	3	2	2	–	–
6% Code Limit on Steel Area[4]	4	0.76	3	2	1	1	1	–	–	–
	6	1.26	6	4	2	2	1	1	1	–
	8	1.80	9	5	4	3	2	1	1	1
	10	2.52	12	8	5	4	3	2	2	1
	12	3.24	16	10	7	5	4	3	2	2

1. Values based on Table GN-24b.
2. Based on MSJC Code Section 3.3.3.1 for Strength Design.
3. Strength Design does not allow the use of bars larger than #9.
4. Based on MSJC Code Table 1.16.1 Footnote 4 for Allowable Stress Design.

Table GN-25a Conversion of Measurement Systems

English Measurement to SI (Metric) Measurement			
Unit	**Exact conversion**		**Approximate Conversion**
Length			
1 mile	1.609344	kilometers	1.6 km or 1 1/2 km
1 yard	0.9144	meter	0.9 m or 1 meter
1 foot	0.3048	meter	0.3 m or 1/3 meter
1 inch	25.40	millimeters	25 mm or 1/40 meter
Area			
1 acre	4,046.856	square meters	4000 m^2
1 square foot	0.0929	square meters	1/10 m^2 or 1000 cm^2
1 square inch	645.2	square millimeters	6 cm^2 or 650 mm^2
Volume			
1 cubic yard	0.7646	cubic meter or	3/4 m^3 or
	764.56	liters	750 liters
1 cubic foot	0.0283	cubic meter or	1/35 m^3 or
	28.217	liters	30 liters
1 cubic inch	16.387	cubic centimeters	16 cm^3 or 16,000 mm^3
1 gallon	3,785.4	cubic centimeters or	4,000 cm^3 or
	3.785	liters	4 liters
1 quart	946.35	cubic centimeters or	1,000 cm^3 or
	0.94635	liters	1 liter
Speed			
1 mile per hour	1.609344	kilometers per hour	1.6 km/h or 1 1/2 km/h
1 foot per second	0.3048	meters per second	0.3 m/s or 1/3 m/s
Weight or Mass			
1 ounce	28.35	grams	30 grams
1 pound	0.45359	kilogram or 453 grams	1/2 kg or 500 g
1 kip	453.59	kilograms	500 kg or 0.5 Mg
1 Ton (Short)*	907.18	kilograms	1 Mg
Density			
1 pound/cubic foot	16.018	kilograms/cubic meter	16 kg/m^3
1 pound/gallon	119.83	kilograms/cubic meter	120 kg/m^3
Force			
1 pound force	4.448	newtons	4 1/2 N
1 kip force	4.448	kilo newtons	4500 N or 4 1/2 kN
1 pound force/lin. ft	14.59	newtons/meter	14 1/2 N/m
1 kip force/lin. ft	14.59	kilo newtons/meter	14 1/2 kN/m
Pressure			
1 pound/sq. in	6,894.8	pascals	7000 Pa or 7 kPa
1 kip/sq. in	6.895	mega pascals	7 MPa
1 pound force/sq. ft	47.9	pascals	48 Pa
1 kip force/sq. ft	47.9	kilo pascals	48 kPa
Moment			
1 foot pound force	1.356	newton meters	1.36 Nm
1 foot kip force	1.356	kilo newton meters	1.36 kNm
1 foot pound force/foot	4.448	newton meters/meters	4.45 Nm/m
Energy			
1 BTU	1,054.35	joule or 1.054 kj	1 kj
Temperature			
°Fahrenheit	[(°F-32) (5/9)]	°Celcius	

* A Short Ton is a unit of weight equal to 2,000 pounds (0.907 metric ton or 907.18 kilograms), as compared to a Long Ton which is a unit of weight equal to 2,240 pounds (1.016 metric tons or 1,016.04 kilograms).

Table GN-25a Conversion of Measurement Systems (Continued)

English Measurement to SI (Metric) Measurement			
Unit	**Exact conversion**		**Approximate Conversion**
Length			
1 kilometer	0.6214	miles	5/8 mile or 0.6 mile
1 meter	3.2808	feet or 39 3/8 inches	3 ft. 3 in. or 3 ft. +
1 centimeter	0.3937	inch	0.4 inch or 3/8 inch
1 millimeter	0.0394	inch	1/32 inch
Area			
1 square kilometer	0.3861	square mile or 247.1 acres	1/3 mi² or 250 acres
1 square meter	1.196	square yds or 10.764 sq. ft.	1/2 yd² or 10 ft²
1 square centimeter	0.155	square inch	1/6 in²
Volume			
1 cubic meter	35.315	cubic feet or	35 ft³ or
	264.17	gallons	265 gallons
1 liter	0.0353	cubic feet or	1/4 gallon or
	0.2642	gallon or 61.024 in³	1 quart or 60 in³
1 cubic centimeter	0.061	cubic inch	1/16 in³
Speed			
1 kilometer per hour	0.6214	mile per hour	5/8 mph or 0.6 mph
1 meter per second	3.2808	feet per second	3 ft/s or 1 yd/s
Weight or Mass			
1 gram	0.03527	ounces	1/30 ounce
1 kilogram	2.205	pounds	2 1/4 pounds or 2 pounds
1 megagram	2.205 kips or 2,205	pounds	2 1/4 kips or 2,000 pounds
1 gigagram	1,102	tons or 2,205,000 lbs	1,000 Tons or 2 million lbs
Density			
1 gram/cubic centimeter	8.345	lbs/gal or 62.428 lbs/cu ft	8 1/2 lbs/gal or 62 lbs/ft³
1 kg/cubic meter	0.0081345	lbs/gal or 0.062428 lbs/cu ft	1/8 oz/gal or 1/16 lb/ft³
Force			
1 newton	0.2248	pound force	1/4 pound force
1 kilo newton	224.8	pound force	225 pound force
Pressure			
1 pascal	0.000145	pounds/square inch	
1 kilo pascal	0.145	pounds/square inch	1/7 psi
1 mega pascal	145	pounds/square inch	150 psi
Moment			
1 newton meter	0.737	foot pound force	3/4 ft lb
1 kilo newton meter	0.737	foot kip force	3/4 ft kip
1 newton meter/meter	0.225	foot pound force/foot	1/4 ft lb/ft
Energy			
1000 joules	0.94845	BTU	1 BTU
Temperature			
°Celcius	[(1.8 x °C) + 32]	°Fahrenheit	

Table GN-25b SI Prefixes for Magnitude

Amount	Exponential Factor	Prefix	Symbol
1 000 000 000 000 1 000 000 000	10^{12} 10^{9}	tera giga	T G
1 000 000 1 000	10^{6} 10^{3}	mega kilo	M k
100 10	10^{2} 10^{1}	hecta deka	h da
0.1 0.01 0.001	10^{-1} 10^{-2} 10^{-3}	deci centi milli	d c m
0.000 001 0.000 000 001	10^{-6} 10^{-9}	micro nano	μ η

Table GN-26a Length Equivalents[1] - Inches to Millimeters

Inches	0	1	2	3	4	5	6	7	8	9
	Millimeters									
0	0.0	25.4	50.8	76.2	101.6	127.0	152.4	177.8	203.2	228.6
10	254.0	279.4	304.8	330.2	355.6	381.0	406.4	431.8	457.2	482.6
20	508.0	533.4	558.8	584.2	609.6	635.0	660.4	685.8	711.2	736.6
30	762.0	787.4	812.8	838.2	863.6	889.0	914.4	939.8	965.2	990.6
40	1016.0	1041.4	1066.8	1092.2	1117.6	1143.0	1168.4	1139.8	1219.2	1244.6
50	1270.0	1295.4	1320.8	1346.2	1371.6	1397.0	1422.4	1447.8	1473.2	1498.6
60	1524.0	1549.4	1574.8	1600.2	1625.6	1651.0	1676.4	1701.8	1727.2	1752.6
70	1778.0	1803.4	1828.8	1854.2	1879.6	1905.0	1930.4	1955.8	1981.2	2006.6
80	2032.0	2057.4	2082.8	2108.2	2133.6	2159.0	2184.4	2209.8	2235.2	2260.6
90	2286.0	2311.4	2336.8	2362.2	2387.6	2413.0	2438.4	2463.8	2489.2	2514.6
100	2540.0	2565.4	2590.8	2616.2	2641.6	2667.0	2692.4	2717.8	2743.2	2768.6

1. All values in this table are based on the relation, 1 in. = 25.4 mm. By manipulation of the decimal point any decimal value or multiple of an inch may be converted to its equivalent in millimeters.

Table GN-26b Length Equivalents[1] - Feet to Meters

Feet	0	1	2	3	4	5	6	7	8	9
	Meters									
0	0.00	0.30	0.61	0.91	1.22	1.52	1.83	2.13	2.44	2.74
10	3.05	3.35	3.66	3.96	4.27	4.57	4.88	5.18	5.49	5.79
20	6.10	6.40	6.71	7.01	7.32	7.62	7.92	8.23	8.53	8.84
30	9.14	9.45	9.75	10.06	10.36	10.67	10.97	11.28	11.58	11.89
40	12.19	12.50	12.80	13.11	13.41	13.72	14.02	14.33	14.63	14.94
50	15.24	15.54	15.85	16.15	16.46	16.76	17.07	17.37	17.68	17.98
60	18.29	18.59	18.90	19.20	19.51	19.81	20.12	20.42	20.73	21.03
70	21.34	21.64	21.95	22.25	22.56	22.86	23.16	23.47	23.77	24.08
80	24.38	24.69	24.99	25.30	25.60	25.91	26.21	26.52	26.82	27.13
90	27.43	27.74	28.04	28.35	28.65	28.96	29.26	29.57	29.87	30.18
100	30.48	30.78	31.09	31.70	31.70	32.00	32.31	32.61	32.92	33.22

1. All values in this table are based on the relation, 1 ft = 0.3048 m. By manipulation of the decimal point any decimal value or multiple of a foot may be converted to its equivalent in meters.

Table GN-27 Force Equivalents[1] - Pounds Force to Newtons

Pounds Force	0	1	2	3	4	5	6	7	8	9
	Newtons									
0	0.0	4.5	8.9	13.3	17.8	22.2	26.7	31.1	35.6	40.0
10	44.5	48.9	53.4	57.8	62.3	66.7	71.2	75.6	80.1	84.5
20	89.0	93.4	97.9	102.3	106.8	111.2	115.7	120.1	124.5	129.0
30	133.4	137.9	142.3	146.8	151.2	155.7	160.1	164.6	169.0	173.5
40	177.9	182.4	186.8	191.3	195.7	200.2	204.6	209.1	213.5	218.0
50	222.4	226.9	231.3	235.7	240.2	244.6	249.1	253.5	258.0	262.4
60	266.9	271.3	275.8	280.2	284.7	289.1	293.6	298.0	302.5	306.9
70	311.4	315.8	320.3	324.7	329.2	333.6	338.1	342.5	346.9	351.4
80	355.8	360.3	364.7	369.2	373.6	378.1	382.5	387.0	391.4	395.9
90	400.3	404.8	409.2	413.7	418.1	422.6	427.0	431.5	435.9	440.4
100	444.8	449.3	453.7	458.1	462.6	467.0	471.5	475.9	480.4	484.8

1. All values in this table are based on the relation, 1 lbf = 4.448 N. By manipulation of the decimal point any decimal value or multiple of a pounds force may be converted to its equivalent in Newtons.

Table GN-28a Masonry and Steel Stresses[1] - psi to MPa and kg/cm²

psi	MPa	kg/cm²	psi	MPa	kg/cm²	psi	MPa	kg/cm²
100	0.69	7.03	1000	6.90	70.30	4500	31.03	316.35
150	1.03	10.55	1100	7.58	77.33	5000	34.48	351.50
200	1.38	14.06	1167	8.05	82.04	5300	36.54	372.59
250	1.72	17.58	1200	8.27	84.36	10000	68.95	703.00
300	2.07	21.09	1333	9.19	93.71	16000	110.32	1124.80
333	2.30	23.41	1500	10.34	105.45	18000	124.11	1265.40
400	2.76	28.12	1667	11.49	117.19	20000	137.90	1406.00
500	3.45	35.15	2000	13.79	140.60	24000	165.48	1687.20
600	4.14	42.18	2400	16.55	168.72	26700	184.10	1877.01
667	4.60	46.89	2500	17.24	175.75	30000	206.85	2109.00
750	5.17	52.73	2700	18.62	189.81	32000	220.64	2249.60
800	5.52	56.24	3000	20.69	210.90	40000	275.80	2812.00
833	5.74	58.56	3500	24.13	246.50	50000	344.75	3515.00
900	6.21	63.27	4000	27.58	281.20	60000	413.70	4218.00

Note: Modulus of reinforcing steel = 29,000,000 psi = 199 955 MPa = 2 038 700 kg/cm².

1. Values in this table are based on the relations, 1 psi = 0.006 895 MPa = 0.0703 kg/cm².

Table GN-28b Pressure and Stress Equivalents[1] - Pounds per Square Inch to Kilogram per Square Centimeter

psi	0	1	2	3	4	5	6	7	8	9
	kg/cm²									
0	0.00	0.07	0.14	0.21	0.28	0.35	0.42	0.49	0.56	0.63
10	0.71	0.78	0.85	0.92	0.99	1.06	1.13	1.20	1.27	1.34
20	1.41	1.48	1.55	1.62	1.69	1.76	1.83	1.90	1.97	2.05
30	2.12	2.19	2.26	2.33	2.40	2.47	2.54	2.61	2.68	2.75
40	2.82	2.89	2.96	3.03	3.10	3.17	3.24	3.31	3.39	3.46
50	3.53	3.60	3.67	3.74	3.81	3.88	3.95	4.02	4.09	4.16
60	4.23	4.30	4.37	4.44	4.51	4.58	4.65	4.73	4.80	4.87
70	4.94	5.01	5.08	5.15	5.22	5.29	5.36	5.43	5.50	5.57
80	5.64	5.71	5.78	5.85	5.92	5.99	6.07	6.14	6.21	6.28
90	6.35	6.42	6.49	6.56	6.63	6.70	6.77	6.84	6.91	6.98
100	7.05	7.12	7.19	7.26	7.33	7.48	7.48	7.55	7.62	7.69

1. All values in this table are based on the relation, 1 psi = 0.0705 kg/cm². By manipulation of the decimal point any decimal value or multiple of pounds per square inch may be converted to its equivalent in kilograms per square centimenter.

Table GN-28c Pressure and Stress Equivalents[1] (psi to Kilopascals)

psi	0	1	2	3	4	5	6	7	8	9
	Kilopascals									
0	0.00	6.89	13.79	20.68	27.58	34.47	41.37	48.26	55.16	62.05
10	68.95	75.84	82.74	89.63	96.53	103.4	110.3	117.2	124.1	131.0
20	137.9	144.8	151.7	158.6	165.5	172.4	179.3	186.2	193.0	200.0
30	206.8	213.7	220.6	227.5	234.4	241.3	248.2	255.1	262.0	268.9
40	275.8	282.7	289.6	296.5	303.4	310.3	317.2	324.0	331.0	337.8
50	344.7	351.6	358.5	365.4	372.3	379.2	386.1	393.0	399.9	406.8
60	413.7	420.6	427.5	434.4	441.3	448.2	455.0	461.9	468.8	475.7
70	482.6	489.5	496.4	503.3	510.2	517.1	524.0	530.9	537.8	544.7
80	551.6	558.5	565.4	572.3	579.2	586.0	592.9	599.8	606.7	613.6
90	620.5	627.4	634.3	641.2	648.1	655.0	661.9	668.8	675.7	682.6
100	689.5	696.4	703.3	710.2	717.0	723.9	730.8	737.7	744.6	751.5

1. All values in this table are based on the relation, 1 psi = 6.895 kPa. By manipulation of the decimal point any decimal value or multiple of a psi may be converted to its equivalent in kilopascals.

Table GN-28d Pressure and Stress Equivalents[1] - Pounds per Square Foot to Pascals

psf	0	1	2	3	4	5	6	7	8	9
	Pascals									
0	0.0	47.9	95.8	143.7	191.6	239.5	287.4	335.3	383.2	431.1
10	479.0	526.9	574.8	622.7	670.6	718.5	766.4	814.3	862.2	910.1
20	958.0	1005.9	1053.8	1101.7	1149.6	1197.5	1245.4	1293.3	1341.2	1389.1
30	1437.0	1484.9	1532.8	1580.7	1628.6	1676.5	1724.4	1772.3	1820.2	1868.1
40	1916.0	1963.9	2011.8	2059.7	2107.6	2155.5	2203.4	2251.3	2299.2	2347.1
50	2395.0	2442.9	2490.8	2538.7	2586.6	2634.5	2682.4	2730.3	2778.2	2826.1
60	2874.0	2921.9	2969.8	3017.7	3065.6	3113.5	3161.4	3209.3	3257.2	3305.1
70	3353.0	3400.9	3448.8	3496.7	3544.6	3592.5	3640.4	3688.3	3736.2	3784.1
80	3832.0	3879.9	3927.8	3975.7	4023.6	4071.5	4119.4	4167.3	4215.2	4263.1
90	4311.0	4358.9	4406.8	4454.7	4502.6	4550.5	4598.4	4646.3	4694.2	4742.1
100	4790.0	4837.9	4885.8	4933.7	4981.6	5029.5	5077.4	5125.3	5173.2	5221.1

1. All values in this table are based on the relation, 1 psf = 47.90 Pa. By manipulation of the decimal point any decimal value or multiple of a psf may be converted to its equivalent in pascals.

Table GN-29a Moment Equivalents[1] - Foot Pounds Force to Newton Meters

ft lb	0	1	2	3	4	5	6	7	8	9
	N·m									
0	0.00	1.36	2.72	4.08	5.44	6.80	8.15	9.51	10.87	12.23
10	13.59	14.95	16.31	17.67	19.03	20.39	21.74	23.10	24.46	25.82
20	27.18	28.54	29.90	31.26	32.62	33.98	35.33	36.69	38.05	39.41
30	40.77	42.13	43.49	44.85	46.21	47.57	48.92	50.28	51.64	53.00
40	54.36	55.72	57.08	58.44	59.80	61.16	62.51	63.87	65.23	66.59
50	67.95	69.31	70.67	72.03	73.39	74.75	76.10	77.46	78.82	80.18
60	81.54	82.90	84.26	85.62	86.98	88.34	89.69	91.05	92.41	93.77
70	95.13	96.49	97.85	99.21	100.57	101.93	103.28	104.64	106.00	107.36
80	108.72	110.08	111.44	112.80	114.16	115.52	116.87	118.23	119.59	120.95
90	122.31	123.67	125.03	126.39	127.75	129.11	130.46	131.82	133.18	134.54
100	135.90	137.26	138.62	139.98	141.34	142.70	144.05	145.41	146.77	148.13

1. All values in this table are based on the relation, 1 ft lb = 1.359 N•m. By manipulation of the decimal point any decimal value or multiple of foot pounds may be converted to its equivalent in Newton meters.

Table GN-29b Moment Equivalents[1] - Foot Kips to Kilogram Meters

ft kips	0	1	2	3	4	5	6	7	8	9
	kg·m									
0	0.00	1.39	2.77	4.16	5.55	6.94	8.32	9.71	11.10	12.48
10	13.87	15.26	16.64	18.03	19.42	20.81	22.19	23.58	24.97	26.35
20	27.74	29.13	30.51	31.90	33.29	34.68	36.06	37.45	38.84	40.22
30	41.61	43.00	44.38	45.77	47.16	48.55	49.93	51.32	52.71	54.09
40	55.48	56.87	58.25	59.64	61.03	62.42	63.80	65.19	66.58	67.96
50	69.35	70.74	72.12	73.51	74.90	76.29	77.67	79.06	80.45	81.83
60	83.22	84.61	85.99	87.38	88.77	90.16	91.54	92.93	94.32	95.70
70	97.09	98.48	99.86	101.25	102.64	104.03	105.41	106.80	108.19	109.57
80	110.96	112.35	113.73	115.12	116.51	117.90	119.28	120.67	122.06	123.44
90	124.83	126.22	127.60	128.99	130.38	131.77	133.15	134.54	135.93	137.31
100	138.70	140.09	141.47	142.86	144.25	145.64	147.02	148.41	149.80	151.18

1. All values in this table are based on the relation, 1 ft k = 1.387 kg•m. By manipulation of the decimal point any decimal value or multiple of foot kips may be converted to its equivalent in kilograms meters.

