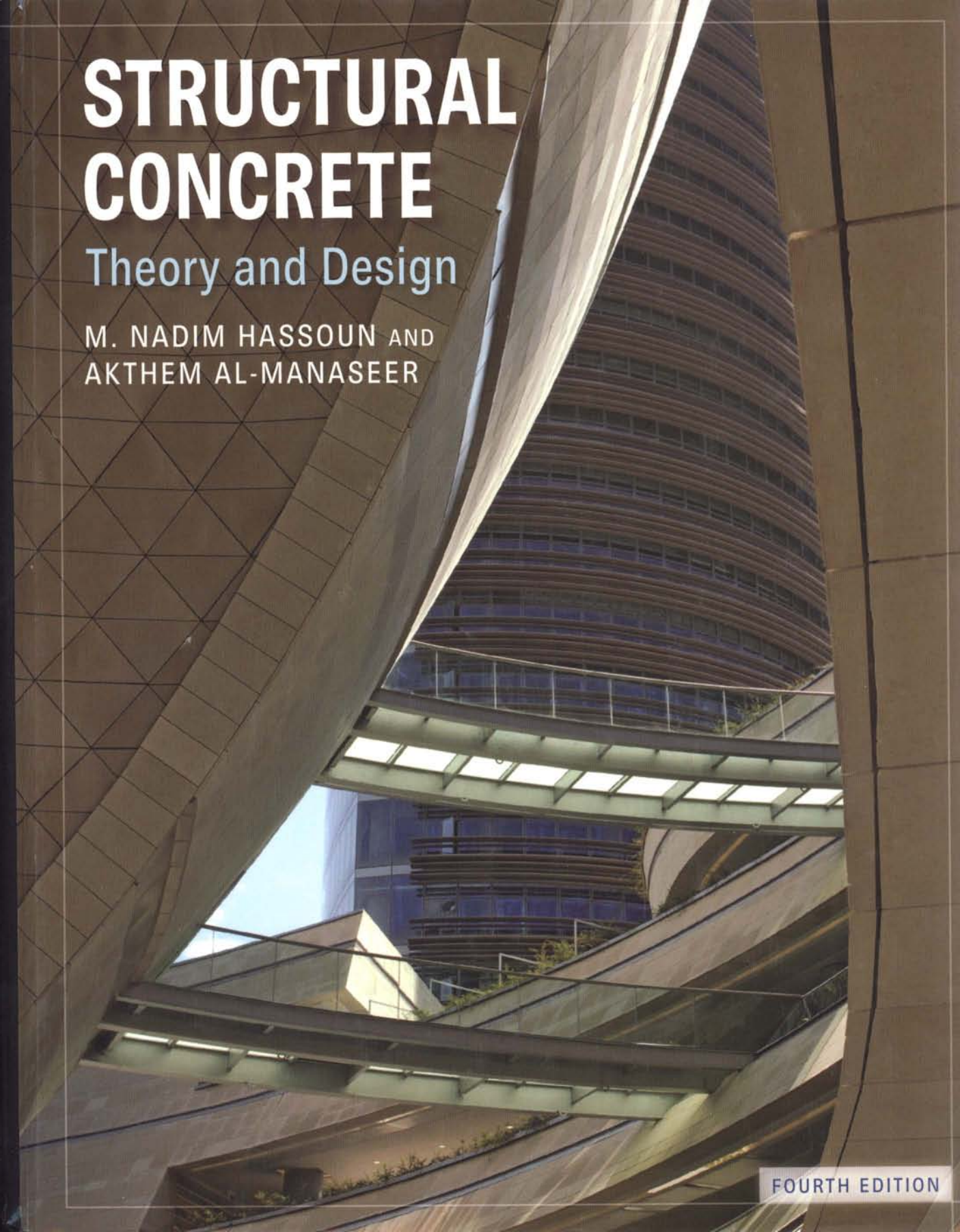
STRUCTURAL CONCRETE

Theory and Design

M. NADIM HASSOUN AND
AKTHEM AL-MANASEER



FOURTH EDITION

Structural Concrete

Structural Concrete

Theory and Design

Fourth Edition

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A companion Web site for the book is available at www.wiley.com/college/hassoun. This Web site contains MSExcel spreadsheets that enable students to evaluate different design aspects of concrete members in an interactive environment, and a solutions manual for instructors.

PREFACE

The main objective of a course on structural concrete design is to develop, in the engineering student, the ability to analyze and design a reinforced concrete member subjected to different types of forces in a simple and logical manner using the basic principles of statistics and some empirical formulas based on experimental results. Once the analysis and design procedure is fully understood, its application to different types of structures becomes simple and direct, provided that the student has a good background in structural analysis.

The material presented in this book is based on the requirements of the American Concrete Institute (ACI) Building Code (318–08). Also, information has been presented on material properties, including volume changes of concrete, stress–strain behavior, creep, and elastic and nonlinear behavior or reinforced concrete.

Concrete structures are widely used in the United States and almost all over the world. The progress in the design concept has increased in the last few decades, emphasizing safety, serviceability, and economy. To achieve economical design of a reinforced concrete member, specific restrictions, rules, and formulas are presented in the codes to ensure both safety and reliability of the structure. Engineering firms expect civil engineering graduates to understand the code rules and, consequently, to be able to design a concrete structure effectively and economically with minimum training period or overhead costs. Taking this into consideration, this book is written to achieve the following objectives:

1. To present the material for the design of reinforced concrete members in a simple and logical approach.
2. To arrange the sequence of chapters in a way compatible with the design procedure of actual structures.
3. To provide a large number of examples in each chapter in clear steps to explain the analysis and design of each type of structural member.
4. To provide an adequate number of practical problems at the end of each chapter to achieve a high level of comprehension.
5. To explain the failure mechanism of a reinforced concrete beam due to flexure and to develop the necessary relationships and formulas for design.

6. To explain *why* the code used specific equations and specific restrictions on the design approach based either on a mathematical model or experimental results. This approach will improve the design ability of the student.
7. To provide adequate number of design aids to help the student in reducing the repetitive computations of specific commonly used values.
8. To enhance the student's ability to use a total quality and economical approach in the design of concrete structures and to help the student to design reinforced concrete members with confidence.
9. To explain the nonlinear behavior and the development of plastic hinges and plastic rotations in continuous reinforced concrete structures.
10. To provide a summary at the end of each chapter to help the student to review the materials of each chapter separately.
11. To provide new information on the design of special members, such as beams with variable depth (Chapter 8), stairs (Chapter 18), seismic design utilizing IBC 2006 (Chapter 20), and beams curved in plan (Chapter 21), that are not covered in other books on concrete.
12. To present information on the design of reinforced concrete frames, principles of limit design, and moment redistribution in continuous reinforced concrete structures.
13. To provide examples in SI units in all chapters of the book. Equivalent conversion factors from customary units to SI units are also presented. Design tables in SI units are given in Appendix B.
14. References are presented at the end of most chapters.

The book is an outgrowth of the author's lecture notes, which represent their teaching and industrial experience over the past 28 years. The industrial experience of the authors includes the design and construction supervision and management of many reinforced, prestressed, and precast concrete structures. This is in addition to the consulting work they performed for international design and construction firms, professional registration in the United Kingdom, Canada, and other countries, and a comprehensive knowledge of other European codes on the design of concrete structures.

The book is written to cover two courses in reinforced concrete design. Depending on the proficiency required, the first course may cover Chapters 1 through 11 and part of Chapter 13, whereas the second course may cover the remaining chapters. Parts of the late chapters may also be taught in the first course as needed. A number of optional sections have been included in various chapters. These sections are indicated by an asterisk (*) in the Table of Contents and may easily be distinguished from those that form the basic requirements of the first course. The optional sections may be covered in the second course or relegated to a reading assignment. Brief descriptions of the chapters are given below.

The first chapter of the book presents information on the historical development of concrete, codes of practice, loads and safety provisions, and design philosophy and concepts. The second chapter deals with the properties of concrete as well as steel reinforcement used in the design of reinforced concrete structures, including stress–strain relationships, modulus of elasticity and shear modulus of concrete, shrinkage, creep, fire resistance, high-performance concrete, and fibrous concrete. Because the current ACI Code emphasizes the strength approach based on strain limits, this approach has been adopted throughout the text. Chapters 3 and 4 cover the analysis and design of reinforced concrete sections based on strain limits. The behavior of reinforced concrete beams loaded to failure, the types of flexural failure, and failure mechanism

are explained very clearly. It is essential for the student to understand the failure concept and the inherent reserve strength and ductility before using the necessary design formulas.

Chapter 5 covers alternative design methods based on methods described in Appendix A, B, and C of the ACI code. It explains the alternative load factors with the relative strength reduction factors and describes the strut and tie provisions.

Chapter 6 deals with the serviceability of reinforced concrete beams, including deflection and control of cracking. Chapters 7 and 8 cover the bond, development length, shear, and diagonal tension. In Chapter 8, expressions are presented for the design of members of variable depth in addition to prismatic sections and deep beams. It is quite common sometimes to design members with variable depth in actual structures. An example is introduced to explain the design of deep beams using the strut and tie approach.

Chapter 9 covers the design of one-way slabs, including joist-floor systems. Distributions of loads from slabs to beams and columns are also presented in this chapter to enhance the student's understanding of the design loads on each structural component. Chapter 10, 11, and 12 cover the design of axially loaded, eccentrically loaded, and long columns, respectively. Chapter 10 allows the student to understand the behavior of columns, failure conditions, ties and spirals, and other code limitations. Absorbing basic information, the student is introduced in Chapter 11 to the design of columns subjected to compression and bending. New mathematical models are introduced to analyze column sections controlled by compression or tension stresses. Biaxial bending for rectangular and circular columns are introduced using Bresler, PCA, and Hsu methods. Design of long columns is presented in Chapter 12 using the ACI moment-magnifier method.