Table GN-30 Pounds per Linear Foot Equivalents to Kilograms per Meter[1]

lb/ft	0	1	2	3	4	5	6	7	8	9
	kg/m									
0	0.00	1.49	2.99	4.48	5.97	7.46	8.96	10.45	11.94	13.44
10	14.93	16.42	17.91	19.41	20.90	22.39	23.88	25.38	26.87	28.36
20	29.86	31.35	32.84	34.33	35.83	37.32	38.81	40.31	41.80	43.29
30	44.78	46.28	47.77	49.26	50.75	52.25	53.74	55.23	56.73	58.22
40	59.71	61.20	62.70	64.19	65.68	67.18	68.67	70.16	71.65	73.15
50	74.64	76.13	77.62	79.12	80.61	82.10	83.60	85.09	86.58	88.07
60	89.57	91.06	92.55	94.05	95.54	97.03	98.52	100.02	101.51	103.00
70	104.49	105.99	107.48	108.97	110.47	111.96	113.45	114.94	116.44	117.93
80	119.42	120.92	122.41	123.90	125.39	126.89	128.38	129.87	131.36	132.86
90	134.35	135.84	137.34	138.83	140.32	141.81	143.31	144.80	146.29	147.79
100	149.28	150.77	152.26	153.76	155.25	156.74	158.23	159.73	161.22	162.71

1. All values in this table are based on the relation, 1 lb/ft = 1.49 kg/m. By manipulation of the decimal point any decimal value or multiple of pounds per foot may be converted to its equivalent in kilograms per meter.

Table GN-31 Moment per Unit Length Equivalents[1] – Foot Pounds Force per Foot to Newton Meters per Meters

ft lb/ft	0	1	2	3	4	5	6	7	8	9
	N·m/m									
0	0.00	4.45	8.90	13.34	17.79	22.24	26.69	31.14	35.58	40.03
10	44.48	48.93	53.38	57.82	62.27	66.72	71.17	75.62	80.06	84.51
20	88.96	93.41	97.86	102.30	106.75	111.20	115.65	120.10	124.54	128.99
30	133.44	137.89	142.34	146.78	151.23	155.68	160.13	164.58	164.58	173.47
40	177.92	182.37	186.82	191.26	195.71	200.16	204.61	209.06	209.06	217.95
50	222.40	226.85	231.30	235.74	240.19	244.64	249.09	253.54	253.54	262.43
60	266.88	271.33	275.78	280.22	284.67	289.12	293.57	298.02	298.02	306.91
70	311.36	315.81	320.26	324.70	329.15	333.60	338.05	342.50	342.50	351.39
80	355.84	360.29	364.74	369.18	373.63	378.08	382.53	386.98	386.98	395.87
90	400.32	404.77	409.22	413.66	418.11	422.56	427.01	431.46	431.46	440.35
100	444.80	449.25	453.70	458.14	462.59	467.04	471.49	475.94	475.94	484.83

1. All values in this table are based on the relation, 1 ft lb/ft = 4.448 N·m/m. By manipulation of the decimal point any decimal value or multiple of foot pounds per foot may be converted to its equivalent in Newton meters per meter.

Table GN-32 Allowable Compressive Stresses for Empirical Design of Masonry

Construction; Compressive Strength of Masonry Unit, Gross Area (psi)	Allowable Compressive Stresses[1] Based on Gross Cross-Sectional Area (psi)	
	Mortar Type	
	M or S	N
Solid masonry of brick and other solid units of clay or shale, sand-lime or concrete brick:		
8000 or greater	350	300
4500	225	200
2500	160	140
1500	115	100
Grouted masonry of clay or shale, sand-lime or concrete:		
4500 or greater	225	200
2500	160	140
1500	115	100
Solid masonry of solid concrete masonry units:		
3000 or greater	225	200
2000	160	140
1200	115	100
Masonry of hollow load bearing units:		
2000 or greater	140	120
1500	115	100
1000	75	70
700	60	55
Hollow walls (non-composite masonry bonded[2]):		
Solid units:		
2500 or greater	160	140
1500	115	100
Hollow units:	75	70
Stone ashlar masonry:		
Granite	720	640
Limestone or marble	450	400
Sandstone or cast stone	360	320
Rubble stone masonry:		
Coursed, rough or random	120	100

1. Linear interpolation shall be permitted for determining allowable stresses for masonry units having compressive strengths which are intermediate between those given in Table GN-32.
2. Where floor and roof loads are carried upon one wythe, the gross cross-sectional area is that of the wythe under load; if both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes. Walls bonded with metal ties shall be considered as non-composite walls, unless collar joints are filled with mortar or grout.

Table GN-91 Percent Tension Capacity of Anchor Bolts Based on Bolt Spacing[1,2,3]

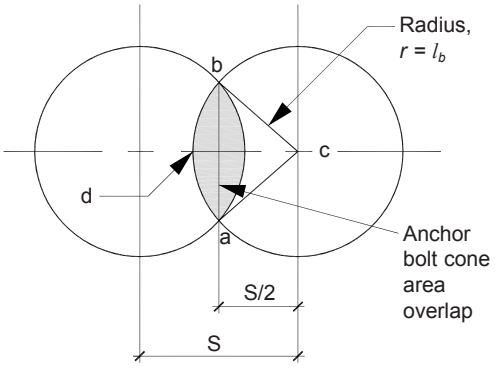
Per MSJC Code Section 2.1.4.2.2.1, the tension capacity of anchor bolts must be reduced if the areas of their tension (pullout) cones, A_p, overlap. The tensile capacity of such bolts must be determined by reducing, A_p, of the bolts by one half the overlapping area. The values in this table show the appropriate percent capacity reduction based on the spacing of the anchor bolts (see figure below).

Area of Segment, adb = Area of Sector, adbc - Area of Triangle, abc

$$= \pi l_b^2 \cos^{-1}\left(\frac{s}{2l_b}\right) - \left(\frac{s}{2}\right)\sqrt{l_b^2 - \left(\frac{s}{2}\right)^2}$$

Tension Cone Area, $A_p = \pi l_b^2$
Reduction % = Area of Segment, adb x $100/A_p$
Note to find the percent reduction, set $l_b = 1.0$

The anchor bolt must be embedded in a solid or grouted cell



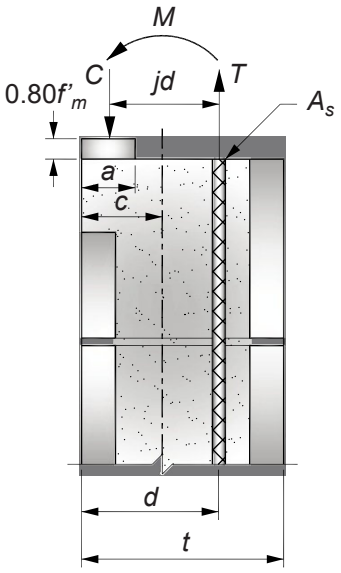
Spacing of Bolts, *s*	0.0	$0.1l_b$	$0.2l_b$	$0.3l_b$	$0.4l_b$	$0.5l_b$	$0.6l_b$	$0.7l_b$	$0.8l_b$	$0.9l_b$	$1.0l_b$
% Capacity	50	53	56	60	63	66	69	72	75	78	80
% Reduction	50	47	44	40	37	34	31	28	25	22	20
Spacing of Bolts, *s*		$1.1l_b$	$1.2l_b$	$1.3l_b$	$1.4l_b$	$1.5l_b$	$1.6l_b$	$1.7l_b$	$1.8l_b$	$1.9l_b$	$2.0l_b$
% Capacity		83	86	88	91	93	95	97	98	99	100
% Reduction		17	14	12	9	7	5	3	2	1	0

1. l_b = Embedment depth of anchor bolts, inches.
2. Embedment length shall be measured perpendicular from the masonry surface to the bearing head of the anchor head for headed anchor bolts (to the bearing surface of the bent end, minus one anchor bolt diameter, for bent bar anchor bolts).
3. The minimum effective embedment length required for placement of headed and bent bar anchor bolts shall be the greater of 2 in. or 4 bolt diameters.
4. The projected area, A_p, shall be reduced by half the overlapping area, between adjacent bolts, and all of any area outside the contiguous solid masonry assembly in which the anchor bolt is placed.

STRENGTH DESIGN TABLES AND DIAGRAMS

Based on the
International
Building
Code
Requirements

TABLE SD-2 Coefficients for Flexural Strength Design:
f'_m = 1500 psi and f_y = 60,000 psi



$f'_m = 1{,}500$ psi $\qquad f_y = 60{,}000$ psi $\qquad \phi = 0.80$

$$K_u = \phi f'_m\, q(1 - 0.625q) = \frac{M_u}{bd^2} \qquad q = \rho\left(\frac{f_y}{f'_m}\right)$$

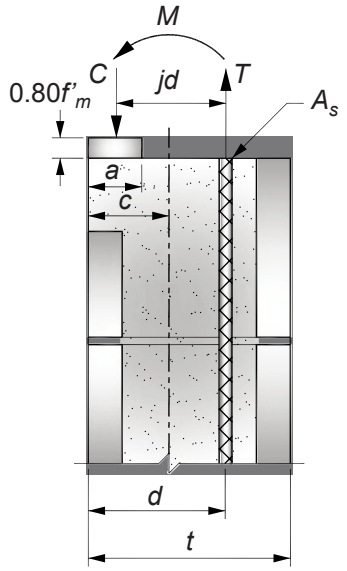
$$a_u = \frac{K_u}{12{,}000\,\rho} = \frac{\phi f_y (1 - 0.625q)}{12{,}000} \qquad \rho = q\left(\frac{f'_m}{f_y}\right) = \frac{A_s}{bd}$$

$$A_s = \frac{M_u}{a_u d} \qquad j = 1 - \frac{1}{2}\left(\frac{a}{d}\right)$$

$$\frac{a}{d} = 0.80\,\frac{c}{d} \qquad \frac{c}{d} = 1.5625q$$

q	K_u	a_u	ρ	c/d	a/d	j
0.030	35.3	3.93	0.0008	0.047	0.038	0.981
0.040	46.8	3.90	0.0010	0.063	0.050	0.975
0.050	58.1	3.88	0.0013	0.078	0.063	0.969
0.060	69.3	3.85	0.0015	0.094	0.075	0.963
0.070	80.3	3.83	0.0018	0.109	0.088	0.956
0.080	91.2	3.80	0.0020	0.125	0.100	0.950
0.090	101.9	3.78	0.0023	0.141	0.113	0.944
0.100	112.5	3.75	0.0025	0.156	0.125	0.938
0.110	122.9	3.73	0.0028	0.172	0.138	0.931
0.120	133.2	3.70	0.0030	0.188	0.150	0.925
0.130	143.3	3.68	0.0033	0.203	0.163	0.919
0.140	153.3	3.65	0.0035	0.219	0.175	0.913
0.150	163.1	3.63	0.0038	0.234	0.188	0.906
0.160	172.8	3.60	0.0040	0.250	0.200	0.900
0.170	182.3	3.58	0.0043	0.266	0.213	0.894
0.180	191.7	3.55	0.0045	0.281	0.225	0.888
0.190	200.9	3.53	0.0048	0.297	0.238	0.881
0.200	210.0	3.50	0.0050	0.313	0.250	0.875
0.210	218.9	3.48	0.0053	0.328	0.263	0.869
0.220	227.7	3.45	0.0055	0.344	0.275	0.863
0.230	236.3	3.43	0.0058	0.359	0.288	0.856
0.240	244.8	3.40	0.0060	0.375	0.300	0.850
0.250	253.1	3.38	0.0063	0.391	0.313	0.844
0.260	261.3	3.35	0.0065	0.406	0.325	0.838
0.270	269.3	3.33	0.0068	0.422	0.338	0.831
0.280	277.2	3.30	0.0070	0.438	0.350	0.825
0.290	284.9	3.28	0.0073	0.453	0.363	0.819
0.300	292.5	3.25	0.0075	0.469	0.375	0.813
0.310	299.9	3.23	0.0078	0.484	0.388	0.806
0.320	307.2	3.20	0.0080	0.500	0.400	0.800

TABLE SD-3 Coefficients for Flexural Strength Design:
f'_m = 2000 psi and f_y = 60,000 psi



$f'_m = 2{,}000$ psi $\qquad f_y = 60{,}000$ psi $\qquad \phi = 0.80$

$$K_u = \phi f'_m\, q(1 - 0.625q) = \frac{M_u}{bd^2} \qquad q = \rho\left(\frac{f_y}{f'_m}\right)$$

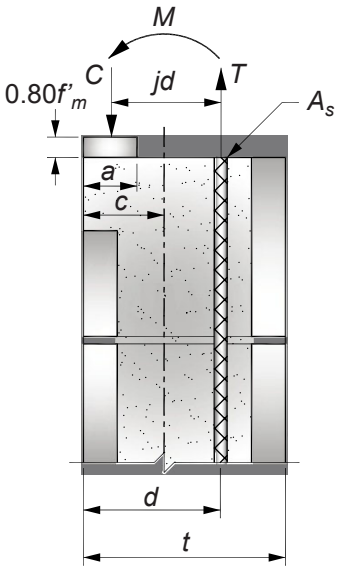
$$a_u = \frac{K_u}{12{,}000\,\rho} = \frac{\phi f_y (1 - 0.625q)}{12{,}000} \qquad \rho = q\left(\frac{f'_m}{f_y}\right) = \frac{A_s}{bd}$$

$$A_s = \frac{M_u}{a_u d} \qquad j = 1 - \frac{1}{2}\left(\frac{a}{d}\right)$$

$$\frac{a}{d} = 0.80\,\frac{c}{d} \qquad \frac{c}{d} = 1.5625q$$

q	K_u	a_u	ρ	c/d	a/d	j
0.030	47.1	3.93	0.0010	0.047	0.038	0.981
0.040	62.4	3.90	0.0013	0.063	0.050	0.975
0.050	77.5	3.88	0.0017	0.078	0.063	0.969
0.060	92.4	3.85	0.0020	0.094	0.075	0.963
0.070	107.1	3.83	0.0023	0.109	0.088	0.956
0.080	121.6	3.80	0.0027	0.125	0.100	0.950
0.090	135.9	3.78	0.0030	0.141	0.113	0.944
0.100	150.0	3.75	0.0033	0.156	0.125	0.938
0.110	163.9	3.73	0.0037	0.172	0.138	0.931
0.120	177.6	3.70	0.0040	0.188	0.150	0.925
0.130	191.1	3.68	0.0043	0.203	0.163	0.919
0.140	204.4	3.65	0.0047	0.219	0.175	0.913
0.150	217.5	3.63	0.0050	0.234	0.188	0.906
0.160	230.4	3.60	0.0053	0.250	0.200	0.900
0.170	243.1	3.58	0.0057	0.266	0.213	0.894
0.180	255.6	3.55	0.0060	0.281	0.225	0.888
0.190	267.9	3.53	0.0063	0.297	0.238	0.881
0.200	280.0	3.50	0.0067	0.313	0.250	0.875
0.210	291.9	3.48	0.0070	0.328	0.263	0.869
0.220	303.6	3.45	0.0073	0.344	0.275	0.863
0.230	315.1	3.43	0.0077	0.359	0.288	0.856
0.240	326.4	3.40	0.0080	0.375	0.300	0.850
0.250	337.5	3.38	0.0083	0.391	0.313	0.844
0.260	348.4	3.35	0.0087	0.406	0.325	0.838
0.270	359.1	3.33	0.0090	0.422	0.338	0.831
0.280	369.6	3.30	0.0093	0.438	0.350	0.825
0.290	379.9	3.28	0.0097	0.453	0.363	0.819
0.300	390.0	3.25	0.0100	0.469	0.375	0.813
0.310	399.9	3.23	0.0103	0.484	0.388	0.806
0.320	409.6	3.20	0.0107	0.500	0.400	0.800

TABLE SD-4 Coefficients for Flexural Strength Design:
f'_m = 2500 psi and f_y = 60,000 psi



$f'_m = 2{,}500$ psi $\qquad f_y = 60{,}000$ psi $\qquad \phi = 0.80$

$$K_u = \phi f'_m\, q(1 - 0.625q) = \frac{M_u}{bd^2} \qquad q = \rho\left(\frac{f_y}{f'_m}\right)$$

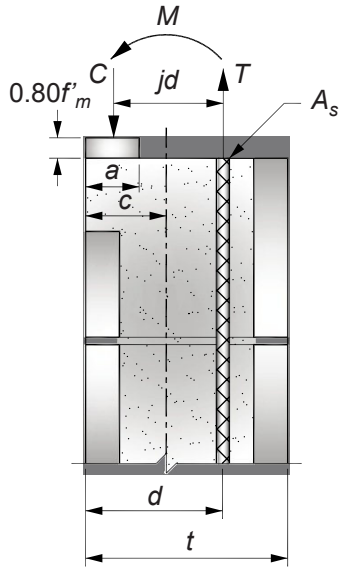
$$a_u = \frac{K_u}{12{,}000\,\rho} = \frac{\phi f_y(1 - 0.625q)}{12{,}000} \qquad \rho = q\left(\frac{f'_m}{f_y}\right) = \frac{A_s}{bd}$$

$$A_s = \frac{M_u}{a_u d} \qquad j = 1 - \frac{1}{2}\left(\frac{a}{d}\right)$$

$$\frac{a}{d} = 0.80\,\frac{c}{d} \qquad \frac{c}{d} = 1.5625q$$

q	K_u	a_u	ρ	c/d	a/d	j
0.030	58.9	3.93	0.0013	0.047	0.038	0.981
0.040	78.0	3.90	0.0017	0.063	0.050	0.975
0.050	96.9	3.88	0.0021	0.078	0.063	0.969
0.060	115.5	3.85	0.0025	0.094	0.075	0.963
0.070	133.9	3.83	0.0029	0.109	0.088	0.956
0.080	152.0	3.80	0.0033	0.125	0.100	0.950
0.090	169.9	3.78	0.0038	0.141	0.113	0.944
0.100	187.5	3.75	0.0042	0.156	0.125	0.938
0.110	204.9	3.73	0.0046	0.172	0.138	0.931
0.120	222.0	3.70	0.0050	0.188	0.150	0.925
0.130	238.9	3.68	0.0054	0.203	0.163	0.919
0.140	255.5	3.65	0.0058	0.219	0.175	0.913
0.150	271.9	3.63	0.0063	0.234	0.188	0.906
0.160	288.0	3.60	0.0067	0.250	0.200	0.900
0.170	303.9	3.58	0.0071	0.266	0.213	0.894
0.180	319.5	3.55	0.0075	0.281	0.225	0.888
0.190	334.9	3.53	0.0079	0.297	0.238	0.881
0.200	350.0	3.50	0.0083	0.313	0.250	0.875
0.210	364.9	3.48	0.0088	0.328	0.263	0.869
0.220	379.5	3.45	0.0092	0.344	0.275	0.863
0.230	393.9	3.43	0.0096	0.359	0.288	0.856
0.240	408.0	3.40	0.0100	0.375	0.300	0.850
0.250	421.9	3.38	0.0104	0.391	0.313	0.844
0.260	435.5	3.35	0.0108	0.406	0.325	0.838
0.270	448.9	3.33	0.0113	0.422	0.338	0.831
0.280	462.0	3.30	0.0117	0.438	0.350	0.825
0.290	474.9	3.28	0.0121	0.453	0.363	0.819
0.300	487.5	3.25	0.0125	0.469	0.375	0.813
0.310	499.9	3.23	0.0129	0.484	0.388	0.806
0.320	512.0	3.20	0.0133	0.500	0.400	0.800

TABLE SD-5 Coefficients for Flexural Strength Design:
f'_m = 3000 psi and f_y = 60,000 psi



$f'_m = 3{,}000$ psi $\qquad f_y = 60{,}000$ psi $\qquad \phi = 0.80$

$$K_u = \phi f'_m\, q(1 - 0.625q) = \frac{M_u}{bd^2} \qquad q = \rho\left(\frac{f_y}{f'_m}\right)$$

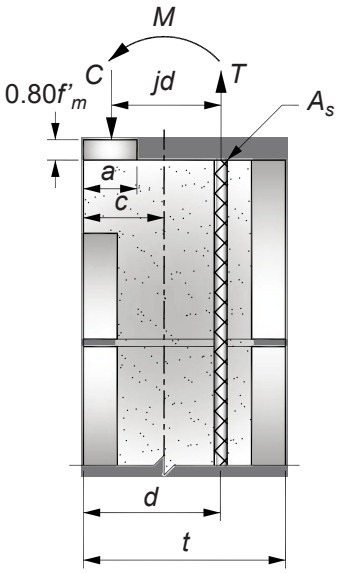
$$a_u = \frac{K_u}{12{,}000\,\rho} = \frac{\phi f_y (1 - 0.625q)}{12{,}000} \qquad \rho = q\left(\frac{f'_m}{f_y}\right) = \frac{A_s}{bd}$$

$$A_s = \frac{M_u}{a_u d} \qquad j = 1 - \frac{1}{2}\left(\frac{a}{d}\right)$$

$$\frac{a}{d} = 0.80\,\frac{c}{d} \qquad \frac{c}{d} = 1.5625q$$

q	K_u	a_u	ρ	c/d	a/d	j
0.030	70.7	3.93	0.0015	0.047	0.038	0.981
0.040	93.6	3.90	0.0020	0.063	0.050	0.975
0.050	116.3	3.88	0.0025	0.078	0.063	0.969
0.060	138.6	3.85	0.0030	0.094	0.075	0.963
0.070	160.7	3.83	0.0035	0.109	0.088	0.956
0.080	182.4	3.80	0.0040	0.125	0.100	0.950
0.090	203.9	3.78	0.0045	0.141	0.113	0.944
0.100	225.0	3.75	0.0050	0.156	0.125	0.938
0.110	245.9	3.73	0.0055	0.172	0.138	0.931
0.120	266.4	3.70	0.0060	0.188	0.150	0.925
0.130	286.7	3.68	0.0065	0.203	0.163	0.919
0.140	306.6	3.65	0.0070	0.219	0.175	0.913
0.150	326.3	3.63	0.0075	0.234	0.188	0.906
0.160	345.6	3.60	0.0080	0.250	0.200	0.900
0.170	364.7	3.58	0.0085	0.266	0.213	0.894
0.180	383.4	3.55	0.0090	0.281	0.225	0.888
0.190	401.9	3.53	0.0095	0.297	0.238	0.881
0.200	420.0	3.50	0.0100	0.313	0.250	0.875
0.210	437.9	3.48	0.0105	0.328	0.263	0.869
0.220	455.4	3.45	0.0110	0.344	0.275	0.863
0.230	472.7	3.43	0.0115	0.359	0.288	0.856
0.240	489.6	3.40	0.0120	0.375	0.300	0.850
0.250	506.3	3.38	0.0125	0.391	0.313	0.844
0.260	522.6	3.35	0.0130	0.406	0.325	0.838
0.270	538.7	3.33	0.0135	0.422	0.338	0.831
0.280	554.4	3.30	0.0140	0.438	0.350	0.825
0.290	569.9	3.28	0.0145	0.453	0.363	0.819
0.300	585.0	3.25	0.0150	0.469	0.375	0.813
0.310	599.9	3.23	0.0155	0.484	0.388	0.806
0.320	614.4	3.20	0.0160	0.500	0.400	0.800

TABLE SD-6 Coefficients for Flexural Strength Design:
f'_m = 3500 psi and f_y = 60,000 psi



$f'_m = 3{,}500$ psi $\qquad f_y = 60{,}000$ psi $\qquad \phi = 0.80$

$$K_u = \phi f'_m\, q(1 - 0.625q) = \frac{M_u}{bd^2} \qquad q = \rho\left(\frac{f_y}{f'_m}\right)$$

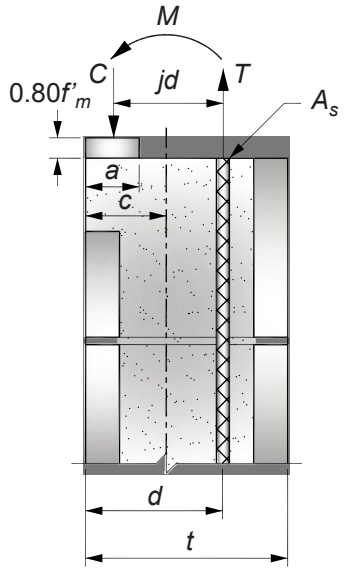
$$a_u = \frac{K_u}{12{,}000\,\rho} = \frac{\phi f_y(1 - 0.625q)}{12{,}000} \qquad \rho = q\left(\frac{f'_m}{f_y}\right) = \frac{A_s}{bd}$$

$$A_s = \frac{M_u}{a_u d} \qquad j = 1 - \frac{1}{2}\left(\frac{a}{d}\right)$$

$$\frac{a}{d} = 0.80\,\frac{c}{d} \qquad \frac{c}{d} = 1.5625q$$

q	K_u	a_u	ρ	c/d	a/d	j
0.030	82.4	3.93	0.0018	0.047	0.038	0.981
0.040	109.2	3.90	0.0023	0.063	0.050	0.975
0.050	135.6	3.88	0.0029	0.078	0.063	0.969
0.060	161.7	3.85	0.0035	0.094	0.075	0.963
0.070	187.4	3.83	0.0041	0.109	0.088	0.956
0.080	212.8	3.80	0.0047	0.125	0.100	0.950
0.090	237.8	3.78	0.0053	0.141	0.113	0.944
0.100	262.5	3.75	0.0058	0.156	0.125	0.938
0.110	286.8	3.73	0.0064	0.172	0.138	0.931
0.120	310.8	3.70	0.0070	0.188	0.150	0.925
0.130	334.4	3.68	0.0076	0.203	0.163	0.919
0.140	357.7	3.65	0.0082	0.219	0.175	0.913
0.150	380.6	3.63	0.0088	0.234	0.188	0.906
0.160	403.2	3.60	0.0093	0.250	0.200	0.900
0.170	425.4	3.58	0.0099	0.266	0.213	0.894
0.180	447.3	3.55	0.0105	0.281	0.225	0.888
0.190	468.8	3.53	0.0111	0.297	0.238	0.881
0.200	490.0	3.50	0.0117	0.313	0.250	0.875
0.210	510.8	3.48	0.0123	0.328	0.263	0.869
0.220	531.3	3.45	0.0128	0.344	0.275	0.863
0.230	551.4	3.43	0.0134	0.359	0.288	0.856
0.240	571.2	3.40	0.0140	0.375	0.300	0.850
0.250	590.6	3.38	0.0146	0.391	0.313	0.844
0.260	609.7	3.35	0.0152	0.406	0.325	0.838
0.270	628.4	3.33	0.0158	0.422	0.338	0.831
0.280	646.8	3.30	0.0163	0.438	0.350	0.825
0.290	664.8	3.28	0.0169	0.453	0.363	0.819
0.300	682.5	3.25	0.0175	0.469	0.375	0.813
0.310	699.8	3.23	0.0181	0.484	0.388	0.806
0.320	716.8	3.20	0.0187	0.500	0.400	0.800

TABLE SD-7 Coefficients for Flexural Strength Design:
f'_m = 4000 psi and f_y = 60,000 psi



$f'_m = 4{,}000$ psi $\qquad f_y = 60{,}000$ psi $\qquad \phi = 0.80$

$$K_u = \phi f'_m\, q(1 - 0.625q) = \frac{M_u}{bd^2} \qquad q = \rho\left(\frac{f_y}{f'_m}\right)$$

$$a_u = \frac{K_u}{12{,}000\,\rho} = \frac{\phi f_y (1 - 0.625q)}{12{,}000} \qquad \rho = q\left(\frac{f'_m}{f_y}\right) = \frac{A_s}{bd}$$

$$A_s = \frac{M_u}{a_u d} \qquad j = 1 - \frac{1}{2}\left(\frac{a}{d}\right)$$

$$\frac{a}{d} = 0.80\,\frac{c}{d} \qquad \frac{c}{d} = 1.5625q$$

q	K_u	a_u	ρ	c/d	a/d	j
0.030	94.2	3.93	0.0020	0.047	0.038	0.981
0.040	124.8	3.90	0.0027	0.063	0.050	0.975
0.050	155.0	3.88	0.0033	0.078	0.063	0.969
0.060	184.8	3.85	0.0040	0.094	0.075	0.963
0.070	214.2	3.83	0.0047	0.109	0.088	0.956
0.080	243.2	3.80	0.0053	0.125	0.100	0.950
0.090	271.8	3.78	0.0060	0.141	0.113	0.944
0.100	300.0	3.75	0.0067	0.156	0.125	0.938
0.110	327.8	3.73	0.0073	0.172	0.138	0.931
0.120	355.2	3.70	0.0080	0.188	0.150	0.925
0.130	382.2	3.68	0.0087	0.203	0.163	0.919
0.140	408.8	3.65	0.0093	0.219	0.175	0.913
0.150	435.0	3.63	0.0100	0.234	0.188	0.906
0.160	460.8	3.60	0.0107	0.250	0.200	0.900
0.170	486.2	3.58	0.0113	0.266	0.213	0.894
0.180	511.2	3.55	0.0120	0.281	0.225	0.888
0.190	535.8	3.53	0.0127	0.297	0.238	0.881
0.200	560.0	3.50	0.0133	0.313	0.250	0.875
0.210	583.8	3.48	0.0140	0.328	0.263	0.869
0.220	607.2	3.45	0.0147	0.344	0.275	0.863
0.230	630.2	3.43	0.0153	0.359	0.288	0.856
0.240	652.8	3.40	0.0160	0.375	0.300	0.850
0.250	675.0	3.38	0.0167	0.391	0.313	0.844
0.260	696.8	3.35	0.0173	0.406	0.325	0.838
0.270	718.2	3.33	0.0180	0.422	0.338	0.831
0.280	739.2	3.30	0.0187	0.438	0.350	0.825
0.290	759.8	3.28	0.0193	0.453	0.363	0.819
0.300	780.0	3.25	0.0200	0.469	0.375	0.813
0.310	799.8	3.23	0.0207	0.484	0.388	0.806
0.320	819.2	3.20	0.0213	0.500	0.400	0.800

TABLE SD-12 Design Coefficient *q* for the Determination of the Reinforcing Ratio ρ

$$\frac{M_u}{f'_m\, bd^2} = q(1-0.625q) \qquad q = \rho\left(\frac{f_y}{f'_m}\right) \qquad \rho = q\left(\frac{f'_m}{f_y}\right) \qquad \text{Nominal Moment } M_n = \phi M_u$$

$$\frac{M_u}{f'_m\, bd^2} = A_s f_y\left(d - \frac{a}{2}\right)f'_m\, bd^2 = q(1-0.625q) \text{ where } q = \rho\left(\frac{f_y}{f'_m}\right) \text{ and } a = \frac{A_s f_y}{0.80 f'_m\, b}$$

Design: Using the factored moment, M_u, enter the table with $\frac{M_u}{\phi f'_m\, bd^2}$; find *a* and compute the steel percentage ρ from $\rho = q\left(\frac{f'_m}{f_y}\right)$.

Investigation: Enter the table with *q* from $q = \rho\left(\frac{f_y}{f'_m}\right)$; find the value of $\frac{M_n}{f'_m\, bd^2}$ and solve for the nominal moment strength, M_n.

	$\frac{M_u}{f'_m\, bd^2}$									
q	**0**	**0.001**	**0.002**	**0.003**	**0.004**	**0.005**	**0.006**	**0.007**	**0.008**	**0.009**
0	0.0000	0.0010	0.0020	0.0030	0.0040	0.0050	0.0060	0.0070	0.0080	0.0089
0.01	0.0099	0.0109	0.0119	0.0129	0.0139	0.0149	0.0158	0.0168	0.0178	0.0188
0.02	0.0198	0.0207	0.0217	0.0227	0.0236	0.0246	0.0256	0.0265	0.0275	0.0285
0.03	0.0294	0.0304	0.0314	0.0323	0.0333	0.0342	0.0352	0.0361	0.0371	0.0380
0.04	0.0390	0.0399	0.0409	0.0418	0.0428	0.0437	0.0447	0.0456	0.0466	0.0475
0.05	0.0484	0.0494	0.0503	0.0512	0.0522	0.0531	0.0540	0.0550	0.0559	0.0568
0.06	0.0578	0.0587	0.0596	0.0605	0.0614	0.0624	0.0633	0.0642	0.0651	0.0660
0.07	0.0669	0.0678	0.0688	0.0697	0.0706	0.0715	0.0724	0.0733	0.0742	0.0751
0.08	0.0760	0.0769	0.0778	0.0787	0.0796	0.0805	0.0814	0.0823	0.0832	0.0840
0.09	0.0849	0.0858	0.0867	0.0876	0.0885	0.0894	0.0902	0.0911	0.0920	0.0929
0.10	0.0938	0.0946	0.0955	0.0964	0.0972	0.0981	0.0990	0.0998	0.1007	0.1016
0.11	0.1024	0.1033	0.1042	0.1050	0.1059	0.1067	0.1076	0.1084	0.1093	0.1101
0.12	0.1110	0.1118	0.1127	0.1135	0.1144	0.1152	0.1161	0.1169	0.1178	0.1186
0.13	0.1194	0.1203	0.1211	0.1219	0.1228	0.1236	0.1244	0.1253	0.1261	0.1269
0.14	0.1278	0.1286	0.1294	0.1302	0.1310	0.1319	0.1327	0.1335	0.1343	0.1351
0.15	0.1359	0.1367	0.1376	0.1384	0.1392	0.1400	0.1408	0.1416	0.1424	0.1432
0.16	0.1440	0.1448	0.1456	0.1464	0.1472	0.1480	0.1488	0.1496	0.1504	0.1511
0.17	0.1519	0.1527	0.1535	0.1543	0.1551	0.1559	0.1566	0.1574	0.1582	0.1590
0.18	0.1598	0.1605	0.1613	0.1621	0.1628	0.1636	0.1644	0.1651	0.1659	0.1667
0.19	0.1674	0.1682	0.1690	0.1697	0.1705	0.1712	0.1720	0.1727	0.1735	0.1742
0.20	0.1750	0.1757	0.1765	0.1772	0.1780	0.1787	0.1795	0.1802	0.1810	0.1817
0.21	0.1824	0.1832	0.1839	0.1846	0.1854	0.1861	0.1868	0.1876	0.1883	0.1890
0.22	0.1898	0.1905	0.1912	0.1919	0.1926	0.1934	0.1941	0.1948	0.1955	0.1962
0.23	0.1969	0.1976	0.1984	0.1991	0.1998	0.2005	0.2012	0.2019	0.2026	0.2033
0.24	0.2040	0.2047	0.2054	0.2061	0.2068	0.2075	0.2082	0.2089	0.2096	0.2102
0.25	0.2109	0.2116	0.2123	0.2130	0.2137	0.2144	0.2150	0.2157	0.2164	0.2171
0.26	0.2178	0.2184	0.2191	0.2198	0.2204	0.2211	0.2218	0.2224	0.2231	0.2238
0.27	0.2244	0.2251	0.2258	0.2264	0.2271	0.2277	0.2284	0.2290	0.2297	0.2303
0.28	0.2310	0.2316	0.2323	0.2329	0.2336	0.2342	0.2349	0.2355	0.2362	0.2368
0.29	0.2374	0.2381	0.2387	0.2393	0.2400	0.2406	0.2412	0.2419	0.2425	0.2431
0.30	0.2438	0.2444	0.2450	0.2456	0.2462	0.2469	0.2475	0.2481	0.2487	0.2493
0.31	0.2499	0.2505	0.2512	0.2518	0.2524	0.2530	0.2536	0.2542	0.2548	0.2554
0.32	0.2560	0.2566	0.2572	0.2578	0.2584	0.2590	0.2596	0.2602	0.2608	0.2613
0.33	0.2619	0.2625	0.2631	0.2637	0.2643	0.2649	0.2654	0.2660	0.2666	0.2672
0.34	0.2678	0.2683	0.2689	0.2695	0.2700	0.2706	0.2712	0.2717	0.2723	0.2729
0.35	0.2734	0.2740	0.2746	0.2751	0.2757	0.2762	0.2768	0.2773	0.2779	0.2784
0.36	0.2790	0.2795	0.2801	0.2806	0.2812	0.2817	0.2823	0.2828	0.2834	0.2839
0.37	0.2844	0.2850	0.2855	0.2860	0.2866	0.2871	0.2876	0.2882	0.2887	0.2892
0.38	0.2898	0.2903	0.2908	0.2913	0.2918	0.2924	0.2929	0.2934	0.2939	0.2944
0.39	0.2949	0.2954	0.2960	0.2965	0.2970	0.2975	0.2980	0.2985	0.2990	0.2995

TABLE SD-14 Moment Capacity of Walls and Beams:
f'_m = 1,500 psi and f_y = 60,000 psi

f'_m = 1,500 psi f_y = 60,000 psi

$M_u = \phi d^2 f'_m q\,(1 - 0.625q)$ ft-kip

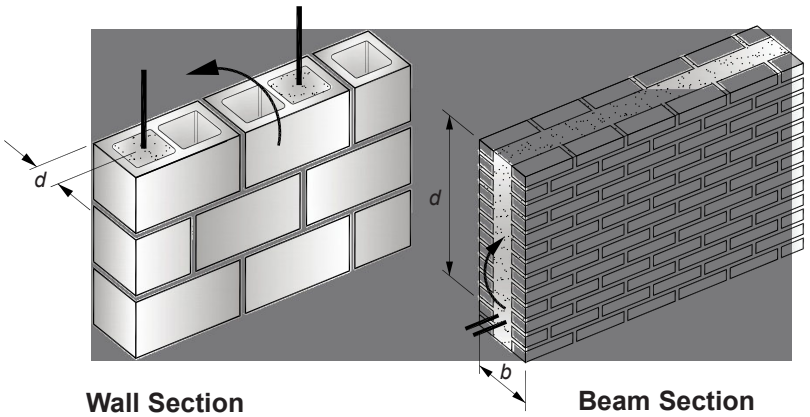
(b = 1 foot)

$q = \rho\left(\frac{f_y}{f'_m}\right)$ ϕ = 0.80

for members where:

$\frac{M_u}{V_u d_v} < 1$ and $R \leq 1.5$

Moment Capacity in ft-kip/ft



Wall Section **Beam Section**

ρ[1,2]	*d* (inches)											
	2.8	**3.8**	**4.8**	**5.3**	**7.3**	**9.0**	**12.0**	**18.0**	**22.0**	**26.0**	**30.0**	**36.0**
0.0010	0.37	0.68	1.08	1.31	2.49	3.79	6.74	15.16	22.65	31.64	42.12	60.65
0.0015	0.54	1.00	1.60	1.95	3.69	5.61	9.98	22.45	33.54	46.85	62.37	89.81
0.0020	0.72	1.32	2.10	2.56	4.86	7.39	13.13	29.55	44.14	61.65	82.08	118.20
0.0025	0.88	1.62	2.59	3.16	6.00	9.11	16.20	36.45	54.45	76.05	101.25	145.80
0.0030	1.04	1.92	3.07	3.74	7.10	10.79	19.18	43.16	64.47	90.04	119.88	172.63
0.0035	1.20	2.21	3.53	4.31	8.17	12.42	22.08	49.67	74.20	103.63	137.97	198.68
0.0040	1.35	2.50	3.98	4.85	9.21	14.00	24.88	55.99	83.64	116.81	155.52	223.95
0.0045	1.50	2.77	4.42	5.38	10.22	15.53	27.60	62.11	92.78	129.59	172.53	248.44
0.0050	1.65	3.03	4.84	5.90	11.19	17.01	30.24	68.04	101.64	141.96	189.00	272.16
0.0055	1.79	3.29	5.25	6.40	12.13	18.44	32.79	73.77	110.21	153.93	204.93	295.10
0.0060	1.92	3.53	5.64	6.88	13.05	19.83	35.25	79.32	118.48	165.48	220.32	317.26
0.0065	2.05	3.77	6.02	7.34	13.92	21.17	37.63	84.66	126.47	176.64	235.17	338.64
0.0070	2.17	4.00	6.39	7.79	14.77	22.45	39.92	89.81	134.16	187.39	249.48	359.25
0.0075	2.29	4.22	6.74	8.22	15.59	23.69	42.12	94.77	141.57	197.73	263.25	379.08
0.0080	2.41	4.44	7.08	8.63	16.37	24.88	44.24	99.53	148.68	207.67	276.48	398.13
0.0085	2.52	4.64	7.40	9.03	17.12	26.03	46.27	104.10	155.51	217.20	289.17	416.40
0.0090	2.63	4.84	7.72	9.41	17.86	27.14	48.25	108.56	162.17	226.50	301.56	434.24
0.0095	2.73	5.02	8.02	9.77	18.54	28.18	50.11	112.74	168.41	235.22	313.16	450.95
0.0100	2.82	5.20	8.30	10.12	19.20	29.18	51.87	116.72	174.36	243.52	324.22	466.87

1. ρ in excess of 0.0088 applies to clay masonry (shaded), but exceeds $\rho_{balanced}$ for concrete masonry.
2. ρ may be limited by MSJC Code Section 3.3.3.5 maximum area of flexural tensile reinforcement; dashed lines represent limit based on MSJC Code Section 3.3.3.5.1 for concrete and clay masonry respectively, where $R \geq 1.5$ or $M_u/V_u d_v \geq 1$.