Chapter 13 and 14 cover the design of footings and retaining walls, whereas Chapter 15 covers the design of reinforced concrete sections for shear and torsion. Torsional theories as well as ACI Code design procedure are explained. Chapter 16 deals with continuous beams and frames. A unique feature of this chapter is the introduction of the design of frames, frame hinges, limit state design collapse mechanism, rotation and plastic hinges, and moment redistribution. Adequate examples are presented to explain these concepts.

Design of two-way slabs introduced in Chapter 17. All types of two-way slabs, including waffle slabs, are presented with adequate examples. Summary of the design procedure is introduced with tables and diagrams. Chapter 18 covers the design of reinforced concrete stairs. Slabtype as well as stepped-type stairs are explained. The second type, although quite common, has not been covered in any text. Chapter 19 covers an introduction to prestressed concrete. Methods of prestressing, fully and partially prestressed concrete design, losses, and shear design are presented with examples. Chapter 20 presents the seismic design and analysis of members utilizing the IBC 2006 and the ACI code. Chapter 21 deals with the design of curved beams. In actual structures curved beams are used frequently. These beams are subjected to flexure, shear, and torsion.

In Appendix A and B of this book, design tables using customary units and SI units are presented.

The photos shown in this book were taken by the authors. We wish to express appreciation to John Gardner and Murat Saatcioglu from the University of Ottawa, Canada, for the photos provided in the seismic chapter.

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Finally, the book is written to provide basic reference materials on the analysis and design of structural concrete members in a simple, practical, and logical approach. Because this is a required course for seniors in civil engineering, we believe this book will be accepted by reinforced concrete instructors at different universities as well as designers who can make use of the information in their practical design of reinforced concrete structures.

M. Nadim Hassoun *Akthem Al-Manaseer*

NOTATION

c	Distance from extreme compression fiber to neutral axis
c_2	Side of rectangular column measured transverse to the span
C	Cross-sectional constant $\sum (1 - 0.63x/y)x^3y/3$; compression force
C_c	Compression force in a concrete section with a depth equal to a
C_m	Correction factor applied to the maximum end moment in columns
C_r	Creep coefficient = creep strain per unit stress per unit length
C_s	Force in compression steel
C_t	Factor relating shear and torsional stress properties $= b_w d/\sum x^2y$
C_w	Compression force in web
C_1	Force in the compression steel
d	Distance from extreme compression fiber to centroid of tension steel
d'	Distance from extreme compression fiber to centroid of compression steel
d_b	Nominal diameter of reinforcing bar
d_c	Distance from tension extreme fiber to center of bar closest to that fiber, used for crack control
d_t	Distance from extreme compression fibers to extreme tension steel
D	Dead load, diameter of a circular section
e	Eccentricity of load
e'	Eccentricity of load with respect to centroid of tension steel
E	Modulus of elasticity, force created by earthquake
E_c	Modulus of elasticity of concrete $= 33w^{1.5}\sqrt{f'_c}$
E_{cb}	Modulus of elasticity of beam concrete
E_{cc}	Modulus of elasticity of column concrete
E_{cs}	Modulus of elasticity of slab concrete
EI	Flexural stiffness of compression member
E_s	Modulus of elasticity of steel $= 29 \times 10^6$ psi $= 2 \times 10^5$ MPa
f	Flexural stress

f_c	Maximum flexural compressive stress in concrete due to service loads
f_{ca}	Allowable compressive stress in concrete (alternate design method)
f'_c	28-day compressive strength of concrete (standard cylinder strength)
f_d	Compressive strength of concrete at transfer (initial prestress)
f_{pc}	Compressive stress in concrete due to prestress after all losses
f_{pe}	Compressive stress in concrete at extreme fiber due to the effective prestressing force after all losses
f_{ps}	Stress in prestress steel at nominal strength
f_{pu}	Tensile strength of prestressing tendons
f_{py}	Yield strength of prestressing tendons
f_r	Modulus of rupture of concrete $= 7.5\lambda\sqrt{f'_c}$ psi
f_s	Stress in tension steel due to service load
f'_s	Stress in the compression steel due to service load
f_{se}	Effective stress in prestressing steel after all losses
f_t	Tensile stress in concrete
f_y	Yield strength of steel reinforcement
F	Lateral pressure of liquids
F_n	Nominal strength of a strut, tie, or nodal zone
F_{ns}	Nominal strength of a strut
F_{nt}	Nominal strength of a tie
G	Shear modulus of concrete (in torsion) $= 0.45E_c$
h	Total depth of beam or slab or column
h_f	Depth of flange in flanged sections
h_p	Total depth of shearhead cross section
H	Lateral earth pressure
I	Moment of inertia
I_b	Moment of inertia of gross section of beam about its centroidal axis
I_c	Moment of inertia of gross section of column
I_{cr}	Moment of inertia of cracked transformed section
I_e	Effective moment of inertia, used in deflection
I_g	Moment of inertia of gross section neglecting steel
I_s	Moment of inertia of gross section of slab
I_{se}	Moment of inertia of steel reinforcement about centroidal axis of section
J	Polar moment of inertia
K	Kip = 1000 lb, a factor used to calculate effective column length
K_b	Flexural stiffness of beam
K_c	Flexural stiffness of column
K_{ec}	Flexural stiffness of equivalent column
K_s	Flexural stiffness of slab
K_t	Torsional stiffness of torsional member
KN	Kilonewton
Ksi	Kip per square inch
ℓ_n	Clear span
ℓ_u	Unsupported length of column
L	Live load, span length
l_d	Development length

l_{dh}	l_{hb} times the applicable modification factor
l_{hb}	Basic development length of a standard hook
l_n	Clear span
l_u	Unsupported length of compression member
l_v	Length of shearhead arm
l_1	Span length in the direction of moment
l_2	Span length in direction transverse to span l_1
M	Bending moment
M_1	Smaller end moment at end of column
M_2	Larger end moment at end of column
M_a	Maximum service load moment
M_b	Balanced moment in columns, used with P_b
M_{cr}	Cracking moment
M_m	Modified moment
M_n	Nominal moment strength $= M_u/\phi$
M'_n	Nominal moment strength using an eccentricity e'
M_o	Total factored moment
M_p	Plastic moment
M_u	Moment strength due to factored loads
M_{u1}	Part of M_u when calculated as singly reinforced
M_{u2}	Part of M_u due to compression reinforcement or overhanging flanges in T- or L-sections
M'_u	Moment strength using an eccentricity e'
M_v	Shearhead moment resistance
n	Modular ratio $= E_s/E_c$
N	Normal force
N_u	Factored normal load
N_1	Normal force in bearing at base of column
NA	Neutral axis
psi	Pounds per square inch
P_{cp}	Outside perimeter of gross area $= 2(x_0 + y_0)$
P_o	Perimeter of shear flow in area A_o
P	Unfactored concentrated load
P_b	Balanced load in column (at failure)
P_c	Euler buckling load
P_n	Nominal axial strength of column for a given e
P_o	Axial strength of a concentrically loaded column
P_s	Prestressing force in the tendon at the jacking end
P_u	Factored load $= \phi P_n$
P_x	Prestressing force in the tendon at any point x
q	Soil-bearing capacity
q_a	Allowable bearing capacity of soil
q_u	Ultimate bearing capacity of soil using factored loads
r	Radius of gyration, radius of a circle
R	Resultant of force system, reduction factor for long columns, or $R = R_u/\phi$
R_u	A factor $= M_u/bd^2$
s	Spacing between bars, stirrups, or ties

SI	International system of units
t	Thickness of a slab
T	Torque, tension force
T_c	Nominal torsional strength provided by concrete
T_{cr}	Cracking torsional moment
T_n	Nominal torsional strength provided by concrete and steel
T_s	Nominal torsional strength provided by reinforcement
T_u	Torque provided by factored load $= \phi T_n$
u	Bond stress
U	Design strength required to resist factored loads
V	Shear stress produced by working loads
v_c	Shear stress of concrete
v_{cr}	Shear stress at which diagonal cracks develop
v_h	Horizontal shear stress
v_t	Shear stress produced by a torque
v_u	Shear stress produced by factored loads
V	Unfactored shear force
V_c	Shear strength of concrete
V_{ci}	Nominal shear strength of concrete when diagonal cracking results from combined shear and moment
V_{cw}	Nominal shear strength of concrete when diagonal cracking results from excessive principal tensile stress in web
V_d	Shear force at section due to unfactored dead load (d = distance from the face of support)
V_n	Nominal shear strength $= V_c + V_s$
V_p	Vertical component of effective prestress force at section
V_s	Shear strength carried by reinforcement
V_u	Shear force due to factored loads
w	Width of crack at the extreme tension fiber, unit weight of concrete
w_u	Factored load per unit length of beam or per unit area of slab
W	Wind load or total load
x_o	Length of the short side of a rectangular section
x_1	Length of the short side of a rectangular closed stirrup
y_b	Same as y_t, except to extreme bottom fibers
y_o	Length of the long side of a rectangular section
y_t	Distance from centroidal axis of gross section, neglecting reinforcement, to extreme top fiber
y_1	Length of the long side of a rectangular closed stirrup
α	Angle of inclined stirrups with respect to longitudinal axis of beam, ratio of stiffness of beam to that of slab at a joint
α_c	Ratio of flexural stiffness of columns to combined flexural stiffness of the slabs and beams at a joint; $(\Sigma\ K_c)/\Sigma(K_s + K_b)$
α_{ec}	Ratio of flexural stiffness of equivalent column to combined flexural stiffness of the slabs and beams at a joint: $(K_{ec})/\Sigma(K_s + K_b)$
α_m	Average value of a for all beams on edges of a panel
α_v	Ratio of stiffness of shearhead arm to surrounding composite slab section
β	Ratio of long to short side of rectangular footing, measure of curvature in biaxial bending