TABLE SD-15 Moment Capacity of Walls and Beams:
f'_m = 2,000 psi and f_y = 60,000 psi

f'_m = 2,000 psi f_y = 60,000 psi

$M_u = \phi d^2 f'_m q\,(1 - 0.625q)$ ft-kip

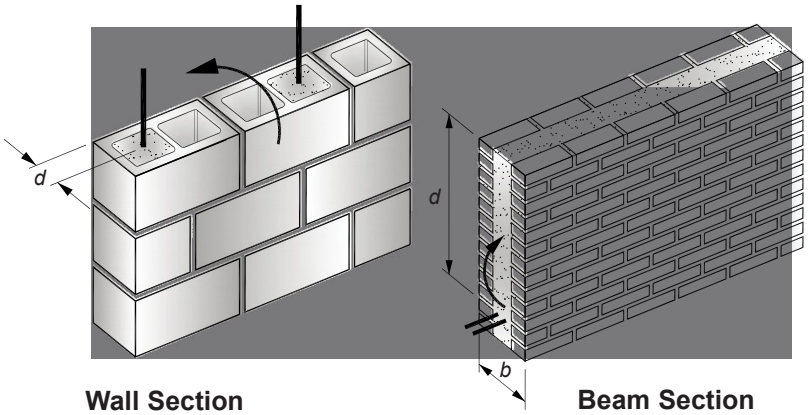
(b = 1 foot)

$q = \rho\left(\frac{f_y}{f'_m}\right)$ ϕ = 0.80

for members where:

$\frac{M_u}{V_u d_v} < 1$ and $R \leq 1.5$

Moment Capacity in ft-kip/ft



Wall Section **Beam Section**

$\rho^{1,2}$	*d* (inches)											
	2.8	**3.8**	**4.8**	**5.3**	**7.3**	**9.0**	**12.0**	**18.0**	**22.0**	**26.0**	**30.0**	**36.0**
0.0010	0.37	0.68	1.09	1.32	2.51	3.82	6.78	15.26	22.80	31.84	42.39	61.04
0.0015	0.55	1.01	1.61	1.97	3.73	5.67	10.08	22.67	33.87	47.30	62.98	90.69
0.0020	0.72	1.33	2.13	2.60	4.92	7.48	13.31	29.94	44.72	62.46	83.16	119.75
0.0025	0.90	1.65	2.64	3.21	6.10	9.26	16.47	37.06	55.36	77.32	102.94	148.23
0.0030	1.07	1.96	3.13	3.82	7.24	11.01	19.57	44.03	65.78	91.87	122.31	176.13
0.0035	1.23	2.27	3.62	4.41	8.37	12.71	22.60	50.86	75.98	106.12	141.28	203.44
0.0040	1.39	2.56	4.09	4.99	9.46	14.39	25.57	57.54	85.96	120.06	159.84	230.17
0.0045	1.55	2.86	4.56	5.56	10.54	16.02	28.48	64.08	95.72	133.70	178.00	256.32
0.0050	1.71	3.14	5.01	6.11	11.59	17.62	31.32	70.47	105.27	147.03	195.75	281.88
0.0055	1.86	3.42	5.46	6.65	12.62	19.18	34.10	76.72	114.60	160.06	213.10	306.86
0.0060	2.00	3.69	5.89	7.18	13.62	20.70	36.81	82.81	123.71	172.79	230.04	331.26
0.0065	2.15	3.96	6.31	7.70	14.60	22.19	39.45	88.77	132.60	185.21	246.58	355.07
0.0070	2.29	4.22	6.73	8.20	15.56	23.64	42.03	94.58	141.28	197.32	262.71	378.30
0.0075	2.43	4.47	7.13	8.69	16.49	25.06	44.55	100.24	149.74	209.14	278.44	400.95
0.0080	2.56	4.71	7.52	9.17	17.39	26.44	47.00	105.75	157.98	220.65	293.76	423.01
0.0085	2.69	4.95	7.90	9.63	18.28	27.78	49.39	111.12	166.00	231.85	308.68	444.50
0.0090	2.82	5.19	8.27	10.09	19.14	29.09	51.71	116.35	173.80	242.75	323.19	465.39
0.0095	2.94	5.41	8.63	10.53	19.97	30.36	53.97	121.43	181.39	253.35	337.30	485.71
0.0100	3.06	5.63	8.99	10.96	20.78	31.59	56.16	126.36	188.76	263.64	351.00	505.44
0.0105	3.17	5.84	9.33	11.37	21.57	32.79	58.29	131.15	195.91	273.63	364.30	524.59
0.0110	3.29	6.05	9.66	11.77	22.33	33.95	60.35	135.79	202.84	283.31	377.19	543.15
0.0115	3.39	6.25	9.98	12.16	23.07	35.07	62.35	140.28	209.56	292.69	389.68	561.14
0.0120	3.50	6.45	10.29	12.54	23.79	36.16	64.28	144.63	216.06	301.77	401.76	578.53
0.0125	3.60	6.63	10.58	12.90	24.48	37.21	66.15	148.84	222.34	310.54	413.44	595.35
0.0130	3.70	6.81	10.87	13.26	25.15	38.22	67.95	152.90	228.40	319.00	424.71	611.58

1. ρ in excess of 0.0117 applies to clay masonry (shaded), but exceeds $\rho_{balanced}$ for concrete masonry.
2. ρ may be limited by MSJC Code Section 3.3.3.5 maximum area of flexural tensile reinforcement; dashed lines represent limit based on MSJC Code Section 3.3.3.5.1 for concrete and clay masonry respectively, where $R \geq 1.5$ or $M_u/V_u d_v \geq 1$.

TABLE SD-16 Moment Capacity of Walls and Beams:
f'_m = 2,500 psi and f_y = 60,000 psi

f'_m = 2,500 psi f_y = 60,000 psi

$M_u = \phi d^2 f'_m q\,(1 - 0.625q)$ ft-kip

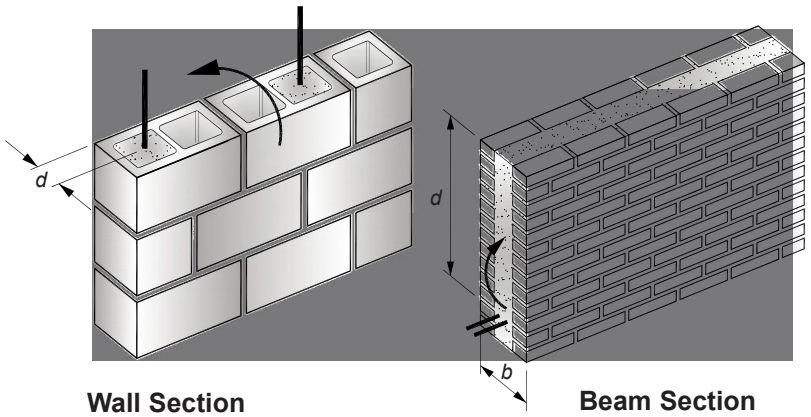
(b = 1 foot)

$q = \rho\left(\frac{f_y}{f'_m}\right)$ ϕ = 0.80

for members where:

$\frac{M_u}{V_u d_v} < 1$ and $R \leq 1.5$

Moment Capacity in ft-kip/ft



Wall Section **Beam Section**

ρ[1,2]	*d* (inches)											
	2.8	**3.8**	**4.8**	**5.3**	**7.3**	**9.0**	**12.0**	**18.0**	**22.0**	**26.0**	**30.0**	**36.0**
0.0010	0.37	0.68	1.09	1.33	2.52	3.83	6.81	15.32	22.88	31.96	42.55	61.27
0.0015	0.55	1.02	1.62	1.98	3.75	5.70	10.13	22.80	34.06	47.58	63.34	91.21
0.0020	0.73	1.34	2.15	2.62	4.96	7.54	13.41	30.17	45.07	62.95	83.81	120.68
0.0025	0.91	1.67	2.66	3.24	6.15	9.36	16.63	37.42	55.90	78.08	103.95	149.69
0.0030	1.08	1.99	3.17	3.86	7.33	11.14	19.80	44.56	66.56	92.96	123.77	178.23
0.0035	1.25	2.30	3.67	4.47	8.48	12.89	22.92	51.57	77.04	107.61	143.26	206.30
0.0040	1.41	2.61	4.16	5.07	9.62	14.62	25.99	58.48	87.35	122.00	162.43	233.90
0.0045	1.58	2.91	4.64	5.66	10.73	16.32	29.00	65.26	97.49	136.16	181.28	261.04
0.0050	1.74	3.21	5.11	6.24	11.83	17.98	31.97	71.93	107.45	150.07	199.80	287.71
0.0055	1.90	3.50	5.58	6.80	12.91	19.62	34.88	78.48	117.23	163.74	218.00	313.92
0.0060	2.05	3.78	6.04	7.36	13.97	21.23	37.74	84.91	126.85	177.17	235.87	339.66
0.0065	2.21	4.07	6.49	7.91	15.01	22.81	40.55	91.23	136.28	190.35	253.42	364.93
0.0070	2.36	4.34	6.93	8.45	16.03	24.36	43.30	97.43	145.55	203.29	270.65	389.73
0.0075	2.50	4.61	7.36	8.97	17.03	25.88	46.01	103.52	154.64	215.98	287.55	414.07
0.0080	2.65	4.88	7.79	9.49	18.01	27.37	48.66	109.49	163.55	228.43	304.13	437.94
0.0085	2.79	5.14	8.20	10.00	18.97	28.83	51.26	115.34	172.29	240.64	320.38	461.35
0.0090	2.93	5.40	8.61	10.50	19.91	30.27	53.81	121.07	180.86	252.61	336.31	484.29
0.0095	3.07	5.65	9.01	10.98	20.84	31.67	56.31	126.69	189.25	264.33	351.92	506.76
0.0100	3.20	5.89	9.40	11.46	21.74	33.05	58.75	132.19	197.47	275.81	367.20	528.77
0.0105	3.33	6.13	9.78	11.93	22.63	34.39	61.15	137.58	205.52	287.04	382.16	550.31
0.0110	3.46	6.37	10.16	12.38	23.49	35.71	63.49	142.85	213.39	298.03	396.79	571.38
0.0115	3.58	6.60	10.52	12.83	24.34	37.00	65.78	148.00	221.08	308.78	411.10	591.99
0.0120	3.70	6.82	10.88	13.27	25.17	38.26	68.01	153.03	228.60	319.29	425.09	612.13
0.0125	3.82	7.04	11.23	13.69	25.98	39.49	70.20	157.95	235.95	329.55	438.75	631.80
0.0130	3.94	7.25	11.57	14.11	26.77	40.69	72.33	162.75	243.12	339.57	452.09	651.01
0.0135	4.05	7.46	11.91	14.52	27.54	41.86	74.42	167.44	250.12	349.34	465.10	669.75
0.0140	4.16	7.67	12.23	14.91	28.29	43.00	76.45	172.01	256.95	358.87	477.79	688.02
0.0145	4.27	7.86	12.55	15.30	29.02	44.11	78.43	176.46	263.60	368.16	490.16	705.83
0.0150	4.37	8.06	12.86	15.67	29.74	45.20	80.35	180.79	270.07	377.21	502.20	723.17
0.0155	4.48	8.25	13.16	16.04	30.43	46.25	82.23	185.01	276.37	386.01	513.92	740.04
0.0160	4.58	8.43	13.45	16.40	31.10	47.28	84.05	189.11	282.50	394.57	525.31	756.45
0.0165	4.67	8.61	13.73	16.74	31.76	48.27	85.82	193.10	288.45	402.88	536.38	772.39

1. ρ in excess of 0.0146 applies to clay masonry (shaded), but exceeds $\rho_{balanced}$ for concrete masonry.
2. ρ may be limited by MSJC Code Section 3.3.3.5 maximum area of flexural tensile reinforcement; dashed lines represent limit based on MSJC Code Section 3.3.3.5.1 for concrete and clay masonry respectively, where $R \geq 1.5$ or $M_u/V_u d_v \geq 1$.

TABLE SD-17 Moment Capacity of Walls and Beams:
f'_m = 3,000 psi and f_y = 60,000 psi

f'_m = 3,000 psi f_y = 60,000 psi

$M_u = \phi d^2 f'_m q\,(1 - 0.625q)$ ft-kip

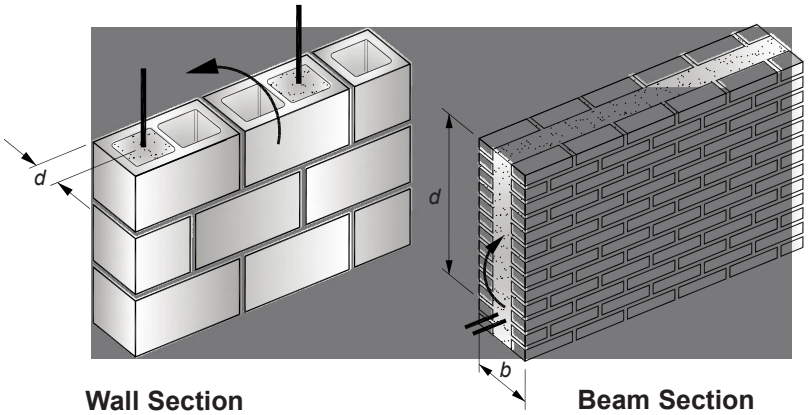
(b = 1 foot)

$q = \rho\left(\frac{f_y}{f'_m}\right)$ ϕ = 0.80

for members where:

$\frac{M_u}{V_u d_v} < 1$ and $R \leq 1.5$

Moment Capacity in ft-kip/ft



Wall Section **Beam Section**

ρ[1,2]	*d* (inches)											
	2.8	**3.8**	**4.8**	**5.3**	**7.3**	**9.0**	**12.0**	**18.0**	**22.0**	**26.0**	**30.0**	**36.0**
0.0010	0.37	0.68	1.09	1.33	2.53	3.84	6.83	15.36	22.94	32.04	42.66	61.43
0.0015	0.55	1.02	1.63	1.98	3.76	5.72	10.17	22.89	34.19	47.76	63.59	91.56
0.0020	0.73	1.35	2.16	2.63	4.99	7.58	13.48	30.33	45.30	63.27	84.24	121.31
0.0025	0.91	1.68	2.68	3.27	6.19	9.42	16.74	37.67	56.27	78.59	104.63	150.66
0.0030	1.09	2.00	3.19	3.89	7.39	11.23	19.96	44.91	67.08	93.69	124.74	179.63
0.0035	1.26	2.32	3.70	4.51	8.56	13.01	23.13	52.05	77.75	108.60	144.59	208.20
0.0040	1.43	2.63	4.20	5.12	9.72	14.77	26.27	59.10	88.28	123.30	164.16	236.39
0.0045	1.60	2.94	4.70	5.73	10.86	16.51	29.35	66.05	98.66	137.80	183.47	264.19
0.0050	1.76	3.25	5.18	6.32	11.99	18.23	32.40	72.90	108.90	152.10	202.50	291.60
0.0055	1.93	3.55	5.66	6.91	13.10	19.91	35.40	79.66	118.99	166.19	221.27	318.62
0.0060	2.09	3.85	6.14	7.48	14.20	21.58	38.36	86.31	128.94	180.09	239.76	345.25
0.0065	2.25	4.14	6.60	8.05	15.28	23.22	41.28	92.87	138.74	193.78	257.99	371.50
0.0070	2.40	4.43	7.06	8.61	16.34	24.83	44.15	99.34	148.39	207.26	275.94	397.35
0.0075	2.56	4.71	7.52	9.16	17.39	26.43	46.98	105.71	157.91	220.55	293.63	422.82
0.0080	2.71	4.99	7.96	9.71	18.42	27.99	49.77	111.97	167.27	233.63	311.04	447.90
0.0085	2.86	5.27	8.40	10.24	19.43	29.54	52.51	118.15	176.49	246.50	328.19	472.59
0.0090	3.01	5.54	8.83	10.77	20.43	31.06	55.21	124.22	185.57	259.18	345.06	496.89
0.0095	3.15	5.80	9.26	11.29	21.41	32.55	57.87	130.20	194.50	271.65	361.67	520.80
0.0100	3.29	6.06	9.68	11.80	22.38	34.02	60.48	136.08	203.28	283.92	378.00	544.32
0.0105	3.43	6.32	10.09	12.30	23.33	35.47	63.05	141.86	211.92	295.99	394.07	567.45
0.0110	3.57	6.58	10.49	12.79	24.27	36.89	65.58	147.55	220.41	307.85	409.86	590.20
0.0115	3.71	6.83	10.89	13.28	25.19	38.28	68.06	153.14	228.76	319.51	425.39	612.55
0.0120	3.84	7.07	11.28	13.75	26.09	39.66	70.50	158.63	236.97	330.97	440.64	634.52
0.0125	3.97	7.31	11.66	14.22	26.98	41.01	72.90	164.03	245.03	342.23	455.63	656.10
0.0130	4.10	7.55	12.04	14.68	27.85	42.33	75.25	169.32	252.94	353.28	470.34	677.29
0.0135	4.22	7.78	12.41	15.13	28.70	43.63	77.57	174.52	260.71	364.13	484.79	698.09
0.0140	4.35	8.01	12.77	15.57	29.54	44.91	79.83	179.63	268.33	374.77	498.96	718.50
0.0145	4.47	8.23	13.13	16.01	30.37	46.16	82.06	184.63	275.81	385.22	512.87	738.53
0.0150	4.59	8.45	13.48	16.43	31.17	47.39	84.24	189.54	283.14	395.46	526.50	758.16
0.0155	4.70	8.66	13.82	16.85	31.97	48.59	86.38	194.35	290.33	405.50	539.87	777.41
0.0160	4.82	8.87	14.16	17.26	32.74	49.77	88.47	199.07	297.37	415.33	552.96	796.26
0.0165	4.93	9.08	14.48	17.66	33.50	50.92	90.53	203.68	304.27	424.97	565.79	814.73
0.0170	5.04	9.28	14.81	18.05	34.24	52.05	92.53	208.20	311.02	434.40	578.34	832.81
0.0175	5.15	9.48	15.12	18.43	34.97	53.16	94.50	212.63	317.63	443.63	590.63	850.50
0.0180	5.25	9.67	15.43	18.81	35.68	54.24	96.42	216.95	324.09	452.65	602.64	867.80
0.0185	5.35	9.86	15.73	19.18	36.38	55.29	98.30	221.18	330.40	461.47	614.39	884.71
0.0190	5.45	10.04	16.02	19.53	37.06	56.33	100.14	225.31	336.57	470.09	625.86	901.24
0.0195	5.55	10.22	16.31	19.88	37.72	57.34	101.93	229.34	342.60	478.51	637.07	917.37
0.0200	5.64	10.40	16.59	20.22	38.37	58.32	103.68	233.28	348.48	486.72	648.00	933.12

1. ρ in excess of 0.0175 applies to clay masonry (shaded), but exceeds $\rho_{balanced}$ for concrete masonry.
2. ρ may be limited by MSJC Code Section 3.3.3.5 maximum area of flexural tensile reinforcement; dashed lines represent limit based on MSJC Code Section 3.3.3.5.1 for concrete and clay masonry respectively, where $R \geq 1.5$ or $M_u/V_u d_v \geq 1$.

TABLE SD-18 Moment Capacity of Walls and Beams:
f'_m = 3,500 psi and f_y = 60,000 psi

f'_m = 3,500 psi f_y = 60,000 psi

$M_u = \phi d^2 f'_m q\,(1 - 0.625q)$ ft-kip

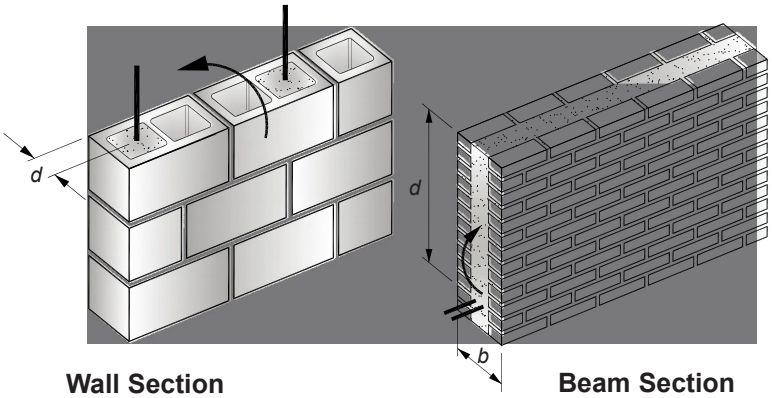
(b = 1 foot)

$q = \rho\left(\frac{f_y}{f'_m}\right)$ ϕ = 0.80

for members where:

$\frac{M_u}{V_u d_v} < 1$ and $R \leq 1.5$

Moment Capacity in ft-kip/ft



Wall Section **Beam Section**

ρ[1,2]	*d* (inches)											
	2.8	**3.8**	**4.8**	**5.3**	**7.3**	**9.0**	**12.0**	**18.0**	**22.0**	**26.0**	**30.0**	**36.0**
0.0010	0.37	0.69	1.09	1.33	2.53	3.85	6.84	15.39	22.98	32.10	42.74	61.54
0.0015	0.56	1.02	1.63	1.99	3.78	5.74	10.20	22.95	34.29	47.89	63.76	91.81
0.0020	0.74	1.36	2.16	2.64	5.01	7.61	13.53	30.44	45.47	63.51	84.55	121.75
0.0025	0.92	1.69	2.69	3.28	6.22	9.46	16.82	37.84	56.52	78.95	105.11	151.35
0.0030	1.09	2.01	3.21	3.91	7.43	11.29	20.07	45.16	67.46	94.22	125.43	180.63
0.0035	1.27	2.33	3.73	4.54	8.62	13.10	23.28	52.39	78.26	109.31	145.53	209.56
0.0040	1.44	2.65	4.23	5.16	9.79	14.89	26.46	59.54	88.95	124.23	165.39	238.17
0.0045	1.61	2.97	4.74	5.77	10.96	16.65	29.60	66.61	99.50	138.98	185.03	266.44
0.0050	1.78	3.28	5.23	6.38	12.10	18.40	32.71	73.59	109.94	153.55	204.43	294.38
0.0055	1.95	3.59	5.72	6.98	13.24	20.12	35.78	80.50	120.25	167.95	223.60	321.98
0.0060	2.11	3.89	6.21	7.57	14.36	21.83	38.81	87.31	130.43	182.17	242.54	349.25
0.0065	2.28	4.19	6.69	8.15	15.47	23.51	41.80	94.05	140.49	196.22	261.24	376.19
0.0070	2.44	4.49	7.16	8.73	16.56	25.17	44.76	100.70	150.43	210.10	279.72	402.80
0.0075	2.60	4.78	7.63	9.30	17.64	26.82	47.67	107.27	160.24	223.80	297.96	429.07
0.0080	2.75	5.07	8.09	9.86	18.71	28.44	50.56	113.75	169.93	237.33	315.98	455.01
0.0085	2.91	5.35	8.54	10.42	19.76	30.04	53.40	120.15	179.49	250.69	333.76	480.61
0.0090	3.06	5.64	8.99	10.96	20.80	31.62	56.21	126.47	188.93	263.87	351.31	505.88
0.0095	3.21	5.91	9.44	11.51	21.83	33.18	58.98	132.71	198.24	276.88	368.63	530.82
0.0100	3.36	6.19	9.87	12.04	22.84	34.71	61.71	138.86	207.43	289.71	385.71	555.43
0.0105	3.51	6.46	10.31	12.56	23.84	36.23	64.41	144.93	216.49	302.37	402.57	579.70
0.0110	3.65	6.73	10.73	13.08	24.82	37.73	67.07	150.91	225.43	314.86	419.19	603.64
0.0115	3.79	6.99	11.15	13.60	25.79	39.20	69.69	156.81	234.25	327.17	435.59	627.25
0.0120	3.94	7.25	11.56	14.10	26.75	40.66	72.28	162.63	242.94	339.31	451.75	650.52
0.0125	4.07	7.50	11.97	14.60	27.69	42.09	74.83	168.36	251.51	351.28	467.68	673.46
0.0130	4.21	7.76	12.37	15.09	28.62	43.50	77.34	174.02	259.95	363.07	483.38	696.06
0.0135	4.35	8.00	12.77	15.57	29.54	44.90	79.82	179.58	268.27	374.69	498.84	718.34
0.0140	4.48	8.25	13.16	16.05	30.44	46.27	82.25	185.07	276.46	386.13	514.08	740.28
0.0145	4.61	8.49	13.54	16.51	31.33	47.62	84.65	190.47	284.53	397.40	529.08	761.88
0.0150	4.74	8.73	13.92	16.97	32.20	48.95	87.02	195.79	292.47	408.50	543.86	783.15
0.0155	4.86	8.96	14.30	17.43	33.06	50.26	89.34	201.02	300.29	419.42	558.40	804.09
0.0160	4.99	9.19	14.66	17.87	33.91	51.54	91.63	206.18	307.99	430.17	572.71	824.70
0.0165	5.11	9.41	15.02	18.31	34.74	52.81	93.89	211.24	315.56	440.74	586.79	844.97
0.0170	5.23	9.64	15.38	18.75	35.56	54.06	96.10	216.23	323.01	451.14	600.63	864.91
0.0175	5.35	9.86	15.72	19.17	36.37	55.28	98.28	221.13	330.33	461.37	614.25	884.52
0.0180	5.47	10.07	16.07	19.59	37.16	56.49	100.42	225.95	337.53	471.42	627.63	903.79
0.0185	5.58	10.28	16.40	20.00	37.94	57.67	102.53	230.68	344.60	481.30	640.79	922.73
0.0190	5.69	10.49	16.73	20.40	38.71	58.83	104.59	235.34	351.55	491.01	653.71	941.34
0.0195	5.81	10.69	17.06	20.80	39.46	59.98	106.62	239.90	358.37	500.54	666.40	959.61
0.0200	5.91	10.89	17.38	21.19	40.20	61.10	108.62	244.39	365.07	509.90	678.86	977.55
0.0205	6.02	11.09	17.69	21.57	40.92	62.20	110.57	248.79	371.65	519.08	691.08	995.16
0.0210	6.12	11.28	18.00	21.94	41.63	63.28	112.49	253.11	378.10	528.09	703.08	1012.44
0.0215	6.23	11.47	18.30	22.31	42.33	64.34	114.38	257.34	384.43	536.93	714.84	1029.38
0.0220	6.33	11.65	18.60	22.67	43.01	65.37	116.22	261.50	390.63	545.59	726.38	1045.98
0.0225	6.43	11.84	18.88	23.02	43.68	66.39	118.03	265.56	396.71	554.08	737.68	1062.26
0.0230	6.52	12.01	19.17	23.37	44.33	67.39	119.80	269.55	402.66	562.39	748.75	1078.20
0.0235	6.62	12.19	19.45	23.71	44.98	68.36	121.53	273.45	408.49	570.53	759.59	1093.81

1. ρ in excess of 0.0204 applies to clay masonry (shaded), but exceeds $\rho_{balanced}$ for concrete masonry.
2. ρ may be limited by MSJC Code Section 3.3.3.5 maximum area of flexural tensile reinforcement; dashed lines represent limit based on MSJC Code Section 3.3.3.5.1 for concrete and clay masonry respectively, where $R \geq 1.5$ or $M_u/V_u d_v \geq 1$.

TABLE SD-19 Moment Capacity of Walls and Beams:
f'_m = 4,000 psi and f_y = 60,000 psi

f'_m = 4,000 psi f_y = 60,000 psi

$M_u = \phi d^2 f'_m q\,(1 - 0.625q)$ ft-kip

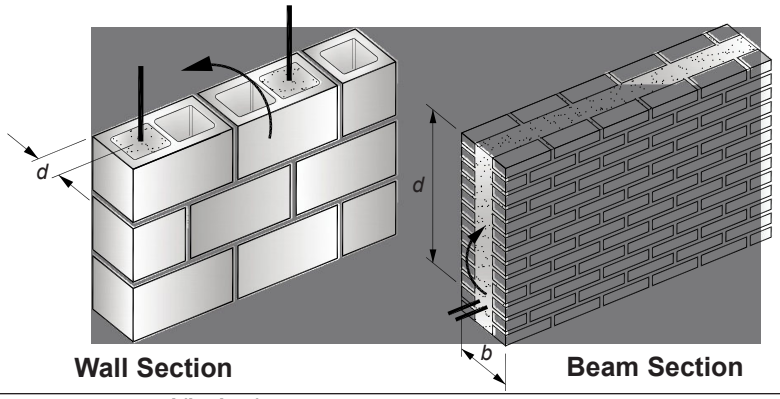
(b = 1 foot)

$q = \rho\left(\frac{f_y}{f'_m}\right)$ ϕ = 0.80

for members where:

$\frac{M_u}{V_u d_v} < 1$ and $R \leq 1.5$

Moment Capacity in ft-kip/ft



Wall Section **Beam Section**

ρ[1,2]	*d* (inches)											
	2.8	**3.8**	**4.8**	**5.3**	**7.3**	**9.0**	**12.0**	**18.0**	**22.0**	**26.0**	**30.0**	**36.0**
0.0010	0.37	0.69	1.10	1.34	2.53	3.85	6.85	15.41	23.01	32.14	42.80	61.62
0.0015	0.56	1.02	1.64	1.99	3.78	5.75	10.22	23.00	34.36	47.99	63.89	92.00
0.0020	0.74	1.36	2.17	2.65	5.02	7.63	13.56	30.52	45.59	63.68	84.78	122.08
0.0025	0.92	1.69	2.70	3.29	6.24	9.49	16.88	37.97	56.72	79.22	105.47	151.88
0.0030	1.10	2.02	3.22	3.93	7.46	11.34	20.15	45.34	67.74	94.61	125.96	181.38
0.0035	1.27	2.35	3.74	4.56	8.66	13.16	23.40	52.65	78.64	109.84	146.24	210.58
0.0040	1.45	2.67	4.26	5.19	9.85	14.97	26.61	59.88	89.44	124.92	166.32	239.50
0.0045	1.62	2.99	4.77	5.81	11.03	16.76	29.79	67.03	100.13	139.86	186.20	268.13
0.0050	1.79	3.30	5.27	6.43	12.19	18.53	32.94	74.12	110.72	154.64	205.88	296.46
0.0055	1.96	3.62	5.77	7.03	13.34	20.28	36.06	81.13	121.19	169.26	225.35	324.50
0.0060	2.13	3.92	6.26	7.63	14.48	22.02	39.14	88.06	131.55	183.74	244.62	352.25
0.0065	2.30	4.23	6.75	8.23	15.61	23.73	42.19	94.93	141.81	198.06	263.69	379.71
0.0070	2.46	4.53	7.23	8.82	16.73	25.43	45.21	101.72	151.95	212.23	282.56	406.88
0.0075	2.62	4.83	7.71	9.40	17.84	27.11	48.20	108.44	161.99	226.25	301.22	433.76
0.0080	2.78	5.13	8.18	9.98	18.93	28.77	51.15	115.08	171.92	240.12	319.68	460.34
0.0085	2.94	5.42	8.65	10.55	20.01	30.41	54.07	121.66	181.74	253.83	337.94	486.63
0.0090	3.10	5.71	9.11	11.11	21.08	32.04	56.96	128.16	191.45	267.39	356.00	512.63
0.0095	3.26	6.00	9.57	11.67	22.14	33.65	59.82	134.59	201.05	280.80	373.85	538.34
0.0100	3.41	6.28	10.02	12.22	23.18	35.24	62.64	140.94	210.54	294.06	391.50	563.76
0.0105	3.56	6.56	10.47	12.76	24.21	36.81	65.43	147.22	219.92	307.17	408.95	588.89
0.0110	3.71	6.84	10.91	13.30	25.24	38.36	68.19	153.43	229.20	320.12	426.20	613.72
0.0115	3.86	7.11	11.35	13.83	26.24	39.89	70.92	159.57	238.36	332.92	443.24	638.26
0.0120	4.01	7.38	11.78	14.36	27.24	41.41	73.61	165.63	247.42	345.57	460.08	662.52
0.0125	4.15	7.65	12.20	14.88	28.23	42.90	76.28	171.62	256.37	358.07	476.72	686.48
0.0130	4.30	7.91	12.62	15.39	29.20	44.38	78.90	177.54	265.21	370.41	493.16	710.14
0.0135	4.44	8.17	13.04	15.90	30.16	45.84	81.50	183.38	273.94	382.61	509.39	733.52
0.0140	4.58	8.43	13.45	16.40	31.11	47.29	84.07	189.15	282.56	394.65	525.42	756.60
0.0145	4.71	8.68	13.86	16.89	32.05	48.71	86.60	194.85	291.07	406.54	541.25	779.40
0.0150	4.85	8.93	14.26	17.38	32.97	50.12	89.10	200.48	299.48	418.28	556.88	801.90
0.0155	4.99	9.18	14.65	17.86	33.89	51.51	91.57	206.03	307.77	429.86	572.30	824.11
0.0160	5.12	9.43	15.04	18.34	34.79	52.88	94.00	211.51	315.96	441.29	587.52	846.03
0.0165	5.25	9.67	15.42	18.81	35.68	54.23	96.41	216.91	324.03	452.57	602.54	867.66
0.0170	5.38	9.91	15.80	19.27	36.55	55.56	98.78	222.25	332.00	463.70	617.36	888.99
0.0175	5.51	10.14	16.18	19.72	37.42	56.88	101.12	227.51	339.86	474.68	631.97	910.04
0.0180	5.63	10.37	16.55	20.17	38.27	58.17	103.42	232.70	347.61	485.50	646.38	930.79
0.0185	5.75	10.60	16.91	20.62	39.11	59.45	105.69	237.81	355.25	496.18	660.59	951.25
0.0190	5.88	10.82	17.27	21.05	39.94	60.71	107.94	242.85	362.78	506.70	674.60	971.42
0.0195	6.00	11.04	17.62	21.49	40.76	61.96	110.14	247.82	370.21	517.06	688.40	991.29
0.0200	6.12	11.26	17.97	21.91	41.57	63.18	112.32	252.72	377.52	527.28	702.00	1010.88
0.0205	6.23	11.48	18.31	22.33	42.36	64.39	114.46	257.54	384.73	537.34	715.40	1030.17
0.0210	6.35	11.69	18.65	22.74	43.14	65.57	116.58	262.29	391.82	547.26	728.60	1049.18
0.0215	6.46	11.90	18.98	23.15	43.91	66.74	118.65	266.97	398.81	557.02	741.59	1067.89
0.0220	6.57	12.10	19.31	23.55	44.67	67.89	120.70	271.58	405.69	566.62	754.38	1086.31
0.0225	6.68	12.31	19.63	23.94	45.41	69.03	122.72	276.11	412.46	576.08	766.97	1104.44
0.0230	6.79	12.50	19.95	24.32	46.15	70.14	124.70	280.57	419.12	585.38	779.36	1122.27
0.0235	6.90	12.70	20.26	24.70	46.87	71.24	126.65	284.95	425.67	594.53	791.54	1139.82
0.0240	7.00	12.89	20.57	25.08	47.58	72.32	128.56	289.27	432.12	603.53	803.52	1157.07
0.0245	7.10	13.08	20.87	25.45	48.27	73.38	130.45	293.51	438.45	612.38	815.30	1174.03
0.0250	7.20	13.27	21.17	25.81	48.96	74.42	132.30	297.68	444.68	621.08	826.88	1190.70
0.0255	7.30	13.45	21.46	26.16	49.63	75.44	134.12	301.77	450.79	629.62	838.25	1207.08
0.0260	7.40	13.63	21.75	26.51	50.30	76.45	135.91	305.79	456.80	638.01	849.42	1223.16
0.0265	7.49	13.80	22.03	26.85	50.94	77.43	137.66	309.74	462.70	646.25	860.39	1238.96

1. ρ in excess of 0.0233 applies to clay masonry (shaded), but exceeds $\rho_{balanced}$ for concrete masonry.
2. ρ may be limited by MSJC Code Section 3.3.3.5 maximum area of flexural tensile reinforcement; dashed lines represent limit based on MSJC Code Section 3.3.3.5.1 for concrete and clay masonry respectively, where $R \geq 1.5$ or $M_u/V_u d_v \geq 1$.

Table SD-22 Standard Bends and Hooks and Basic Development Length Provided

BAR	Bar Diameter (in.)	f_y (ksi)	Minimum Diameters of Bend[2] (in.)	Minimum Extension Beyond Bend[3] (in.)		Development Length Provided[1] (in.)
				180-degree Hook	90-degree Bend	
#3 (#10)	0.375	40	1.875	2.5	4.5	4.88
#3 (#10)	0.375	50, 60	2.25	2.5	4.5	4.88
#4 (#13)	0.500	40	2.5	2.5	6	6.5
#4 (#13)	0.500	50, 60	3	2.5	6	6.5
#5 (#16)	0.625	40	3.125	2.5	7.5	8.12
#5 (#16)	0.625	50, 60	3.75	2.5	7.5	8.12
#6 (#19)	0.750	40	3.75	3	9	9.75
#6 (#19)	0.750	50, 60	4.5	3	9	9.75
#7 (#22)	0.875	40	4.375	3.5	10.5	11.38
#7 (#22)	0.875	50, 60	5.25	3.5	10.5	11.38
#8 (#25)	1.000	50, 60	6	4	12	13
#9 (#29)	1.128	50, 60	9.0	4.5	13.5	14.66

1. Development length provided by the hook or bend - based on MSJC Code Section 3.3.3.2 Eq. 3-14.
2. MSJC Code Section 1.13.6 Table 1.13.6 for Hooks and Bends other than for stirrups and ties.
3. MSJC Code Section 1.13.5 for Hooks and Bends other than for stirrups and ties.