β_1	Ratio of a/c, where a = depth of stress block and c = distance between neutral axis and extreme compression fibers (This factor is 0.85 for $f'_c \leq 4000$ psi and decreases by 0.05 for each 1000 psi in excess of 4000 psi but is at least 0.65.)
β_a	Ratio of unfactored dead load to unfactored live load per unit area
β_c	Ratio of long to short sides of column or loaded area
β_{dns}	Ratio of maximum factored dead load moment to maximum factored total moment
β_t	Ratio of torsional stiffness of edge beam section to flexural stiffness of slab: $E_{cb}C/2E_{cs}I_s$
γ	Distance between rows of reinforcement on opposite sides of columns to total depth of column h
γ_f	Fraction of unbalanced moment transferred by flexure at slab-column connections
γ_p	Factor for type of prestressing tendon (0.4 or 0.28)
γ_v	Fraction of unbalanced moment transferred by eccentricity of shear at slab-column connections
δ	Magnification factor
δ_{ns}	Moment magnification factor for frames braced against sidesway
δ_s	Moment magnification factor for frames not braced against sidesway
Δ	Deflection
ε	Strain
ε_c	Strain in concrete
ε_s	Strain in steel
ε'_s	Strain in compression steel
ε_y	Yield strain = f_y/E_s
θ	Slope angle
λ	Multiplier factor for reduced mechanical properties of lightweight concrete
λ_Δ	Multiplier for additional long-time deflection
μ	Poisson's ratio; coefficient of friction
ζ	Parameter for evaluating capacity of standard hook
π	A constant equal to approximately 3.1416
ρ	Ratio of the tension steel area to the effective concrete area = A_s/bd
ρ'	Ratio of compression steel area to effective concrete area = A'_s/bd
ρ_1	$(\rho - \rho')$
ρ_b	Balanced steel ratio
ρ_g	Ratio of total steel area to total concrete area
ρ_p	Ratio of prestressed reinforcement A_{ps}/bd
ρ_s	Ratio of volume of spiral steel to volume of core
ρ_w	$A_s/b_w d$
ϕ	Strength-reduction factor
ψ_e	Factor used to modify development length based on reinforcement coating
ψ_s	Factor used to modify development length based on reinforcing size
ψ_t	Factor used to modify development length based on reinforcement location
ω	Tension reinforcing index = $\rho f_y/f'c$
ω'	Compression reinforcing index = $\rho' f_y/f'_c$
ω_p	Prestressed steel index = $\rho_p f_{ps}/f'_c$
ω_{pw}	Prestressed steel index for flanged sections
ω_w	Tension reinforcing index for flanged sections
ω'_w	Compression reinforcing index for flanged sections computed as for ω, ω_p and ω'

CONVERSION FACTORS

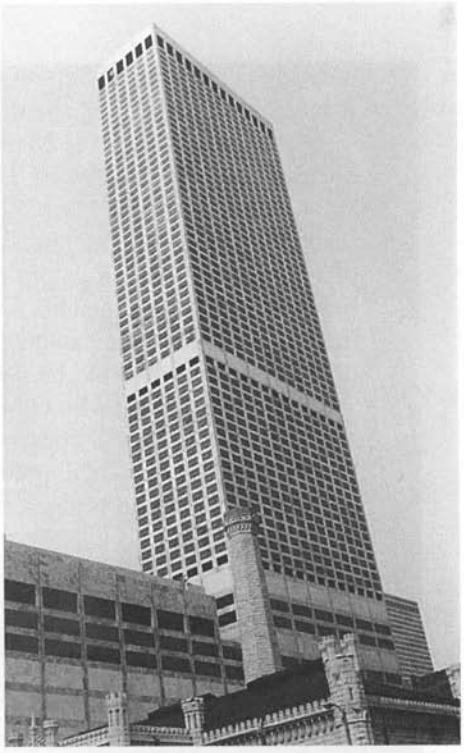
To Convert	to	Multiply By
1. Length		
Inch	Millimeter	25.4
Foot	Millimeter	304.8
Yard	Meter	0.9144
Meter	Foot	3.281
Meter	Inch	39.37
2. Area		
Square inch	Square millimeter	645
Square foot	Square meter	0.0929
Square yard	Square meter	0.836
Square meter	Square foot	10.76
3. Volume		
Cubic inch	Cubic millimeter	16390
Cubic foot	Cubic meter	0.02832
Cubic yard	Cubic meter	0.765
Cubic foot	Liter	28.3
Cubic meter	Cubic foot	35.31
Cubic meter	Cubic yard	1.308
4. Mass		
Ounce	Gram	28.35
Pound (lb)	Kilogram	0.454
Pound	Gallon	0.12

To Convert	to	Multiply By
Short ton (2000 lb)	Kilogram	907
Long ton (2240 lb)	Kilogram	1016
Kilogram	Pound (lb)	2.205
Slug	Kilogram	14.59
5. Density		
Pound/cubic foot	Kilogram/cubic meter	16.02
Kilogram/cubic meter	Pound/cubic foot	0.06243
6. Force		
Pound (lb)	Newton (N)	4.448
Kip (1000 lb)	Kilonewton (kN)	4.448
Newton (N)	Pound	0.2248
Kilonewton (kN)	Kip (K)	0.225
7. Force/length		
Kip/foot	Kilonewton/meter	14.59
Kilonewton/meter	Pound/foot	68.52
Kilonewton/meter	Kip/foot	0.06852
8. Force/area (stress)		
Pound/square inch (psi)	Newton/square centimeter	0.6895
Pound/square inch (psi)	Newton/square millimeter (MPa)	0.0069
Kip/square inch (Ksi)	Meganewton/square meter	6.895
Kip/square inch (Ksi)	Newton/square millimeter	6.895
Pound/square foot	Kilonewton/square meter	0.04788
Pound/square foot	Newton/square meter	47.88
Kip/square foot	Kilonewton/square meter	47.88
Newton/square millimeter	Kip/square inch (Ksi)	0.145
Kilonewton/square meter	Kip/square foot	0.0208
Kilonewton/square meter	Pound/square foot	20.8
9. Moments		
Foot·Kip	Kilonewton·meter	1.356
Inch·Kip	Kilonewton·meter	0.113
Inch·Kip	Kilogram force·meter	11.52
Kilonewton·meter	Foot·Kip	0.7375

Structural Concrete

CHAPTER 1

INTRODUCTION



Water Tower Place, Chicago, 74 stories, tallest concrete building in the United States.

1.1 STRUCTURAL CONCRETE

The design of different structures is achieved by performing, in general, two main steps: (1) determining the different forces acting on the structure using proper methods of structural analysis, and (2) proportioning all structural members economically, considering the safety, stability, serviceability, and functionality of the structure. Structural concrete is one of the materials commonly used to design all types of buildings. Its two component materials, concrete and steel, work together to form structural members that can resist many types of loadings. The key to its performance lies in strengths that are complementary: Concrete resists compression and steel reinforcement resists tension forces.

The term *structural concrete* indicates all types of concrete used in structural applications. Structural concrete may be plain, reinforced, prestressed, or partially prestressed concrete; in addition, concrete is used in composite design. Composite design is used for any structural member, such as beams or columns, when the member contains a combination of concrete and steel shapes.

1.2 HISTORICAL BACKGROUND

The first modern record of concrete is as early as 1760, when John Smeaton used it in Britain in the first lock on the river Calder [1]. The walls of the lock were made of stones filled in with concrete. In 1796, J. Parker discovered Roman natural cement, and 15 years later Vicat burned a mixture of clay and lime to produce cement. In 1824, Joseph Aspdin manufactured

portland cement in Wakefield, Britain. It was called portland cement because when it hardened, it resembled stone from the quarries of the Isle of Portland.

In France, François Marte Le Brun built a concrete house in 1832 in Moissac, in which he used concrete arches of 18-ft span. He used concrete to build a school in St. Aignan in 1834 and a church in Corbarièce in 1835. Joseph Louis Lambot [2] exhibited a small rowboat made of reinforced concrete at the Paris Exposition in 1854. In the same year, W. B. Wilkinson of England obtained a patent for a concrete floor reinforced by twisted cables. The Frenchman François Cignet obtained his first patent in 1855 for his system of iron bars, which were embedded in concrete floors and extended to the supports. One year later, he added nuts at the screw ends of the bars, and in 1869, he published a book describing the applications of reinforced concrete.

Joseph Monier, who obtained his patent in Paris on July 16, 1867, was given credit for the invention of reinforced concrete [3]. He made garden tubs and pots of concrete reinforced with iron mesh, which he exhibited in Paris in 1867. In 1873, he registered a patent to use reinforced concrete in tanks and bridges, and four years later, he registered another patent to use it in beams and columns [1].

In the United States, Thaddeus Hyatt conducted flexural tests on 50 beams that contained iron bars as tension reinforcement and published the results in 1877. He found that both concrete and steel can be assumed to behave in a homogeneous manner for all practical purposes. This assumption was important for the design of reinforced concrete members using elastic theory. He used prefabricated slabs in his experiments and considered prefabricated units to be best cast in T-sections and placed side by side to form a floor slab. Hyatt is generally credited with developing the principles upon which the analysis and design of reinforced concrete are now based.

A reinforced concrete house was built by W. E. Ward near Port Chester, New York, in 1875. It used reinforced concrete for walls, beams, slabs, and staircases. P. B. In 1877, Write described in the *American Architect and Building News* the applications of reinforced concrete in Ward's house as a new method in building construction.