Table SD-24 Modulus of Rupture (f_r) for Clay and Concrete Masonry (psi)[2]

Direction of Flexural Tensile Stress and Masonry Type	Mortar Types			
	Portland Cement/Lime or Mortar Cement		Masonry Cement or Air Entrained Portland Cement/Lime	
	M or S	N	M or S	N
Normal to bed joints				
Solid units	100	75	60	38
Hollow units[1]				
Ungrouted	63	48	38	23
Fully grouted	163	158	153	145
Parallel to bed joints in running bond				
Solid units	200	150	120	75
Hollow units				
Ungrouted and partially grouted	125	95	75	48
Fully grouted	200	150	120	75
Parallel to bed joints in stack bond	0	0	0	0

1. For partially grouted masonry, modulus of rupture shall be determined on the basis of linear interpolation between fully grouted hollow units and ungrouted hollow units based on amount (percentage) of grouting.
2. Based on MSJC Code Table 3.1.8.2.1.

Table SD-26 Maximum Nominal Shear Stress Provided by the Masonry, V_m, psi

$M_u/V_u d_v$	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
[4.0 - 1.75 ($M_u/V_u d_v$)][1]	4.00	3.83	3.65	3.48	3.30	3.13	2.95	2.78	2.60	2.43	2.25
f'_m (psi)	Maximum Nominal Shear Stress Provided by the Masonry, v_m, psi[2]										
1500	155	148	141	135	128	121	114	107	101	94	87
2000	179	171	163	155	148	140	132	124	116	108	101
2500	200	191	183	174	165	156	148	139	130	121	113
3000	219	210	200	190	181	171	162	152	142	133	123
3500	237	226	216	206	195	185	175	164	154	143	133
4000	253	242	231	220	209	198	187	176	164	153	142

1. Based on MSJC Code Eq. 3-21.
2. Value equal to $\left[4.0 - 1.75\left(M_u/V_u d_v\right)\right]\sqrt{f'_m}$. To use in MSJC Code Eq. 3-21, multiply by A_n and add the result to $0.25P_u$.

Diagram SD-26 Maximum Nominal Shear Stress Provided by the Masonry, V_m, psi

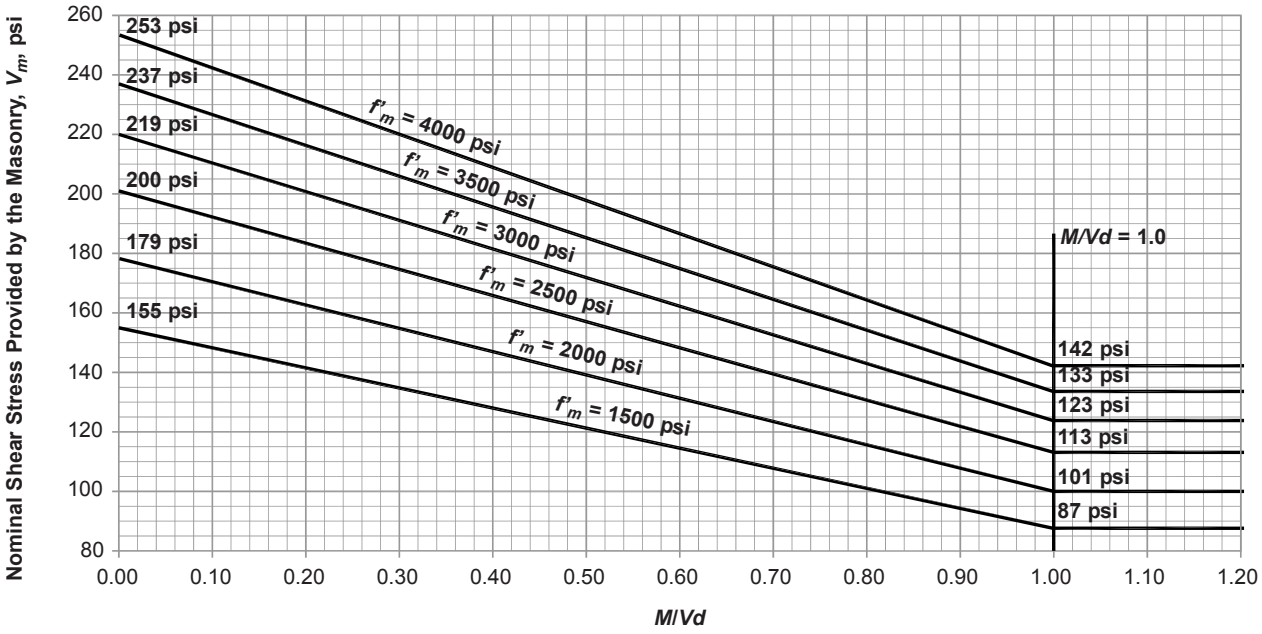


Table SD-27 Maximum Nominal Shear Stress of Masonry and Reinforcement, V_n, psi

$M_u/V_u d_v$	0.00	0.10	0.20	0.25	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
Coefficient[1] $\frac{V_n}{A_n\sqrt{f'_m}}$	6.00	6.00	6.00	6.00	5.87	5.60	5.33	5.07	4.80	4.53	4.27	4.00
f'_m (psi)	**Maximum Nominal Shear Stress Values, v_n, psi[2]**											
1500	232	232	232	232	227	217	207	196	186	176	165	155
2000	268	268	268	268	262	250	239	227	215	203	191	179
2500	300	300	300	300	293	280	267	253	240	227	213	200
3000	329	329	329	329	321	307	292	278	263	248	234	219
3500	355	355	355	355	347	331	316	300	284	268	252	237
4000	379	379	379	379	371	354	337	320	304	287	270	253

1. Based on MSJC Code Equation 3-19 and 3-20.
2. Value equal to Coefficient times $\sqrt{f'_m}$. Maximum Shear Strength Value, V_n, can thus be determined by multiplying values by A_n.

Diagram SD-27 Maximum Nominal Shear Stress of Masonry and Reinforcing Steel, V_n, psi

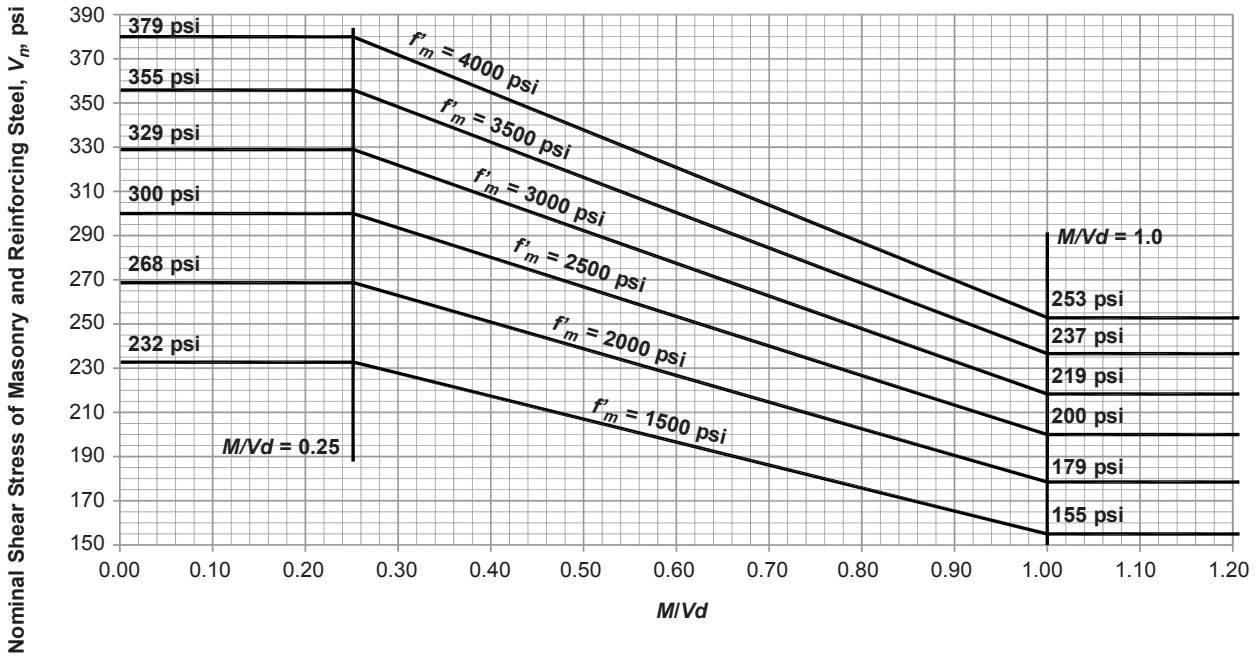
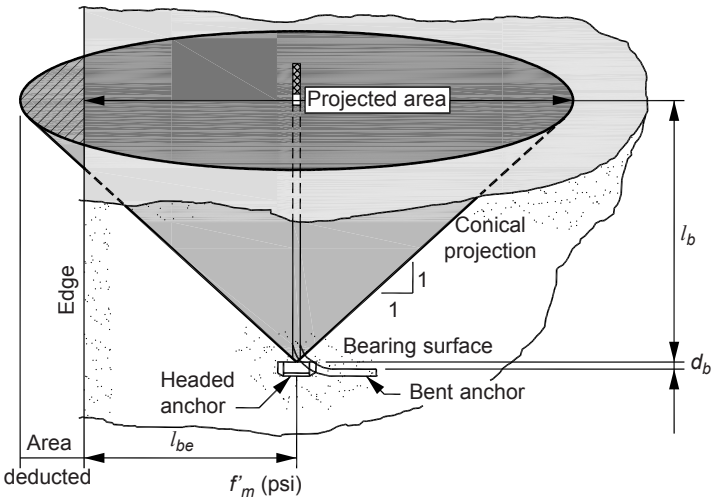


Table SD-91 Nominal Axial Tensile Strength B_{an} (pounds) in Anchor Bolts Based on l_b or l_{be}[1]

$B_{an} = 4A_{pt}\sqrt{f'_m}$

$A_{pt} = \pi\, l_b^2$



Embedment Length[2], l_b (in.)	Edge Distance, l_{be} (in.)	Projected Area, A_{pt} (sq in.)	1000	1500	2000	2500	3000	3500	4000	4500	5000	5500	6000
2	2	12.566	1,590	1,947	2,248	2,513	2,753	2,974	3,179	3,372	3,554	3,728	3,894
3	3	28.274	3,576	4,380	5,058	5,655	6,195	6,691	7,153	7,587	7,997	8,388	8,760
3	2	25.177	3,185	3,900	4,504	5,035	5,516	5,958	6,369	6,756	7,121	7,469	7,801
4	4	50.265	6,358	7,787	8,992	10,053	11,013	11,895	12,716	13,488	14,217	14,911	15,574
4	3	46.639	5,899	7,225	8,343	9,328	10,218	11,037	11,799	12,515	13,191	13,835	14,451
4	2	40.439	5,115	6,265	7,234	8,088	8,860	9,570	10,230	10,851	11,438	11,996	12,529
5	5	78.54	9,935	12,167	14,050	15,708	17,207	18,586	19,869	21,074	22,214	23,299	24,335
5	4	74.452	9,418	11,534	13,318	14,890	16,312	17,619	18,835	19,978	21,058	22,086	23,068
5	3	67.357	8,520	10,435	12,049	13,471	14,757	15,940	17,040	18,074	19,052	19,981	20,870
5	2	58.723	7,428	9,097	10,505	11,745	12,866	13,896	14,856	15,757	16,609	17,420	18,195
6	6	113.1	14,306	17,521	20,231	22,619	24,778	26,764	28,612	30,347	31,989	33,550	35,042
6	5	108.6	13,736	16,824	19,426	21,719	23,792	25,698	27,473	29,139	30,716	32,215	33,647
6	4	100.71	12,739	15,602	18,015	20,141	22,064	23,832	25,477	27,023	28,484	29,875	31,203
6	3	90.987	11,509	14,096	16,276	18,197	19,934	21,531	23,018	24,414	25,735	26,991	28,191
6	2	80.097	10,131	12,408	14,328	16,019	17,548	18,954	20,263	21,492	22,655	23,760	24,817
7	7	153.94	19,472	23,848	27,537	30,788	33,726	36,428	38,944	41,306	43,540	45,665	47,696
7	6	149.06	18,854	23,092	26,664	29,811	32,657	35,273	37,709	39,996	42,160	44,218	46,184
7	5	140.45	17,765	21,758	25,124	28,090	30,771	33,236	35,531	37,686	39,725	41,664	43,516
7	4	129.75	16,412	20,101	23,211	25,950	28,427	30,705	32,825	34,816	36,699	38,490	40,202
7	3	117.65	14,881	18,226	21,045	23,529	25,775	27,840	29,762	31,568	33,275	34,899	36,451
7	2	104.58	13,229	16,202	18,708	20,917	22,913	24,749	26,458	28,063	29,581	31,024	32,404
8	8	201.06	25,433	31,148	35,967	40,212	44,050	47,580	50,865	53,951	56,869	59,645	62,297
8	7	195.83	24,771	30,338	35,031	39,166	42,904	46,342	49,541	52,547	55,389	58,092	60,676
8	6	186.56	23,598	28,901	33,372	37,311	40,872	44,147	47,195	50,058	52,766	55,341	57,802
8	5	174.96	22,131	27,105	31,299	34,993	38,333	41,404	44,263	46,948	49,487	51,903	54,211
8	4	161.75	20,460	25,059	28,935	32,351	35,439	38,278	40,921	43,403	45,751	47,984	50,118
8	3	147.38	18,642	22,832	26,364	29,476	32,290	34,877	37,285	39,546	41,686	43,720	45,664
8	2	132.19	16,721	20,479	23,648	26,439	28,962	31,283	33,443	35,471	37,390	39,215	40,959
9	9	254.47	32,188	39,422	45,521	50,894	55,751	60,218	64,376	68,281	71,975	75,488	78,844
9	8	248.91	31,485	38,561	44,526	49,781	54,533	58,902	62,969	66,789	70,402	73,838	77,121
9	7	239.01	30,233	37,028	42,756	47,803	52,365	56,561	60,466	64,134	67,603	70,903	74,056
9	6	226.59	28,662	35,103	40,534	45,318	49,644	53,621	57,324	60,801	64,090	67,218	70,207
9	5	212.36	26,862	32,899	37,989	42,473	46,526	50,254	53,724	56,983	60,065	62,997	65,798
9	4	196.79	24,892	30,486	35,203	39,358	43,114	46,569	49,784	52,804	55,660	58,377	60,973
9	3	180.22	22,796	27,919	32,238	36,043	39,484	42,647	45,592	48,357	50,973	53,461	55,838
9	2	162.94	20,610	25,242	29,147	32,587	35,697	38,558	41,220	43,720	46,085	48,335	50,484
10	10	314.16	39,738	48,669	56,199	62,832	68,829	74,344	79,477	84,298	88,858	93,195	97,339
10	9	308.29	38,996	47,760	55,148	61,657	67,542	72,954	77,991	82,722	87,197	91,453	95,519
10	8	297.81	37,670	46,136	53,274	59,562	65,247	70,475	75,340	79,911	84,233	88,344	92,273
10	7	284.61	36,001	44,091	50,912	56,922	62,355	67,351	72,001	76,369	80,500	84,429	88,183
10	6	269.43	34,080	41,740	48,197	53,886	59,029	63,759	68,161	72,296	76,206	79,926	83,480
10	5	252.74	31,969	39,154	45,212	50,548	55,373	59,809	63,939	67,817	71,486	74,975	78,309
10	4	234.89	29,712	36,389	42,019	46,978	51,462	55,586	59,423	63,028	66,437	69,680	72,779
10	3	216.17	27,343	33,488	38,669	43,233	47,360	51,154	54,686	58,004	61,141	64,126	66,977
10	2	196.81	24,895	30,490	35,207	39,362	43,119	46,574	49,790	52,810	55,667	58,384	60,980

1. Projected area A_{pt} and Nominal Axial Tensile Strength B_{an} have been reduced by the projected area extending beyond a single edge where $l_{be} < l_b$.
2. The minimum effective embedment length required for placement of headed and bent bar anchor bolts shall be the greater of 2 in. or 4 bolt diameters.

Table SD-92 Nominal Axial Tensile Strength B_{an} (pounds) Based on ASTM A307 Grade A Steel Bolts

Diameter (in.)	1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4
Area, A_b (sq in.)	0.0491	0.1104	0.1963	0.3068	0.4418	0.6013	0.7854	0.9940	1.2272
Strength[1], $B_{an} = A_b f_y$ (pounds)	1,767	3,976	7,069	11,045	15,904	21,648	28,274	35,785	44,179

1. Strength Values based on MSJC Code Eqs. 3-2 and 3-5, in MSJC Code Sections 3.1.6.1 and 3.1.6.2.

Table SD-93 Anchor Bolt Shear Strength ϕB_{vn} (pounds) Based on Bolt Steel Strength and Masonry Breakout Strength[1,2,3,4]

Bolt Steel Strength
ϕ = 0.90 (bolt steel strength)
$B_{vn} = 0.6 A_b f_y$

Masonry Breakout
ϕ = 0.50
$B_{vn} = 4 A_{pv} \sqrt{f'_m}$
$A_{pv} = \frac{\pi l_{be}^2}{2}$

Anchor Bolt Diameter (in.)	Anchor Bolt Area A_b (sq in.)	Anchor Bolt Steel Shear Strength, ϕB_{vn} (pounds)	Edge Distance, l_{be} (in.)	Projected Area, A_{pv} (sq in.)	Anchor Bolt Masonry Breakout Strength, ϕB_{vn} f'_m (psi)										
					1000	1500	2000	2500	3000	3500	4000	4500	5000	5500	6000
			1.5	3.53	224	274	316	353	387	418	447	474	500	524	548
			2	6.28	397	487	562	628	688	743	795	843	889	932	973
			2.5	9.82	621	760	878	982	1,075	1,162	1,242	1,317	1,388	1,456	1,521
1/4	0.0491	954	3	14.14	894	1,095	1,264	1,414	1,549	1,673	1,788	1,897	1,999	2,097	2,190
			3.5	19.24	1,217	1,490	1,721	1,924	2,108	2,277	2,434	2,582	2,721	2,854	2,981
			4	25.13	1,590	1,947	2,248	2,513	2,753	2,974	3,179	3,372	3,554	3,728	3,894
3/8	0.1104	2,147	4.5	31.81	2,012	2,464	2,845	3,181	3,484	3,764	4,024	4,268	4,498	4,718	4,928
			5	39.27	2,484	3,042	3,512	3,927	4,302	4,646	4,967	5,269	5,554	5,825	6,084
			5.5	47.52	3,005	3,681	4,250	4,752	5,205	5,622	6,010	6,375	6,720	7,048	7,361
1/2	0.1963	3,817	6	56.55	3,576	4,380	5,058	5,655	6,195	6,691	7,153	7,587	7,997	8,388	8,760
			6.5	66.37	4,197	5,141	5,936	6,637	7,270	7,853	8,395	8,904	9,386	9,844	10,281
			7	76.97	4,868	5,962	6,884	7,697	8,432	9,107	9,736	10,326	10,885	11,416	11,924
5/8	0.3068	5,964	7.5	88.36	5,588	6,844	7,903	8,836	9,679	10,455	11,176	11,854	12,496	13,106	13,688
			8	100.53	6,358	7,787	8,992	10,053	11,013	11,895	12,716	13,488	14,217	14,911	15,574
			8.5	113.49	7,178	8,791	10,151	11,349	12,432	13,428	14,355	15,226	16,050	16,833	17,582
3/4	0.4418	8,588	9	127.23	8,047	9,856	11,380	12,723	13,938	15,055	16,094	17,070	17,994	18,872	19,711
			9.5	141.76	8,966	10,981	12,680	14,176	15,530	16,774	17,932	19,020	20,049	21,027	21,962
			10	157.08	9,935	12,167	14,050	15,708	17,207	18,586	19,869	21,074	22,214	23,299	
7/8	0.6013	11,690	10.5	173.18	10,953	13,414	15,490	17,318	18,971	20,491	21,906	23,235			
			11	190.07	12,021	14,722	17,000	19,007	20,821	22,489					
			11.5	207.74	13,138	16,091	18,581	20,774	22,757						
			12	226.19	14,306	17,521	20,231	22,619							
1	0.7854	15,268	12.5	245.44	15,523	19,011	21,953								
			13	265.46	16,789	20,563	23,744								
1 1/8	0.994	19,324	13.5	286.28	18,106	22,175									
			14	307.88	19,472	23,848									
			14.5	330.26	20,887										
1 1/4	1.2272	23,856	15	353.43	22,353										

1. Based on MSJC Code Sections 3.1.4.4 and 3.1.6.3, and MSJC Code Eqs. 3-8, 3-9 and 3-10.
2. Based on ASTM A307 Grade A Anchor Bolts.
3. Projected area A_{pv} and masonry breakout controls for the anchor bolt shear strengths ϕB_{vn} provided on the right hand side of the table, for edge distances above the solid line associated with a particular anchor bolt diameter. Anchor bolt strength controls below the solid lines associated with a particular anchor bolt diameter. The anchor bolt strength associated with a solid line is the anchor bolt with a diameter listed on the left hand side of the table just above the solid line.
4. Based on masonry strength f'_m and edge distance in the direction of the shear force, the masonry breakout limiting force can be determined as well as the most efficient bolt diameter available to resist the lateral shear load.

CHAPTER 15

REFERENCES

GENERAL

American Concrete Institute; Structural Engineering Institute of the American Society of Civil Engineers and The Masonry Society. (2005). *Building Code Requirements for Concrete Masonry Structures (ACI 530-05/ASCE 5-05/TMS 402-05)* and *Specifications for Masonry Structures (ACI 530.1-05/ASCE 6-05/TMS 602-05)*. Farmington Hills, MI: ACI; Reston, VA: SEI/ASCE; and Boulder, CO: TMS.

Amrhein, J.E. (1998). *Reinforced Masonry Engineering Handbook*, 5th Edition. Los Angeles, Masonry Institute of America.

ASTM International (2004). *ASTM International's Masonry Standards for the Building Industry*, 5th Edition. Philadelphia: ASTM.

Brick Industry Association. (July 2002). "Overview of Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02) and Specification for Masonry Structurces (ACI 530.1-02/ASCE 6-02/TMS 602-02)" *Technical Notes on Brick Construction*. No. 3. Reston: BIA.

Chrysler, et al. (2007). *Masonry Design Manual*, 4th Edition. Los Angeles: Masonry Institute of America.

International Code Council. (2006). *2006 International Building Code*. Washington, DC: ICC.

International Conference of Building Officials. (1997). *1997 Uniform Building Code*. Whittier: ICBO

International Conference of Building Officials. (1997). *Uniform Building Code Standards, 1997*. Whittier: ICBO

Structural Engineering Institute of the American Society of Civil Engineers. (2005). *American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures*.

SECTION 1 MATERIALS

Amrhein, J.E. (June 1977). "Grout...The Third Ingredient." Masonry Industry Magazine, pp. 9-14.

Brick Industry Association. (1989). Principles of Brick Masonry. Reston: BIA.

Brick Industry Association. (December 2006). "Manufacturing of Brick." Technical Notes on Brick Construction. No. 9.

Brick Industry Association. (October 2007). "Specification for and Classification of Brick." Technical Notes on Brick Construction. No. 9A.

Brick Industry Association. (December 2003). "Selection of Brick, Classification." Technical Notes on Brick Construction. No. 9B.

Building News, Inc. (1981). Concrete Masonry Design Manual, 4th Edition. Los Angeles: Building News, Inc.

Chrysler, J. (2000). Reinforced Grouted Brick Masonry, 14th Edition. Los Angeles: Masonry Institute of America.

Matthys, J. (1990). "Concrete Masonry Flexural Bond Strength Prisms vs Wall Tests." *5th North American Masonry Conference,* Vol. 2. Urbana-Champaign: University of Illinois, pp. 677.

National Concrete Masonry Association. (1999). "Building Code Requirements For Masonry Structures." NCMA TEK Notes. No. 1-3B. Herndon.

NCMA Engineered Concrete Masonry Design Committee. (August, 1988). *Research Investigation of the Properties of Masonry Grout in Concrete Masonry.* National Concrete Masonry Association.

Panarese, W.; Kosmatka, S.H.: Randall, F. (1991). Concrete Masonry Handbook. Skokie: Portland Cement Association.

Qui-Gu, Hu. (1987). "Quality Requirements & Control of Masonry Materials." *4th North American Masonry Conference*. Los Angeles: University of California, pp. 12.

Senbu, O.; Abe, M.; Matsushima, Y.; Baba, A.; Sugiyama, M. (1991). "Effect of Admixtures on Compactibility and Properties of Grout." 9th International Brick/Block Masonry Conference, Vol. 1. Berlin: pp. 109.

SECTION 2 MASONRY ASSEMBLIES

Assis, G.; Hamid, A.; Harris, H.G. (1990). "Compressive Behavior of Block Masonry Prisms Under Strain Gradient." *5th North American Masonry Conference,* Vol. 1. Urbana-Champaign: University of Illinois, pp. 615.

Atkinson, R.H. (September 15, 1991). *Development of a Database for Compressive Stress-Strain Behavior of Masonry.* Boulder: Atkinson-Noland & Associates, Inc. Final Report.

Atkinson, R.H. (November 1990). *Evaluation of Strength and Modulus Tables for Grouted and Ungrouted Hollow Unit Masonry*. Boulder: Atkinson-Noland & Associates, Inc.

Baba, A. & Senbu, O. (1986). "Influencing Factors on Prism Strength of Grouted Masonry and Fracture Mechanism Under Uniaxial Loading." *4th Canadian Masonry Symposium,* Vol 2. Fredericton, NB: University of New Brunswick, pp. 1081.

Baba, A. & Senbu, O. (1986). "Mechanical Properties of Masonry Components." *4th Canadian Masonry Symposium,* Vol 2. Fredericton, NB: University of New Brunswick, pp. 1066.

Becica, I.J. & Harris, H.G. (1982). "Ultimate Strength Behavior of Hollow Concrete Masonry Prisms Under Axial Load and Bending." *2nd North American Masonry Conference.* College Park: University of Maryland. paper #3.

Bexten, Karen A.; Tadros, Maher K.; Horton, Richard T. (1989). "Compression Strength of Masonry." 5th Canadian Masonry Symposium, Vol. 2. Vancouver, BC: University of British Columbia. pp. 629.

Brown, R. (1975). Prediction of Brick Masonry Prism Strength from Reduced Constraint Brick Tests, ASTM STP589.

California Concrete Masonry Technical Committee. (1975). Recommended Testing Procedures for Concrete Masonry Units, Prisms. Grout and Mortar, Los Angeles: CCMTC.

Colville, J. & Wolde-Tinsae A. (1990). "Compressive Strength of Hollow Concrete Masonry." *5th North American Masonry Conference,* Vol. 2. Urbana-Champaign: University of Illinois, pp. 663.

Drysdale, R. & Hamid, A.A. (1982). "Influence of the Unit Strength of Block Masonry." *2nd North American Masonry Conference*. College Park: University of Maryland. paper #2.

Fishburn, C.C. (1961). Effect of Mortar Properties on Strength of Masonry, Washington DC: National Bureau of Standards Monograph 36.

Ghosh, S. & Neis, V. (1990). "A Photoelastic Examination of Stress-Strain Behavior of Grouted Concrete Block Prisms." *5th North American Masonry Conference,* Vol. 2. Urbana-Champaign: University of Illinois, pp. 627.

Hamid, A.A.; Assis, G.F.; Harris H.G. (1987). "Compression Behavior of Grouted Concrete Block Masonry - Some Preliminary Results." *4th North American Masonry Conference,* Vol. 2. Los Angeles: University of California. paper #43.

Hamid, A.A.; Ziab, G.: ElNawawy, O. (1987). "Modulus of Elasticity of Concrete Block Masonry." *4th North American Masonry Conference,* Vol. 1. Los Angeles: University of California. paper #7.

Hamid, A.A.: Drysdale, R.G.; Heidebrecht, A.C. (August 1978). "Effect of Grouting on the Strength Characteristics of Concrete Block Masonry." Proceedings of the North American Masonry Conference. Boulder: The Masonry Society. paper #11.

Hendry, A.W. (1987). "Testing Methods in Masonry Engineering." *4th North American Masonry Conference,* Vol. 2. Los Angeles: University of California. paper #49.

Holm, T.A. (August 1978). "Structural Properties of Block Concrete." Proceedings of the North American Masonry Conference. Boulder: The Masonry Society. paper #5.

Khalaf, F.; Handry, A.; Fairbairn, D. (1990). "Concrete Block Masonry Prisms Compressed Normal & Parallel to Bed Face." *5th North American Masonry Conference,* Vol. 1. Urbana-Champaign: University of Illinois, pp. 595.

Kingsley, G.R.; Atkinson, R.H.; Noland, J.R.; Hart, G.C. (1989). "The Effect of Height on Stress-Strain Measurements on Grouted Masonry Prisms." 5th Canadian Masonry Symposium, Vol. 2. Vancouver, BC: University of British Columbia, pp. 587.

Lenczner, D.; Foster, D. (1979). "Strength and Deformation of Brickwork Prisms in Three Directions." *5th International Brick Masonry Conference, Session 2.* Washington DC: B.I.A. paper #4.

Maurenbrecher, A.H.P. (1983). "Compressive Strength of Eccentrically Loaded Masonry Prisms." *3rd Canadian Masonry Symposium.* Edmonton: University of Alberta, pp. 10.

Maurenbrecher, A.H.P. (1986). "Compressive Strength of Hollow Concrete Blockwork." 4*th Canadian Masonry Symposium.* Vol. 2. Fredericton, NB: University of New Brunswick, pp. 1000.

Maurenbrecher, A.H.P. (1980). "The Effect of Test Procedures on the Compressive Strength of Masonry Prisms." *2nd Canadian Masonry Symposium.* Ottawa: Carleton University, pp. 119.

McAskill, N. & Morgan, D.R. (1983). "Inspection and Testing of Reinforced Masonry." *3rd Canadian Masonry Symposium.* Edmonton: University of Alberta, pp. 26.

Miller, D.; Noland, J.; Feng, C. (1979). "Factors Influencing the Compressive Strength of Hollow Clay Unit Prisms." *5th International Brick Masonry Conference, Session 2.* Washington DC: B.I.A., paper #15.

National Concrete Masonry Association. (2001). "Prism Testing and the Unit Strength Method for Evaluating the Compressive Strength of Concrete Masonry." NCMA TEK Notes. No. 18-1A. Herndon.

National Concrete Masonry Association. (1997). "Compressive Strength Testing Variables for Concrete Masonry Units." NCMA TEK Notes. No. 18-7. Herndon.

National Concrete Masonry Association. (1997). "Structural Testing of Concrete Masonry Assemblages." NCMA TEK Notes. No. 18-6. Herndon.

Page, A.W.; Kleeman, P.W. (1991). "The Influence of Capping Material and Platen Restraint of the Failure of Hollow Masonry Units and Prisms." 9th International Brick/Block Masonry Conference, Vol. 1. Berlin: pp. 662.

Redmond, T.; Allen, M. (1975). Compressive Strength of Composite Brick and Concrete Masonry Walls, ASTM STP589.

Sahlin, Sven. (1971). Structural Masonry. Englewood: Prentis-Hall, Inc.

Schubert, P. (1979). "Modulus of Elasticity of Masonry." *5th International Brick Masonry Conference, Session 2.* Washington DC: B.I.A. paper 17.

U.S. Department of Commerce. (September 1977). "Earthquake Resistant Masonry Construction." National Workshop. National Bureau of Standards Building Science Series 106.

Yao, Chichao; Nathan, N.D. (1989). "Axial Capacity of Grouted Concrete Masonry." 5th Canadian Masonry Symposium, Vol. 1. Vancouver, BC: University of British Columbia, pp. 45.

Yao, Chichao. (1986). "Joint Effect on Fully Bedded Plain Concrete Masonry." *5th Canadian Masonry Symposium, Vol. 1*. Vancouver, BC: University of British Columbia, pp. 55

SECTION 3 LOADS

American National Standards Institute. (1982). Minimum Design Loads for Buildings and Other Structures. New York: ANSI.

American Society of Civil Engineers. (1970). Lateral Stresses in the Ground and Design of Earth Retaining Structures. New York: Speciality Proceedings Conference.

Applied Technology Council. (1979). Seismic Design Guidelines for Highway Bridges. ATC-306, Palo Alto: ATC.

Blume; Corning; Newmark. (1961). Design of Multistory Reinforced Concrete Buildings for Earthquake Motions. Skokie: Portland Cement Association.

Dimarogons, P.D. (December 1983). “Distribution of Lateral Earthquake Pressure on a Retaining Wall.” Soils and Foundations (Japanese Society of Soil Mechanics). Vol. 23, No. 4.

Los Angeles City. (2002). Los Angeles City 2002 Building Code, Los Angeles.

Mononobe, N. (1929). “Earthquake-Proof Construction of Masonry Dams,” Proceedings, World Engineering Conference, Vol. 9.

Okbe, S. (1926). “General Theory of Earth Pressure,” Journal, Japanese Society of Civil Engineers, Vol. 12.

Seed, Bolton H. & Whitman, Robert V. (1970). Design of Earth Retaining Structures for Dynamic Loads. New York: ASCE.

Structural Engineers Association of California. (1988). Recommended Lateral force Requirements and Commentary. Sacramento: SEAOC.

Terry, Phillip. (March-April, 1991). “Reviewing the Seismic Provisions of the 1988 Uniform Building Code.” Building Standards.

Terzaghi, & Peck. (1948). Soil Mechanics in Engineering Practice. New York: John Wiley & Sons, Inc.

Virdee, A. S. (1966). Soil Pressures on Structures Due to Backfill Under Seismic Conditions. Sacramento: Department of Water Resources, State of California.

SECTION 4 DISTRIBUTION OF LOADS

Ambrose, James, & Vergun. (1987). Design for Lateral Forces. New York: John Wiley & Sons.

Blakeley, R.W.G. (June, 1979). “Recommendations for the Design and Construction of Base Isolated Structures.” Bulletin of the New Zealand National Society for Earthquake Engineering. Vol. 12, No. 2.

Buckle, Ian. (June, 1988). Basic Principles, Real World Case Studies, Evaluation of SEAOC Provisions. Los Angeles: SEAOC Seminar Notes.

Dodds, Steve. (June 1986). "Effect of Edge Fasteners on the Seismic Resistance of Composite Floor Diaphragms", a MS Thesis. Iowa State University.

Easterling, Samuel. (December 1987). "Analysis and Design of Steel-Deck-Reinforced Concrete Diaphragms", a PhD dissertation. Iowa State University.

Easterling, W. Samuel, Porter, Max L. (February 1994) "Steel-Deck-Reinforced Concrete Diaphragms: Part I", Journal of the Structural Engineering of the American Society of Civil Engineers, New York, NY, Vol. 120, No. 2.

Easterling, W. Samuel, Porter, Max L. (February 1994) "Steel-Deck-Reinforced Concrete Diaphragms: Part II", Journal of the Structural Engineering of the American Society of Civil Engineers, New York, NY, Vol. 120, No. 2.

Easterling, W. S. and Porter, M. L. (April, 1986). "Hysteretic Behavior of Composite Slabs", Dynamic Response of Structures, edited by Gary Hart and Richard Nelson, Proceedings of Third Engineering Mechanics Conference, American Society of Civil Engineers, New York, NY. pp. 155-161.

Khouri, Roger. (August 1990). "Effect of Connection and Supporting Elements on Cyclic Resistance of Precast Hollow-Core Plank Diaphragms", a MS Thesis. Iowa State University.

Mayes, R.L.; Weissberg, S.M.; Jones, L.R.; & Van Volkinburg. (Spring, 1991). Seismic Isolation: Enhancing the Earthquake Resistance of Masonry. Herndon: Council for Masonry Research Report. Vol. 4, No. 1.

Merryman, K.M.; Leiva, G.; Antrobus, N.; Klingner, R.E. (May, 1990). “In-Plane Seismic Resistance of Two-Story Concrete Masonry Coupled Shear Walls.” *U.S.-Japan Coordinated Program for Masonry Building Research,* Report No. 3.1(c)-1. Austin: The University of Texas.

Meyer, Ron. (April 1988). "Effect of Plank Depth Parameter on Seismic Resistance of Precast Hollow-Core Plank Diaphragms", a MS Thesis. Iowa State University.

Nielson, Mike. (November 1984). "Effects of Gravity Load on Composite Floor Diaphragm Behavior", a MS Thesis. Iowa State University.

Porter, M. L. (June, 1995) "NSF/ISU Diaphragm Floor Slab Results", First National Conference and Workshop on Research Transformed Into Practice: Implementation of NSF Research Proceedings, University of Maryland, College Park, MD.

Porter, Max L. (July, 1994). "Diaphragms in Buildings", Proceedings Fifth U.S. National Conference on Earthquake Engineering.

Porter, M. L. and Sabri, A. (June 1990) "Diaphragm Floor Research for Masonry Buildings", Proceedings of Fifth North American Masonry Conference, University of Illinois at Urbana-Champaign.

Porter, M. L. and Easterling, W.S. (August 1986). "Results of Full-Scale Tests of Steel-Deck Reinforced Concrete Floor Diaphragms", Proceedings of Third U.S. National Conference on Earthquake Engineering, Charleston, SC.

Porter, M. L., Sabri, A. A. (August 1990). "Hollow-Core Plank Diaphragms in Masonry Buildings", Proceedings of Sixth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, Seattle, WA.

Porter, M. L., Yeomans, F. S. (August 1990). "A Hysteretic Model for Hollow-Core Plank Diaphragms", Proceedings of Sixth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research, Seattle, WA.

Porter, M. L. and Sabri, A. A. (October 1988). "Diaphragm Floor Slabs for TCCMAR Study", Proceedings of Fourth Meeting of the Joint Technical Coordinating Committee on Masonry Research of the U.S.-Japan Joint Coordinated Earthquake Research Program, San Diego, CA.

Porter, M. L. and Easterling, W.S. (August, 1987). "Proposed Design Techniques for Composite Diaphragm Slabs", Proceedings ASCE Structures Congress, Orlando, FL.

Porter, M. L., Ekberg, C. E., R. Meyer and Tremel, P. (October 1987). "Diaphragm Floor Slabs for TCCMAR Study", Proceedings of Third Meeting of the Joint Technical Coordinating Committee on Masonry Research of the U.S. - Japan Coordinated Earthquake Research Program, Sapporo, Japan.

Porter, M. L. (September 1986) "Sequential Phased Displacement Loading for TCCMAR", paper distributed at the Technical Coordinating Group of U.S. Researchers Meeting, Keystone, CO, Revised and redistributed.

Porter, M. L. (September 1986) "Diaphragm Floor Slabs for TCCMAR Study, Proceedings of the Second Meeting of the Joint Technical Coordinating Committee on Masonry Research with U.S. and Japanese researchers, Keystone, CO.

Porter, M. L. and Greimann, L. F. (July 1984). "Composite Floor Diaphragm Slab Tests", Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, CA.

Porter, M. L. and Greimann, L. F. (1981). "Test Facility for Floor Diaphragms", Proceedings of the 27th International Instrumentation Symposium, Indianapolis, Ind., Vol. 27, Part 1, Instrument Society of America, Res. Triangle Park, NC.

Porter, M. L. and Greimann, L. F. (May 1980). "Composite Floor Diaphragms", Proceedings Sixth National Meeting Universities Council for Earthquake Engineering Research, University of Illinois, Urbana-Champaign.

Porter, M. L. and Greimann, L. F. (June 1978). "Earthquake Resistance of Composite Floor Diaphragms", Proceedings Fifth National Meeting Universities Council for Earthquake Engineering Research, Massachusetts Institute of Technology.

Porter, M. L. and Greimann, L. F. (June 1979). "Pilot Tests of Composite Floor Diaphragms", Proceedings of Third Canadian Conference on Earthquake Engineering, McGill University, Montreal, Canada. pp. 24.

Preistley, M.; Crosbie, R.; Carr, A. (June, 1977). “Seismic Forces in Base-Isolated Masonry Structures.” Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 10, No. 2.