E. L. Ransome, head of the Concrete Steel Company in San Francisco, used reinforced concrete in 1879 and deformed bars for the first time in 1884. During 1889–1891, he built the two-story Leland Stanford Museum in San Francisco using reinforced concrete. He also built a reinforced concrete bridge in San Francisco. In 1900, after Ransome introduced the reinforced concrete skeleton, the thick wall system started to disappear in construction. He registered the skeleton type of structure in 1902, using spiral reinforcement in the columns as was suggested by Armand Considére of France. A. N. Talbot, of the University of Illinois, and F. E. Turneaure and M. O. Withney, of the University of Wisconsin, conducted extensive tests on concrete to determine its behavior, compressive strength, and modulus of elasticity.

In Germany, G. A. Wayass bought the French Monier patent in 1879 and published his book on Monier methods of construction in 1887. Rudolph Schuster bought the patent rights in Austria, and the name of Monier spread throughout Europe, which is the main reason for crediting Monier as the inventor of reinforced concrete.

In 1900, the Ministry of Public Works in France called for a committee headed by Armand Considére, chief engineer of roads and bridges, to establish specifications for reinforced concrete, which were published in 1906.

Reinforced concrete was further refined by introducing some precompression in the tension zone to decrease the excessive cracks. This refinement was the preliminary introduction of partial and full prestressing. In 1928, Eugene Freyssinet established the practical technique of using prestressed concrete [4].



The Barwick House, a three-story concrete building built in 1905, Montreal, Canada.

From 1915 to 1935, research was conducted on axially loaded columns and creep effects on concrete; in 1940, eccentrically loaded columns were investigated. Ultimate-strength design started to receive special attention, in addition to diagonal tension and prestressed concrete. The American Concrete Institute Code (ACI Code) specified the use of ultimate-strength design in 1963 and included this method in all later codes. Building codes and specifications for the design of reinforced concrete structures are established in most countries, and research continues on developing new applications and more economical designs.

1.3 ADVANTAGES AND DISADVANTAGES OF REINFORCED CONCRETE

Reinforced concrete, as a structural material, is widely used in many types of structures. It is competitive with steel if economically designed and executed.

The advantages of reinforced concrete can be summarized as follows:

1. It has a relatively high compressive strength.
2. It has better resistance to fire than steel.
3. It has a long service life with low maintenance cost.
4. In some types of structures, such as dams, piers, and footings, it is the most economical structural material.
5. It can be cast to take the shape required, making it widely used in precast structural components. It yields rigid members with minimum apparent deflection.

The disadvantages of reinforced concrete can be summarized as follows:

1. It has a low tensile strength of about one-tenth of its compressive strength.
2. It needs mixing, casting, and curing, all of which affect the final strength of concrete.
3. The cost of the forms used to cast concrete is relatively high. The cost of form material and artisanry may equal the cost of concrete placed in the forms.
4. It has a low compressive strength as compared to steel (the ratio is about 1:10, depending on materials), which leads to large sections in columns of multistory buildings.
5. Cracks develop in concrete due to shrinkage and the application of live loads.

1.4 CODES OF PRACTICE

The design engineer is usually guided by specifications called the codes of practice. Engineering specifications are set up by various organizations to represent the minimum requirements necessary for the safety of the public, although they are not necessarily for the purpose of restricting engineers.

Most codes specify design loads, allowable stresses, material quality, construction types, and other requirements for building construction. The most significant code for structural concrete design in the United States is the Building Code Requirements for Structural Concrete, ACI 318, or the ACI Code. Most of the design examples of this book are based on this code. Other codes of practice and material specifications in the United States include the International Code, the Uniform Building Code, Standard Building Code, National Building Code, Basic Building Code, South Florida Building Code, American Association of State Highway and Transportation Officials (AASHTO) specifications, and specifications issued by the American Society for Testing and Materials (ASTM), American Railway Engineering Association (AREA), and Bureau of Reclamation, Department of the Interior.

Different codes other than those of the United States include the British Standard (BS) Code of Practice for Reinforced Concrete, CP 110 and BS 8110; the National Building Code of Canada; the German Code of Practice for Reinforced Concrete, DIN 1045; Specifications for Steel Reinforcement (U.S.S.R.); and Technical Specifications for the Theory and Design of Reinforced Concrete Structures, CC-BA (France), and the CEB Code (Comitè European Du Beton).

1.5 DESIGN PHILOSOPHY AND CONCEPTS

The design of a structure may be regarded as the process of selecting the proper materials and proportioning the different elements of the structure according to state-of-the-art engineering science and technology. In order to fulfill its purpose, the structure must meet the conditions of safety, serviceability, economy, and functionality. This can be achieved using design approach-based strain limits in concrete and steel reinforcement.

The unified design method (UDM) is based on the strength of structural members assuming a failure condition, whether due to the crushing of the concrete or to the yield of the reinforcing steel bars. Although there is some additional strength in the bars after yielding (due to strain hardening), this additional strength is not considered in the analysis of reinforced concrete members. In this approach, the actual loads, or working loads, are multiplied by load factors to obtain the factored design loads. The load factors represent a high percentage of the factor for safety required in the design. Details of this method are presented in Chapters 3, 4, and 11. The

ACI Code emphasizes this method of design, and its provisions are presented in the body of the Code. The reason for introducing this approach by the ACI Code relates to the fact that different design methods were developed for reinforced and prestressed concrete beams and columns. Also, design procedures for prestressed concrete were different from reinforced concrete. The purpose of the Code approach is to simplify and unify the design requirements for reinforced and prestressed flexural members and compression members.

A second approach for the design of reinforced and prestressed concrete flexural and compression members is called the strength design method, or the alternative provisions (ADM), as introduced in the ACI Code, Appendix B. When this method is used in the design, the designer must adhere to all sections of Appendixes B and C and substitute accordingly for the corresponding sections of the Code. Reinforcement limits, strength reduction factors, load factors, and moment redistribution are affected. The provisions of this method satisfy the Code and are equally acceptable.

A third approach for the design of concrete members is called the strut and tie method (STM). The provisions of this method are introduced in the ACI Code, Appendix A. It applies effectively in regions of discontinuity such as support and load applications on beams. Consequently, the structural element is divided into segments and then analyzed using the truss analogy approach, where the concrete resists compression forces as a strut, while the steel reinforcement resists tensile forces as a tie.

A basic method that is not commonly used is called the working stress design or the elastic design method. The design concept is based on the elastic theory assuming a straight line stress distribution along the depth of the concrete section under service loads. The members are proportioned on the basis of certain allowable stresses in concrete and steel. The allowable stresses are fractions of the crushing strength of concrete and yield strength of steel. This method has been deleted from the ACI Code. The application of this approach is still used in the design of prestressed concrete members under service load conditions, as shown in Chapter 19.

Limit state design is a further step in the strength design method. It indicates the state of the member in which it ceases to meet the service requirements such as losing its ability to withstand external loads, or suffering excessive deformation, cracking, or local damage. According to the limit state design, reinforced concrete members have to be analyzed with regard to three limiting states:

1. Load carrying capacity (safety, stability, and durability)
2. Deformation (deflections, vibrations, and impact)
3. The formation of cracks.

The aim of this analysis is to ensure that no limiting state will appear in the structural member during its service life.

1.6 UNITS OF MEASUREMENT

Two units of measurement are commonly used in the design of structural concrete. The first is the U.S. customary system (lying mostly in its human scale and its ingenious use of simple numerical proportions), and the second is the SI (Le Système International d'Unités), or metric, system.

The metric system is planned to be in universal use within the coming few years. The United States is committed to change to SI units. Great Britain, Canada, Australia, and other countries have been using SI units for several years.

The base units in the SI system are the units of length, mass, and time, which are the meter (m), the kilogram (kg), and the second (s), respectively. The unit of force, a derived unit called

the newton (N), is defined as the force that gives the acceleration of 1 meter per second per second (1 m/s^2) to a mass of 1 kg, or 1 N = 1 kg × m/s^2.

The weight of a body, W, which is equal to the mass, m, multiplied by the local gravitational acceleration, g (9.81 m/s^2), is expressed in newtons (N). The weight of a body of 1 kg mass is $W = mg = 1\,\text{kg} \times 9.81\,\text{m/s}^2 = 9.81\,\text{N}$.

Multiples and submultiples of the base SI units can be expressed through the use of prefixes. The prefixes most frequently used in structural calculations are the kilo (k), mega (M), milli (m), and micro (μ). For example,

$$1\text{ km} = 1000\text{ m} \quad 1\text{ mm} = 0.001\text{ m} \quad 1\,\mu\text{m} = 10^{-6}\text{ m}$$

$$1\text{ kN} = 1000\text{ N} \quad 1\text{ Mg} = 1000\text{ kg} = 10^{6}\text{ g}$$

1.7 LOADS

Structural members must be designed to support specific loads.

Loads are those forces for which a given structure should be proportioned. In general, loads may be classified as dead or live.

Dead loads include the weight of the structure (its self-weight) and any permanent material placed on the structure, such as tiles, roofing materials, and walls. Dead loads can be determined with a high degree of accuracy from the dimensions of the elements and the unit weight of materials.

Live loads are all other loads that are not dead loads. They may be steady or unsteady or movable or moving; they may be applied slowly, suddenly, vertically, or laterally, and their magnitudes may fluctuate with time. In general, live loads include the following:

- Occupancy loads caused by the weight of the people, furniture, and goods
- Forces resulting from wind action and temperature changes
- The weight of snow if accumulation is probable
- The pressure of liquids or earth on retaining structures
- The weight of traffic on a bridge
- Dynamic forces resulting from moving loads (impact), earthquakes, or blast loading

The ACI Code does not specify loads on structures; however, occupancy loads on different types of buildings are prescribed by the American National Standards Institute (ANSI) [5]. Some typical values are shown in Table 1.1. Table 1.2 on page 7 shows weights and specific gravity of various materials.

AASHTO and AREA specifications prescribe vehicle loadings on highway and railway bridges, respectively. These loads are given in Refs. 6 and 7.

Snow loads on structures may vary between 10 and 40 lb/ft^2 (0.5 and 2 kN/m^2), depending on the local climate.

Wind loads may vary between 15 and 30 lb/ft^2, depending on the velocity of wind. The wind pressure of a structure, F, can be estimated from the following equation:

$$F = 0.00256 C_s V^2 \tag{1.1}$$

where

V = velocity of air (mi/h)
C_s = shape factor of the structure
F = the dynamic wind pressure (lb/ft^2)

Table 1.1 Typical Uniformly Distributed Design Loads

Occupancy	Contents	Design Live Load lb/ft²	Design Live Load kN/m²
Assembly hall	Fixed seats	60	2.9
	Movable seats	100	4.8
Hospital	Operating rooms	60	2.9
	Private rooms	40	1.9
Hotel	Guest rooms	40	1.9
	Public rooms	100	4.8
	Balconies	100	4.8
Housing	Private houses and apartments	40	1.9
	Public rooms	100	4.8
Institution	Classrooms	40	1.9
	Corridors	100	4.8
Library	Reading rooms	60	2.9
	Stack rooms	150	7.2
Office building	Offices	50	2.4
	Lobbies	100	4.8
Stairs (or balconies)		100	4.8
Storage warehouses	Light	100	4.8
	Heavy	250	12.0
Yards and terraces		100	4.8

Table 1.2 Density and Specific Gravity of Various Materials

Material	Density lb/ft³	Density kg/m³	Specific Gravity
Building materials			
Bricks	120	1,924	1.8–2.0
Cement, portland, loose	90	1,443	—
Cement, portland, set	183	2,933	2.7–3.2
Earth, dry, packed	95	1,523	—
Sand or gravel, dry, packed	100–120	1,600–1,924	—
Sand or gravel, wet	118–120	1,892–1,924	—
Liquids			
Oils	58	930	0.9–0.94
Water (at 4 °C)	62.4	1,000	1.0
Ice	56	898	0.88–0.92
Metals and minerals			
Aluminum	165	2,645	2.55–2.75
Copper	556	8,913	9.0
Iron	450	7,214	7.2
Lead	710	11,380	11.38
Steel, rolled	490	7,855	7.85
Limestone or marble	165	2,645	2.5–2.8
Sandstone	147	2,356	2.2–2.5
Shale or slate	175	2,805	2.7–2.9
Normal-weight concrete			
Plain	145	2,324	2.2–2.4
Reinforced or prestressed	150	2,405	2.3–2.5

As an example, for a wind of 100 mi/h with $C_s = 1$, the wind pressure is equal to 25.6 lb/ft^2. It is sometimes necessary to consider the effect of gusts in computing the wind pressure by multiplying the wind velocity in Eq. 1.1 by a gust factor, which generally varies between 1.1 and 1.3.

The shape factor, C_s, varies with the horizontal angle of incidence of the wind. On vertical surfaces of rectangular buildings, C_s may vary between 1.2 and 1.3. Detailed information on wind loads can be found in Ref. 5.

1.8 SAFETY PROVISIONS

Structural members must always be proportioned to resist loads greater than the service or actual load in order to provide proper safety against failure. In the strength design method, the member is designed to resist factored loads, which are obtained by multiplying the service loads by load factors. Different factors are used for different loadings. Because dead loads can be estimated quite accurately, their load factors are smaller than those of live loads, which have a high degree of uncertainty. Several load combinations must be considered in the design to compute the maximum and minimum design forces. Reduction factors are used for some combinations of loads to reflect the low probability of their simultaneous occurrences. The ACI Code presents specific values of load factors to be used in the design of concrete structures (see Chapter 3, Section 3.5).

In addition to load factors, the ACI Code specifies another factor to allow an additional reserve in the capacity of the structural member. The nominal strength is generally calculated using accepted analytical procedure based on statistics and equilibrium; however, in order to account for the degree of accuracy within which the nominal strength can be calculated, and for adverse variations in materials and dimensions, a strength reduction factor, ϕ, should be used in the strength design method. Values of the strength reduction factors are given in Chapter 3, Section 3.6.

To summarize the above discussion, the ACI Code has separated the safety provision into an overload or load factor and to an undercapacity (or strength reduction) factor, ϕ. A safe design is achieved when the structure's strength, obtained by multiplying the nominal strength by the reduction factor, ϕ, exceeds or equals the strength needed to withstand the factored loadings (service loads times their load factors). For example,

$$M_u \leq \phi M_n \quad \text{and} \quad V_u \leq \phi V_n \tag{1.2}$$

where

M_u and V_u = external factored moment and shear forces

M_n and V_n = nominal flexural strength and shear strength of the member, respectively

Given a load factor of 1.2 for dead load and a load factor of 1.6 for live load, the overall safety factor for a structure loaded by a dead load, D, and a live load, L, is

$$\text{Factor of safety} = \frac{1.2D + 1.6L}{D + L}\left(\frac{1}{\phi}\right) = \frac{1.2 + 1.6(L/D)}{1 + (L/D)}\left(\frac{1}{\phi}\right) \tag{1.3}$$

The factor of safety for the various values of ϕ and L/D ratios is shown below.

ϕ	0.9				0.8				0.7			
L/D	0	1	2	3	0	1	2	3	0	1	2	3
Factor of Safety	1.33	1.56	1.63	1.67	1.50	1.74	1.83	1.88	1.71	2.00	2.10	2.15

For members subjected to flexure (beams), with tension-controlled sections, $\phi = 0.9$, and the factor of safety ranges between 1.33 for $L/D = 0$ and 1.67 for $L/D = 3$. These values are less than those specified by the ACI Code 318–99 of 1.56 for $L/D = 0$ and 1.81 for $L/D = 3.0$ based on load factors of 1.4 for the dead load and 1.7 for the live load. This reduction ranges between 17 and 8% respectively.

For members subjected to axial forces (spiral columns), $\phi = 0.7$, and the factor of safety ranges between 1.71 for $L/D = 0$ and 2.15 for $L/D = 3$. The increase in the factor of safety in columns reflects the greater overall safety requirements of these critical building elements.

A general format of Eq. 1.2 may be written as follows [8]:

$$\phi R \geq v_0 \Sigma (v_i Q_i) \tag{1.4}$$

where

R_n = nominal strength of the structural number
ϕ = undercapacity factor (<1.0)
ΣQ_i = sum of load effects
v_i = overload factor
v_0 = analysis factor (>1.0)

The subscript i indicates the load type, such as dead load, live load, and wind load. The analysis factor, v_0, is greater than 1.0 and is introduced to account for uncertainties in structural analysis. The overload factor, v_i, is introduced to account for several factors such as an increase in live load due to a change in the use of the structure and variations in erection procedures. The design concept is referred to as load and resistance factor design (LRFD) [8,9].

1.9 STRUCTURAL CONCRETE ELEMENTS

Structural concrete can be used for almost all buildings, whether single story or multistory. The concrete building may contain some or all of the following main structural elements, which are explained in detail in other chapters of the book:

- *Slabs* are horizontal plate elements in building floors and roofs. They may carry gravity loads as well as lateral loads. The depth of the slab is usually very small relative to its length or width (Chapters 9 and 17).
- *Beams* are long, horizontal or inclined members with limited width and depth. Their main function is to support loads from slabs (Chapters 3 and 4).
- *Columns* are critical members that support loads from beams or slabs. They may be subjected to axial loads or axial loads and moments (Chapters 10 and 11).
- *Frames* are structural members that consist of a combination of beams and columns or slabs, beams, and columns. They may be statically determinate or statically indeterminate frames (Chapter 16).
- *Footings* are pads or strips that support columns and spread their loads directly to the soil (Chapter 13).
- *Walls* are vertical plate elements resisting gravity as well as lateral loads as in the case of basement walls (Chapter 14).

1.10 STRUCTURAL CONCRETE DESIGN

The first step in the design of a building is the general planning carried out by the architect to determine the layout of each floor of the building to meet the owner's requirements. Once the architectural plans are approved, the structural engineer then determines the most adequate structural system to ensure the safety and stability of the building. Different structural options must be considered to determine the most economical solution based on the materials available and the soil condition. This result is normally achieved by

1. Idealizing the building into a structural model of load-bearing frames and elements
2. Estimating the different types of loads acting on the building
3. Performing the structural analysis using computer or manual calculations to determine the maximum moments, shear, torsional forces, axial loads, and other forces
4. Proportioning the different structural elements and calculating the reinforcement needed
5. Producing structural drawings and specifications with enough details to enable the contractor to construct the building properly

1.11 ACCURACY OF CALCULATIONS

In the design of concrete structures, exact calculations to determine the size of the concrete elements are not needed. Calculators and computers can give an answer to many figures after the decimal point. For a practical size of a beam, slab, or column, each dimension should be approximated to the nearest 1 or $\frac{1}{2}$ inch. Moreover, the steel bars available in the market are limited to specific diameters and areas, as shown in Table A.12 (Appendix A). The designer should choose a group of bars from the table with an area equal to or greater than the area obtained from calculations. Also, the design equations in this book based on the ACI Code are approximate. Therefore, for a practical and economical design, it is adequate to use four figures (or the full number with no fractions if it is greater than four figures) for the calculation of forces, stresses, moments, or dimensions such as length or width of section. For strains, use five or six figures because strains are very small quantities measured in a millionth of an inch (for example, a strain of 0.000358 in./in.). Stresses are obtained by multiplying the strains by the modulus of elasticity of the material, which has a high magnitude (for example, 29,000,000 lb/in.^{2}) for steel. Any figures less than five or six figures in strains will produce quite a change in stresses.