Prins, Mike. (May 1985). "Elemental Tests for the Seismic Resistance of Composite Floor Diaphragms", a MS Thesis. Iowa State University.

Sabri, Aziz. (May 1990) "Analysis and Behavioral Characteristics of Hollow-Core Plank Diaphragms in Masonry Buildings", a PhD dissertation. Iowa State University.

Sabri, Aziz A., and Porter, Max L. (June, 1993) "Masonry Buildings with Plank Diaphragms", Proceedings Sixth North American Masonry Conference, Philadelphia, PA.

Shing, P.B.; Noland, J.L.; Klamerus, E.W.; Schuller, M.P. (January, 1991). “Response of Single-Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads.” *U.S.-Japan Coordinated Program for Masonry Building Research.* Report No. 3.1(a)-2. Boulder: University of Colorado.

Structural Engineers Association of California Notes. (June, 1988). Design and Construction of Base Isolated Buildings. Los Angeles: SEAOC Seminar.

Tremel, Paul. (April 1988). "Boundary Conditions and Orientation Behavioral Characteristics of Hollow-Core Diaphragms", a MS Thesis. Iowa State University.

Yeomons, Francisco. (May 1994). Seismic Modeling of Structures with Steel Deck Reinforced Concrete Diaphragms", a PhD dissertation. Iowa State University.

Yeomons, Francisco. (June 1990). "A Hysteretic Model for Precast Prestressed Hollow Core Plank Diaphragms", a MS Thesis. Iowa State University.

SECTION 5 STRUCTURAL DESIGN ASD

American Plywood Association. (1987). APA Design/Construction Guide, Diaphragms. Tacoma: APA.

Blume, J.A. (1968). Shear in Grouted Brick Masonry Wall Elements. San Francisco: Western States Clay Products Association.

Borchelt, G. (1990). “Friction at Supports of Clay Brick Walls.” *5th North American Masonry Conference,* Vol. 3. Urbana-Champaign: University of Illinois, pp. 1053.

Fried, A.N. (1991). “The Position of the Neutral Axis in Masonry Joints.” 9th International Brick/Block Masonry Conference, Vol. 1. Berlin: pp. 188.

Grimm, C. (1990). “Masonry Flexural Strength vs Course Height.” *5th North American Masonry Conference,* Vol. 2. Urbana-Champaign: University of Illinois, pp. 673.

Hamid, A.A.; Ghanem, G.M. (1991). “Partially Reinforced Concrete Masonry.” 9th International Brick/Block Masonry Conference, Vol. 1. Berlin: pp. 368.

Holmes, I.L. (1969). Masonry Building in High Intensity Seismic Zones; Designing Engineering and Constructing with Masonry Products. Houston: Gulf Publishing Co.

Hosny, A.H.; Essawy, A.S.; abou-Elenain, A.’ Higazy, E.M. (1991). “Behavior of Reinforced Block Masonry Walls Under Out-of-Plane Bending.” 9th International Brick/Block Masonry Conference, Vol. 1. Berlin: pp. 387.

Leet, Kenneth. (1982). Reinforced Concrete Design. New York: McGraw-Hill Book Co.

Limin, H. & Priestly, M.J.N. (May, 1988). “Seismic Behavior of Flanged Masonry Shear Walls.” *Structural Systems Research Project.* Report No. SSRP-88/01. La Jolla: University of California, San Diego.

Luttrell, Larry. (1987). Diaphragm Design Manual. Canton: Steel Deck Institute.

Mayes, R.L.; Clough, R.W. (1975). A Literature Survey - Compressive, Tensile, Bond, and Shear Strength of Masonry, EERC 75-21. Berkeley: University of California.

Mayes, R.L.; Clough, R.W. (1975). State-of-the-Art in Seismic Shear Strength of Masonry; An Evaluation and Review, EERC 75-21. Berkeley: University of California.

McGinley, W.M.; Borchlet, J.G. (1989) “Friction Between Brick and Its Support.” 5th Canadian Masonry Symposium, Vol 2. Vancouver, BC: University of British Columbia, pp. 713.

McGinley, W.M.; Borchelt, J.G. (1991). “ Influence of Materials on the Friction Developed at the Base of Clay Brick Walls.” 9th International Brick/Block Masonry Conference, Vol. 1. Berlin: pp. 292.

Modena, C. & Cecchinato, P. (1987). “Researches on the Interaction Mechanisms Between Steel Bars & Hollow Clay Unit Masonry.” *4th North American Masonry Conference.* Los Angeles: University of California, pp. 16.

Pfeffermann, I.O.; Van de Loock, I.G. (1991). “20 Years Experience with Bed Joint Reinforced Masonry in Belgium and Europe.” 9th International Brick/Block Masonry Conference, Vol. 1. Berlin: pp. 427.

Pfeir, I.M. (1987). “Analytical Investigations of Masonry Walls Subjected to Axial Compressive Forces & Bi-axial Bending Moments.” *4th North American Masonry Conference.* Los Angeles: University of California, pp. 13.

Schneider, R.R., Dickey, W.L. (1987). Reinforced Masonry Design, 2nd Edition. Englewood Cliffs: Prentiss Hall, Inc.

Scrivener, J.C. (July, 1986). “Bond Reinforcement in Grouted Hollow-Unit Masonry: A State-of-the-Art.” *U.S.-Japan Coordinated Program for Masonry Building Research.* Report No. 6.2.-1. Boulder: Atkinson-Noland & Associates, Inc.

Soric, Z.; Tulin, L.G. (1987). “Comparison Between Predicted & Observed Responses for Bond Stress and Relative Displacement in Reinforced Concrete Masonry.” *4th North American Masonry Conference.* Los Angeles: University of California, pp. 44.

Soric, Z.; Tulin, L.G. (August 1987). “Bond & Splices in Reinforced Masonry.” U.S. - Japan Coordinated Program for Masonry Building Research, Report No. 6.2-2. Boulder: University of Colorado.

Tawresey, J.G. (1987). “Walls with Axial Load Combined with Bending Moment - Interaction Equations for Masonry.” *4th North American Masonry Conference.* Los Angeles: University of California, pp. 34.

SECTION 6 STRENGTH DESIGN

ACI/SEAOSC Task Force Committee on Slender Walls. (1982). Test Report on Slender Walls. Los Angeles: SEAOSC and the Southern California Chapter of the American Concrete Institute.

Adham, S. & Amrhein, J.E. (1991). “Dynamic and Testing of Tall Slender Reinforced Masonry Walls.” 9th International Brick/Block Masonry Conference, Vol. 1. Berlin: pp. 465.

Agbabian, M.S.; Adham, S.A.; Masri, S.F.; Avanessian, V.; Traina, I. (July 1989). “Out-of-Plane Dynamic Testing of Concrete Masonry Walls.” Volume 1: Final Report; Volume 2: Test Results. *U.S.-Japan Coordinated Program for Masonry Building Research*, Report No. 3.2(b1). Los Angeles: The University of Southern California.

Amrhein, J.E. & Lee. (1986). Design of Reinforced Masonry Tall Slender Walls, 2nd Edition. San Francisco: Western States Clay Products Association.

Amrhein, J.E. & Lee. (1988). Slender Wall Design for Los Angeles, Estimating Curves. Los Angeles: Masonry Institute of America.

Amrhein, J.E. & Lee. (1985) Tall Slender Walls, Estimating Curves. Los Angeles: Masonry Institute of America.

Asher, J. & Selna, L. (1990). “Multistory Slender Wall Design.” *5th North American Masonry Conference,* Vol. 3. Urbana-Champaign: University of Illinois, pp. 915.

Atkinson, R.H. (June, 1991). “An Assessment of Current Material Test Standards for Masonry Limit States Design Methods.” *U.S.-Japan Coordinated Program for Masonry Building Research*, Report No. 1.3-1. Boulder: Atkinson-Noland & Associates, Inc.

Atkinson, R.H.; Noland, J.L.; Hart, G.C. (1991). “Properties of Masonry Materials for Limit States Design.” 9th International Brick/Block Masonry Conference, Vol. 2. Berlin: pp. 678.

Curtin, W.G.; Shaw, G.; Beek, J.K. (1988). Design of Reinforced and Prestressed Masonry. London: Thomas Telford LTD.

Essawy, A.S.; Drysdale, R.G. (1987). "Evaluation of Available Design Methods for Masonry Walls Subject to Out-of-Plane Loading." *4th North American Masonry Conference.* Los Angeles: University of California, pp.32.

Ferguson, P.M. (1973). Reinforced Concrete Fundamentals, 3rd Edition. New York: John Wiley and Sons.

Fling, R.S. (1987). Practical Design of Reinforced Concrete. New York: John Wiley and Sons.

Hart, G.C. (July-Dec. 1989). "Limit State Design Criteria for Minimum Flexural Steel in Concrete Masonry Beams," The Masonry Society Journal, Vol 8, No. 2. pp. 7-18.

Hart G.C.; Noland, J.L. (1991). "Expected Value Limit State Design Criteria for Structural Masonry." 9th International Brick/Block Masonry Conference, Vol. 2. Berlin: pp. 752.

Hart, G.C.; Englekirk, R.E.; Sabol, T.A. (July-Dec. 1986). "Limit State Design Criteria for One to Four Story Reinforced Concrete Masonry Buildings," The Masonry Society Journal, Vol. 5, No. 2. pp. 21-24.

Hart, G.C.; Noland, J.; Kingsley, G.; Englekirk, R.; Sajjad, N. (July-Dec. 1988). "The Use of Confinement Steel to Increase Ductility in Reinforced Concrete Masonry Sheer Walls." The Masonry Journal, Vol 7, No. 2. pp. 19-42.

Hart, G.C.; Bashartchah, M.A.; Zorapapel, G.T. (1987). "Limit State Design Criteria for Minimum Flexural Steel." *4th North American Masonry Conference.* Los Angeles: University of California, pp. 31.

Hart, G.C.; Noland, J.L.; Kingsley, G.R.; Englekirk, R.E. (1987). "Confinement Steel in Reinforced Block Masonry Walls." *4th North American Masonry Conference.* Los Angeles: University of California, pp. 52.

Hart, G.C. (1987). "Technology Transfer, Limit State Design & the Critical Need for a New Direction in Masonry Code." *4th North American Masonry Conference.* Los Angeles: University of California, pp. 41.

Heeringa, R.; McLean, D. (1990). "Ultimate Strength Behavior of Reinforced Concrete Block Walls." *5th North American Masonry Conference,* Vol. 3. Urbana-Champaign: University of Illinois, pp. 1041.

Heeringa, R.L., McLean, D.L. (July-December 1989). "Ultimate Strength Flexural Behavior of Concrete Masonry Walls," The Masonry Society Journal, Vol. 8, No. 2. pp. 19-30.

Hegemier, G.A. (1975). Mechanics of Reinforced Concrete Masonry, A Literature Survey, AMES-NSF TR 75-5. San Diego: University of California.

Hogan, Mark. (April, 1991). "Limit States Design Provisions." The Concrete Specifier.

Leet, Kenneth. (1982). Reinforced Concrete Design. New York: McGraw-Hill Book Co.

Masonry Society, The. (March, 1991). *Limit States Design of Masonry*. The Masonry Society.

Matsumura, A. (1987). "Shear Strength of Reinforced Hollow Unit Masonry Walls." *4th North American Masonry Conference.* Los Angeles: University of California, pp. 6.

Mayes, R.L.; Omote, Y.; Clugh, R.W. (1976). Cyclic Shear Testing of Masonry Piers, Vol. 1, Test Results. EERC-76-8. Berkeley: University of California.

Nakaki, D.K.; Hart, G.C. (1987). "A Proposed Seismic Design Approach for Masonry Shear Walls Incorporating Foundation Uplift." *4th North American Masonry Conference.* Los Angeles: University of California, pp. 25.

Paulay, T. (September 1972). "Some Aspects of Shear Wall Design." Bulletin of New Zealand Society for Earthquake Engineering, Vol. 5, No. 3.

Porter, M.L.; Wolde-Tinsae, A.M.; Ahmed, M.H. (1987). "Strength Design Method for Brick Composite Walls." *4th North American Masonry Conference.* Los Angeles: University of California, pp. 37.

Priestley, M.J.N. (July-Dec. 1986). "Flexural Strength of Rectangular Unconfined Masonry Shear Walls with Distributed Reinforcement." The Masonry Society Journal, Vol. 5, No. 2. pp. 1-15.

Priestly, M.J.N. (1987). Strength Design of Masonry. Los Angeles: Fourth North American Masonry Conference.

Selna, L.G. & Asher, J.W. (1986). "Multistory Slender Masonry Walls; Analysis, Design and Construction." Redondo Beach: Higgins Brick Co.

Shing, P.B.; Schuller, M.; Hoskere, V.S.; Carter, E. (Nov.-Dec. 1990). "Flexural and Shear Response of Reinforced Masonry Walls." ACI Journal: Paper No. 87-S66.

Structural Engineers Association of Southern California Seismology Committee of the SEAOC Strength Design. (1991). Masonry Moment Resisting Wall Frames. San Francisco: SEAOC.

Sveinsson, B.I.; Blondet, M.; Mayes, R.L. (December 1988). "The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes." *U.S.-Japan Coordinated Program for Masonry Building Research,* Report No. 3.2 (b2)-10. Berkeley: Computech Engineering Services, Inc.

Sveinsson, B.I.; Kelley, T.E.; Mayes, R.L.; Jones, L.R. (1987). "Out-of-Plane Response of Masonry Walls to Seismic Loads." *4th North American Masonry Conference.* Los Angeles: University of California, pp. 46.

Wang, C.K. & Salmon, C.J. (1985). Reinforced Concrete Design. New York: Harper & Rowe.

SECTION 7 REINFORCING STEEL

Amrhein, J.E. (1991). Reinforcing Steel in Masonry. Los Angeles: Masonry Institute of America.

Beall, Christine. (2004). Masonry Design and Detailing, 5th Edition. New York: McGraw-Hill Book Co.

Concrete Reinforcing Steel Institute. (1991). CRSI Handbook. Schaumburg: Concrete Reinforcing Steel Institute.

Newman, Morton. (1976). Standard Cantilever Retaining Walls. New York: McGraw-Hill Book Co.

Newman, Morton. (1968). Standard Structural Details for Building Construction. New York: McGraw-Hill Book Co.

Snell, L.M.; Rutledge, R.B. (1987). "Methodology for Accurately Determining the Location of Reinforcement within Masonry." *4th North American Masonry Conference.* Los Angeles: University of California, pp. 11.

Virdee, Ajit. (October 1988). Fundamentals of Reinforced Masonry Design. Citrus Heights: Concrete Masonry Association of California and Nevada.

SECTION 8 BUILDING DETAILS

Beall, Christine. (2004). Masonry Design and Detailing, 5th Edition. New York: McGraw-Hill Book Co.

Curtin, W.G.; Shaw, G.; Beck, J.K.; Parkinson, J.I. (1984). Structural Masonry Detailing. London: Granada Publishing.

Elmiger, A. (1976). Architectural and Engineering Concrete Masonry Details for Building Construction. McLean: National Concrete Masonry Association.

Newman, Morton. (1968). Standard Structural Details For Building Construction. New York: McGraw-Hill Book Co.

SECTION 9 SPECIAL TOPICS

Beall, Christine. (2004). Masonry Design and Detailing, 5th Edition. New York: McGraw-Hill Book Co.

Brick Industry Association. (December 2005). "Water Resistance of Brick Masonry, Design and Detailing." Technical Notes on Brick Construction, No. 7.

Brick Industry Association. (December 2005). "Water Penetration Resistance Materials." Technical Notes on Brick Construction, No. 7A.

Brick Industry Association. (December 1985). "Painting Brick Masonry." Technical Notes on Brick Construction, No. 6. Revised.

Brick Industry Association. (March 2008). "Fire Resistance of Masonry." Technical Notes on Brick Construction, No. 16.

Brick Industry Association. (August 1998). "Brick Masonry Cavity Walls." Technical Notes on Brick Construction, No. 21.

Brick Industry Association. (November 2006). "Accommodating Expansion of Brickwork." Technical Notes on Brick Construction, No. 18A.

Brick Industry Association. (October 2006). "Volume Changes – Analysis and Effects of Movement." Technical Notes on Brick Construction, No. 18.

Concrete Masonry Association of California and Nevada. (1986). Waterproofing Concrete Masonry, Citrus Heights: CMACN.

Concrete Masonry Association of California and Nevada. Fire Resistive Construction Using Concrete Masonry, Citrus Heights: CMACN.

Lauersdorf, Lyn R. (May 1988). "Stopping Rainwater Penetration." The Magazine of Masonry Construction, pp. 74-77.

Masonry Advancement Committee. Guidelines for Clear Waterproofing Masonry Walls, Los Angeles: MAC.

National Concrete Masonry Association. (2001). "Concrete Masonry Basement Wall Construction." NCMA TEK Notes No. 3-11. Herndon.

National Concrete Masonry Association. (2003). "Concrete Masonry Foundation Wall Details." NCMA TEK Notes, No. 5-3A. Herndon.

National Concrete Masonry Association. (2003). "Crack Control in Concrete Masonry Walls." NCMA TEK Notes, No. 10-1A. Herndon.

National Concrete Masonry Association. (1998). "Maintenance of Concrete Masonry Walls." NCMA TEK Notes, No. 8-1A. Herndon.

National Concrete Masonry Association. (2002). "Water Repellent Coatings for Concrete Masonry Walls." NCMA TEK Notes, No. 19-1. Herndon.

National Concrete Masonry Association. (2001). "Preventing Water Penetration in Below-Grade Concrete Masonry Walls." NCMA TEK Notes, No. 19-3A. Herndon.

National Concrete Masonry Association. (2001). "Concrete Basement Wall Construction." NCMA TEK Notes, No. 3-11. Herndon.

National Concrete Masonry Association. (2002). "Design for Dry Single-Wythe Concrete Masonry Walls." NCMA TEK Notes, No. 19-2A. Herndon.

National Concrete Masonry Association. (2001). "Increasing the Fire Resistance of Concrete Masonry." NCMA TEK Notes, No. 7-4. Herndon.

National Concrete Masonry Association. (2003). "Fire Resistance Rating of Concrete Masonry Assemblies." NCMA TEK Notes, No. 7-1A. Herndon.

National Concrete Masonry Association. (2003). "Balanced Design Fire Protection" NCMA TEK Notes, No. 7-2. Herndon.

Panarese, W.C.; Kosmatka, S.H.; Randall, Jr, F.A. (1991). Concrete Masonry Handbook, Skokie: Portland Cement Association.

Schaffler, M.; Chin, I.; Slaton, D. (1990). "Moisture Expansion of Fired Bricks." *5th North American Masonry Conference,* Vol. 2. Urbana-Champaign: University of Illinois, pp. 549.

Suprenant, Bruce. (March 1989). "Painting Concrete Masonry." The Magazine of Masonry Construction, pp. 100-103.

Suprenant, Bruce. (August 1989). "Repelling Water from the Inside." The Magazine of Masonry Construction, pp. 358-360.

Suprenant, Bruce. (April 1990). "Choosing a Water Repellent." The Magazine of Masonry Construction, pp 5-11.

SECTION 13 RETAINING WALLS

Bowles, Joseph E. (1977). Foundation Analysis & Design. New York: McGraw-Hill Book Co.

Das, Braja M. (1984). Principles of Foundation Engineering. Boston: PWS Engineering.

Newman, Morton. (1976). Standard Cantilever Retaining Walls. New York: McGraw-Hill Book Co.

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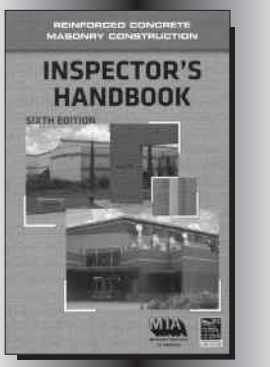

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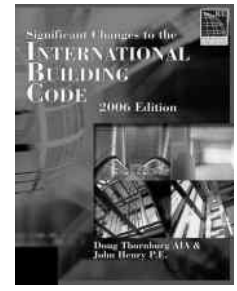

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