Examples

For forces, use 28.45 K, 2845 lb, 567.8 K (four figures).
For force/length, use 2.451 K/ft or 2451 lb/ft.
For length or width, use 14.63 in., 1.219 ft (or 1.22 ft).
For areas, use 7.537 in.^{2}, and for volumes, use 48.72 in.^{3}.
For strains, use 0.002078.

1.12 CONCRETE HIGH-RISE BUILDINGS

High-rise buildings are becoming the dominant feature of many U.S. cities; a great number of these buildings are designed and constructed in structural concrete.

Although at the beginning of the century the properties of concrete and joint behavior of steel and concrete were not fully understood, a 16-story building, the Ingalls Building, was constructed in Cincinnati in 1902 with a total height of 210 ft (64 m). In 1922, the Medical Arts Building, with a height of 230 ft (70 m), was constructed in Dallas, Texas. The design of concrete buildings was based on elastic theory concepts and a high factor of safety, resulting in large concrete sections in beams and columns. After extensive research, high-strength concrete and high-strength steel were allowed in the design of reinforced concrete members. Consequently, small concrete sections as well as savings in materials were achieved, and new concepts of structural design were possible.

To visualize how high concrete buildings can be built, some structural concrete skyscrapers are listed in Table 1.3. The CN Tower is the world's tallest free-standing concrete structure.

The reader should realize that most concrete buildings are relatively low and range from one to five stories. Skyscrapers and high-rise buildings constitute less than 10% of all concrete buildings.

Photos of some different concrete buildings and structures are shown.



Renaissance Center, Detroit, Michigan.



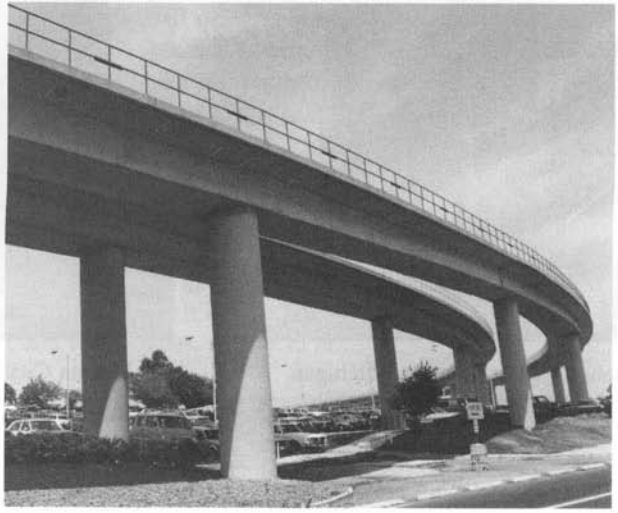
Marina City Towers, Chicago, Illinois.



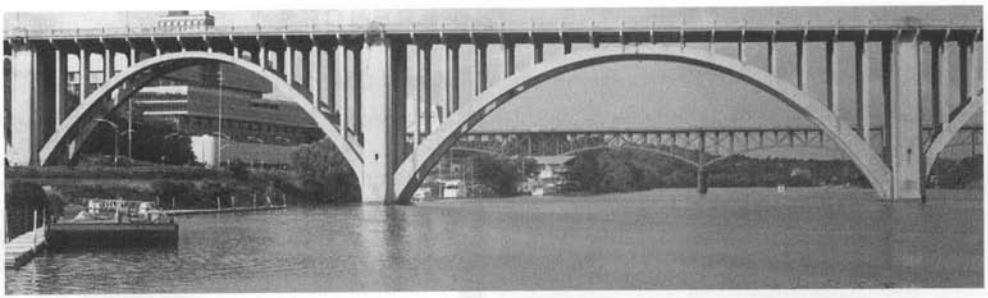
City Center, Minneapolis, Minnesota.



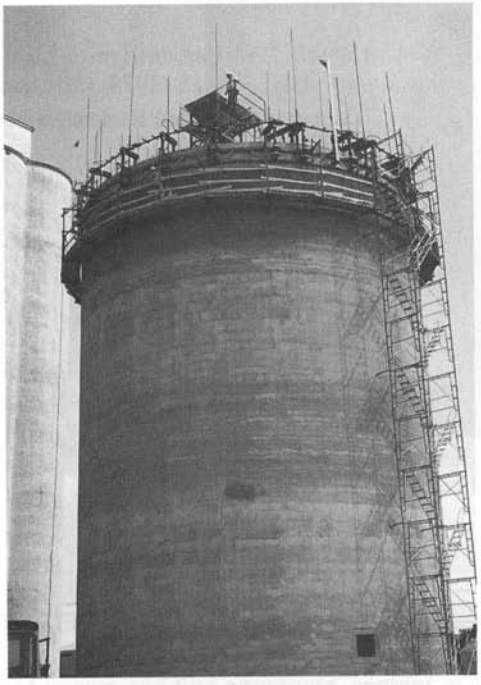
CN Tower, Toronto, Canada
(height 1465 ft, or 447 m).



Concrete bridge for the city transit system, Washington, DC.



Concrete bridge, Knoxville, Tennessee.



Reinforced concrete grain silo using the slip form system, Brookings, South Dakota.

Table 1.3 Examples of Reinforced Concrete Skyscrapers

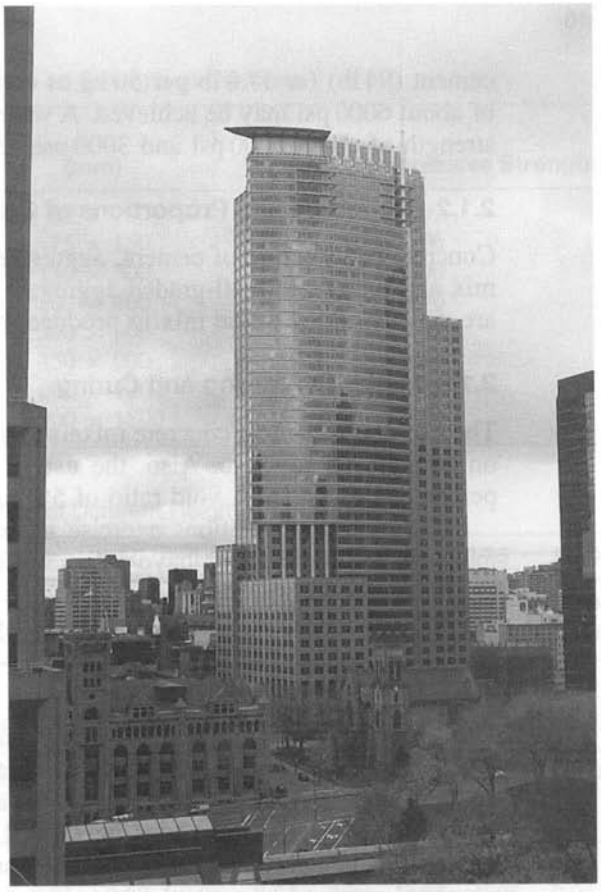
Year	Structure	Location	Stories	Height, ft (m)
1965	Lake Point Tower	Chicago	70	645 (197)
1969	One Shell Plaza	Houston	52	714 (218)
1975	Peachtree Center Plaza Hotel	Atlanta	71	723 (220)
1976	Water Tower Place	Chicago	74	859 (262)
1976	CN Tower	Toronto	—	1465 (447)
1977	Renaissance Center Westin Hotel	Detroit	73	740 (226)
1983	City Center	Minneapolis	40	528 (158)

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CHAPTER 2

PROPERTIES OF REINFORCED CONCRETE



IBM Building, Montreal, Canada (the highest concrete building in Montreal, with 50 stories).

2.1 FACTORS AFFECTING THE STRENGTH OF CONCRETE

In general, concrete consists of coarse and fine aggregate, cement, water, and—in many cases—different type of admixture. The materials are mixed together until a cement paste is developed, filling most of the voids in the aggregates and producing a uniform dense concrete. The plastic concrete is then placed in a mold and left to set, harden, and develop adequate strength. For the design of concrete mixtures, as well as composition and properties of concrete materials, the reader is referred to Refs. 1–6.

The strength of concrete depends upon many factors and may vary within wide limits with the same production method. The main factors that affect the strength of concrete are described next.

2.1.1 Water–Cement Ratio

The water–cement ratio is one of the most important factors affecting the strength of concrete. For complete hydration of a given amount of cement, a water–cement ratio (by weight) equal to 0.25 is needed. A water–cement ratio of about 0.35 or higher is needed for the concrete to be reasonably workable without additives. This ratio corresponds to 4 gal of water per sack of

cement (94 lb) (or 17.8 lb per 50 kg of cement). Based on this cement ratio, a concrete strength of about 6000 psi may be achieved. A water–cement ratio of 0.5 and 0.7 may produce a concrete strength of about 5000 psi and 3000 psi, respectively.

2.1.2 Properties and Proportions of Concrete Constituents

Concrete is a mixture of cement, aggregate, and water. An increase in the cement content in the mix and the use of well-graded aggregate increase the strength of concrete. Special admixtures are usually added to the mix to produce the desired quality and strength of concrete.

2.1.3 Method of Mixing and Curing

The use of mechanical concrete mixers and the proper time of mixing both have favorable effects on strength of concrete. Also, the use of vibrators produces dense concrete with a minimum percentage of voids. A void ratio of 5% may reduce the concrete strength by about 30%.

The curing conditions exercise an important influence on the strength of concrete. Both moisture and temperature have a direct effect on the hydration of cement. The longer the period of moist storage, the greater the strength. If the curing temperature is higher than the initial temperature of casting, the resulting 28-day strength of concrete is reached earlier than 28 days.

2.1.4 Age of the Concrete

The strength of concrete increases appreciably with age, and hydration of cement continues for months. In practice, the strength of concrete is determined from cylinders or cubes tested at the age of 7 days and 28 days. As a practical assumption, concrete at 28 days is 1.5 times as strong as at 7 days: The range varies between 1.3 and 1.7. The British code of practice [2] accepts concrete if the strength at 7 days is not less than two-thirds of the required 28-day strength. For a normal portland cement, the increase of strength with time, relative to 28-day strength, may be assumed as follows:

Age	7 days	14 days	28 days	3 months	6 months	1 year	2 years	5 years
Strength ratio	0.67	0.86	1.0	1.17	1.23	1.27	1.31	1.35

2.1.5 Loading Conditions

The compressive strength of concrete is estimated by testing a cylinder or cube to failure in a few minutes. Under sustained loads for years, the ultimate strength of concrete is reduced by about 30%. Under 1-day sustained loading, concrete may lose about 10% of its compressive strength. Sustained loads and creep effect as well as dynamic and impact effect, if they occur on the structure, should be considered in the design of reinforced concrete members.

2.1.6 Shape and Dimensions of the Tested Specimen

The common sizes of concrete specimens used to predict the compressive strength are either 6-by 12-in. (150- by 300-mm) cylinders or 6-in. (150-mm) cubes. When a given concrete is tested in compression by means of cylinders of like shape but of different sizes, the larger specimens give lower strength indexes. Table 2.1 [4] gives the relative strength for various sizes of cylinders as

Table 2.1 Effect of Size of Compression Specimen on Strength of Concrete

Size of cylinder (in.)	Size of cylinder (mm)	Relative Compressive Strength
2 × 4	50 × 100	1.09
3 × 6	75 × 150	1.06
6 × 12	150 × 300	1.00
8 × 16	200 × 400	0.96
12 × 24	300 × 600	0.91
18 × 36	450 × 900	0.86
24 × 48	600 × 1200	0.84
36 × 72	900 × 1800	0.82

Table 2.2 Strength Correction Factor for Cylinders of Different Height–Diameter Ratios

Ratio	2.0	1.75	1.50	1.25	1.10	1.00	0.75	0.50
Strength correction factor	1.00	0.98	0.96	0.93	0.90	0.87	0.70	0.50
Strength relative to standard cylinder	1.00	1.02	1.04	1.06	1.11	1.18	1.43	2.00

Table 2.3 Relative Strength of Cylinder versus Cube [6]

	(psi)	1000	2200	2900	3500	3800	4900	5300	5900	6400	7300
Compressive Strength	**(N/mm^2)**	7.0	15.5	20.0	24.5	27.0	24.5	37.0	41.5	45.0	51.5
Strength Ratio of Cylinder to Cube		0.77	0.76	0.81	0.87	0.91	0.93	0.94	0.95	0.96	0.96

a percentage of the strength of the standard cylinder; the heights of all cylinders are twice the diameters.

Sometimes concrete cylinders of nonstandard shape are tested. The greater the ratio of specimen height to diameter, the lower the strength indicated by the compression test. To compute the equivalent strength of the standard shape, the results must be multiplied by a correction factor. Approximate values of the correction factor are given in Table 2.2, extracted from ASTM C 42/C 42 M-03. The relative strengths of a cylinder and a cube for different compressive strengths are shown in Table 2.3.

2.2 COMPRESSIVE STRENGTH

In designing structural members, it is assumed that the concrete resists compressive stresses and not tensile stresses; therefore, compressive strength is the criterion of quality concrete. The other concrete stresses can be taken as a percentage of the compressive strength, which can be easily and accurately determined from tests. Specimens used to determine compressive strength may be cylindrical, cubical, or prismatic.

Test specimens in the form of a 6-in. (150 mm) or 8-in. (200 mm) cube are used in Great Britain, Germany, and other parts of Europe.

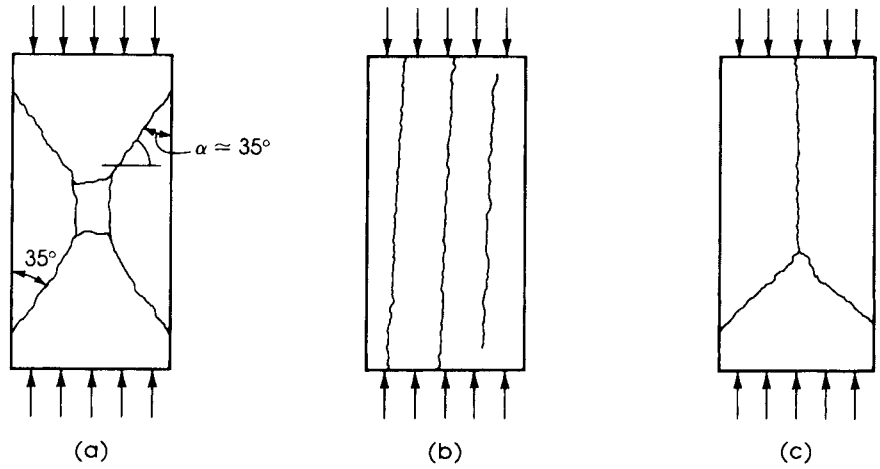


Figure 2.1 Modes of failure of standard concrete cylinders.

Prism specimens are used in France, Russia, and other countries and are usually 70 by 70 by 350 mm or 100 by 100 by 500 mm. They are cast with their longer sides horizontal and are tested, like cubes, in a position normal to the position of cast.

Before testing, the specimens are moist-cured and then tested at the age of 28 days by gradually applying a static load until rupture occurs. The rupture of the concrete specimen may be caused by the applied tensile stress (failure in cohesion), the applied shearing stress (sliding failure), the compressive stress (crushing failure), or combinations of these stresses.

The failure of the concrete specimen can be in one of three modes [5], as shown in Fig. 2.1. First, under axial compression, the specimen may fail in shear, as in Fig. 2.1*a*. Resistance to failure is due to both cohesion and internal friction.

The second type of failure (Fig. 2.1*b*) results in the separation of the specimen into columnar pieces by what is known as splitting, or columnar, fracture. This failure occurs when the strength of concrete is high, and lateral expansion at the end bearing surfaces is relatively unrestrained.

The third type of failure (Fig. 2.1*c*) is seen when a combination of shear and splitting failure occurs.

2.3 STRESS–STRAIN CURVES OF CONCRETE

The performance of a reinforced concrete member under load depends, to a great extent, on the stress–strain relationship of concrete and steel and on the type of stress applied to the member. Stress–strain curves for concrete are obtained by testing a concrete cylinder to rupture at the age of 28 days and recording the strains at different load increments.

Figure 2.2 shows typical stress–strain curves for concretes of different strengths. All curves consist of an initial relatively straight elastic portion, reaching maximum stress at a strain of about 0.002; then rupture occurs at a strain of about 0.003. Concrete having a compressive strength between 3000 and 6000 psi (21 and 42 N/mm^2) may be adopted. High-strength concrete with a compressive strength greater than 6000 psi (6000–15,000 psi) is becoming an important building material for the design of concrete structures.

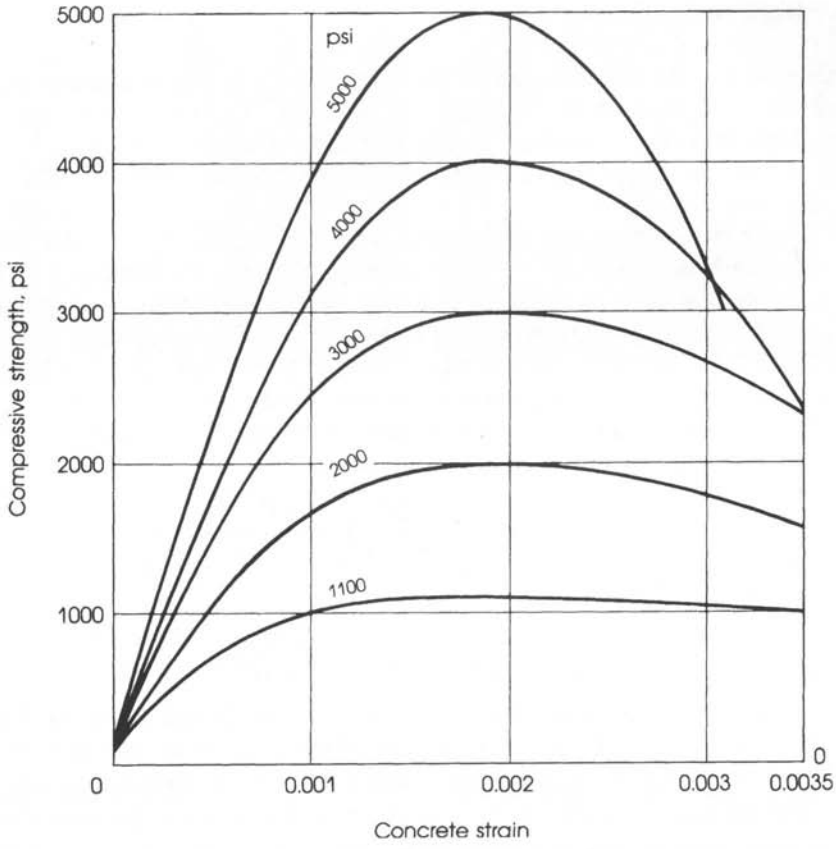


Figure 2.2 Typical stress–strain curves of concrete.



Standard capped cylinders ready for testing.

2.4 TENSILE STRENGTH OF CONCRETE

Concrete is a brittle material, and it cannot resist the high tensile stresses that are important when considering cracking, shear, and torsional problems. The low tensile capacity can be attributed to the high stress concentrations in concrete under load, so that a very high stress is reached in some portions of the specimen, causing microscopic cracks, while the other parts of the specimen are subjected to low stress.

Direct tension tests are not reliable for predicting the tensile strength of concrete, due to minor misalignment and stress concentrations in the gripping devices. An indirect tension test in the form of splitting a 6- by 12-in. (150- by 300-mm) cylinder was suggested by the Brazilian Fernando Carneiro. The test is usually called the *splitting test*. In this test, the concrete cylinder is placed with its axis horizontal in a compression testing machine. The load is applied uniformly along two opposite lines on the surface of the cylinder through two plywood pads, as shown in Fig. 2.3. Considering an element on the vertical diameter and at a distance y from the top fibers, the element is subjected to a compressive stress

$$f_c = \frac{2P}{\pi LD}\left(\frac{D^2}{y(D-y)} - 1\right) \tag{2.1}$$

and a tensile stress

$$f'_{sp} = \frac{2P}{\pi LD} \tag{2.2}$$

where P is the compressive load on the cylinder and D and L are the diameter and length of the cylinder. For a 6- by 12-in. (150- by 300-mm) cylinder and at a distance $y = D/2$, the compression strength is $f_e = 0.0265P$, and the tensile strength is $f'_{sp} = 0.0088P = f_c/3$.

The splitting strength of f'_{sp} can be related to the compressive strength of concrete in that it varies between six and seven times $\sqrt{f'_c}$ for normal concrete and between four and five times $\sqrt{f'_c}$ for lightweight concrete. The direct tensile stress, f'_t, can also be estimated from the

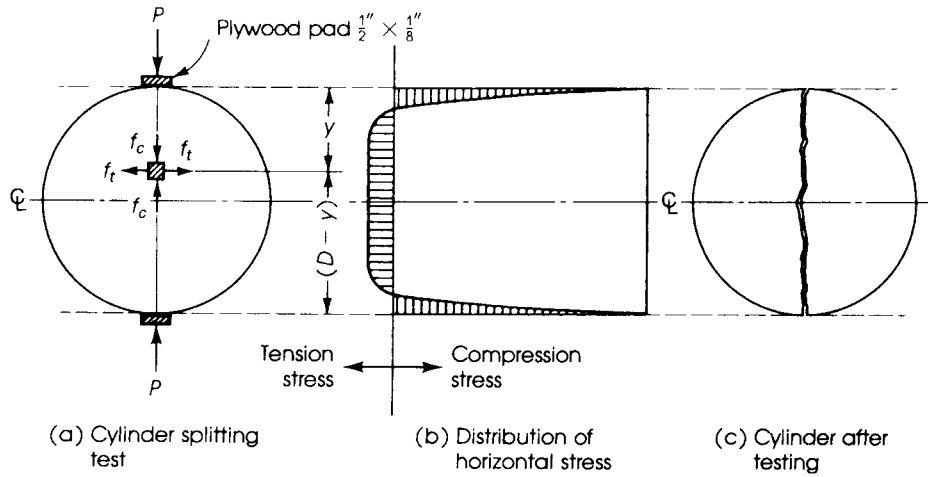
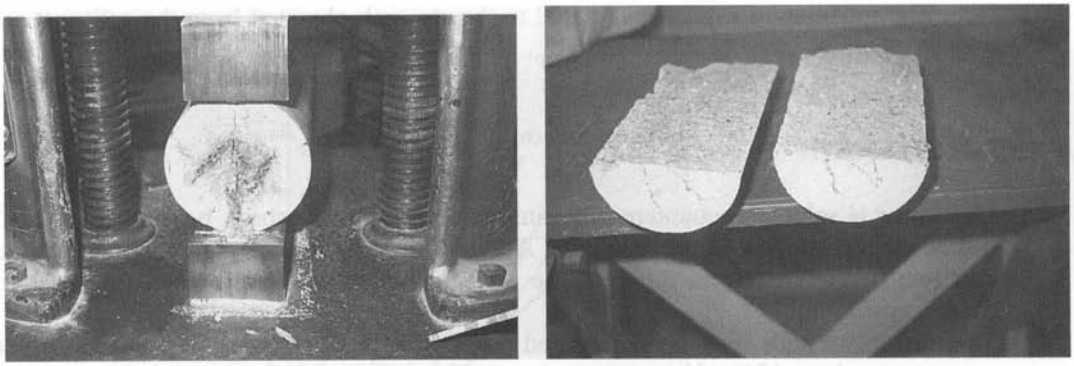


Figure 2.3 Cylinder splitting test [6]: (*a*) configuration of test, (*b*) distribution of horizontal stress, and (*c*) cylinder after testing.



Concrete cylinder splitting test.

split test: Its value varies between 0.5 f'_{sp} and 0.7 f'_{sp}. The smaller of these values applies to higher-strength concrete. The splitting strength, f'_{sp}, can be estimated as 10% of the compressive strength up to $f'_c = 6000$ psi (42 N/mm^2). For higher values of compressive strength, f'_{sp} can be taken as 9% of f'_c.

In general, the tensile strength of concrete ranges from 7% to 11% of its compressive strength, with an average of 10%. The lower the compressive strength, the higher the relative tensile strength.

2.5 FLEXURAL STRENGTH (MODULUS OF RUPTURE) OF CONCRETE

Experiments on concrete beams have shown that ultimate tensile strength in bending is greater than the tensile stress obtained by direct or splitting tests. Flexural strength is expressed in terms of the modulus of rupture of concrete (f_r), which is the maximum tensile stress in concrete in bending. The modulus of rupture can be calculated from the flexural formula used for elastic materials, $f_r = Mc/I$, by testing a plain concrete beam. The beam, 6 by 6 by 28 in. (150 by 150 by 700 mm), is supported on a 24-in. (600-mm) span and loaded to rupture by two loads, 4 in. (100 mm) on either side of the center. A smaller beam of 4 by 4 by 20 in. (100 by 100 by 500 mm) on a 16-in. (400-mm) span may also be used.

The modulus of rupture of concrete ranges between 11% and 23% of the compressive strength. A value of 15% can be assumed for strengths of about 4000 psi (28 N/mm^2). The ACI Code prescribes the value of the modulus of rupture as

$$f_r = 7.5\lambda\sqrt{f'_c}\ \text{(psi)} = 0.62\lambda\sqrt{f'_c}\ \text{(N/mm}^2\text{)} \tag{2.3}$$

where

λ is a modification factor for type of concrete (ACI 8.6.1)

$\lambda = 1.0$ Normal-weight concrete

$\lambda = 0.85$ Sand-lightweight concrete

$\lambda = 0.75$ for all-lightweight concrete

Linear interpolation shall be permitted between 0.85 and 1.0 on the basis of volumetric fractions, for concrete containing normal-weight fine aggregate and a blend of lightweight and normal-weight coarse aggregate.

The modulus of rupture as related to the strength obtained from the split test on cylinders may be taken as $f_r = (1.25 \text{ to } 1.50) f'_{sp}$.

2.6 SHEAR STRENGTH

Pure shear is seldom encountered in reinforced concrete members, because it is usually accompanied by the action of normal forces. An element subjected to pure shear breaks transversely into two parts. Therefore, the concrete element must be strong enough to resist the applied shear forces.

Shear strength may be considered as 20% to 30% greater than the tensile strength of concrete, or about 12% of its compressive strength. The ACI Code allows a nominal shear stress of $2\lambda\sqrt{f'_c}$ psi ($0.17\lambda\sqrt{f'_c}$ N/mm^2) on plain concrete sections. For more information, refer to Chapter 8.

2.7 MODULUS OF ELASTICITY OF CONCRETE

One of the most important elastic properties of concrete is its modulus of elasticity, which can be obtained from a compressive test on concrete cylinders. The modulus of elasticity, E_c, can be defined as the change of stress with respect to strain in the elastic range:

$$E_c = \frac{\text{unit stress}}{\text{unit strain}} \tag{2.4}$$

The modulus of elasticity is a measure of stiffness, or the resistance of the material to deformation. In concrete, as in any elastoplastic material, the stress is not proportional to the strain, and the stress–strain relationship is a curved line. The actual stress–strain curve of concrete can be obtained by measuring the strains under increments of loading on a standard cylinder.

The *initial tangent modulus* (Fig. 2.4) is represented by the slope of the tangent to the curve at the origin under elastic deformation. This modulus is of limited value and cannot be determined with accuracy. Geometrically, the tangent modulus of elasticity of concrete, E_c, is the slope of the tangent to the stress–strain curve at a given stress. Under long-time action of load and due to the development of plastic deformation, the stress-to-total-strain ratio becomes a variable nonlinear quantity.

For practical applications, the *secant modulus* can be used. The secant modulus is represented by the slope of a line drawn from the origin to a specific point of stress (B) on the stress–strain curve (Fig. 2.4). Point B is normally located at $f'_c/2$.

The ACI Code section 8.5.1 gives a simple formula for calculating the modulus of elasticity of normal and lightweight concrete considering the secant modulus at a level of stress, f_c, equal to half the ultimate concrete strength, f'_c,

$$E_c = 33w^{1.5}\sqrt{f'_c} \text{ psi}(w \text{ in pcf}) = 0.043\, w^{1.5}\sqrt{f'_c} \text{ N/mm}^2 \tag{2.5}$$

where w = unit weight of concrete (between 90 and 160 lb/ft^3 (pcf) or 1400 to 2600 kg/m^3) and f'_c = ultimate strength of a standard concrete cylinder. For normal-weight concrete, w is approximately 145 pcf (2320 kg/m^3); thus,

$$E_c = 57{,}600\sqrt{f'_c} \text{ psi} = 4780\sqrt{f'_c} \text{ MPa} \tag{2.6}$$

$$E_c = \frac{df_c}{d\varepsilon_c}$$

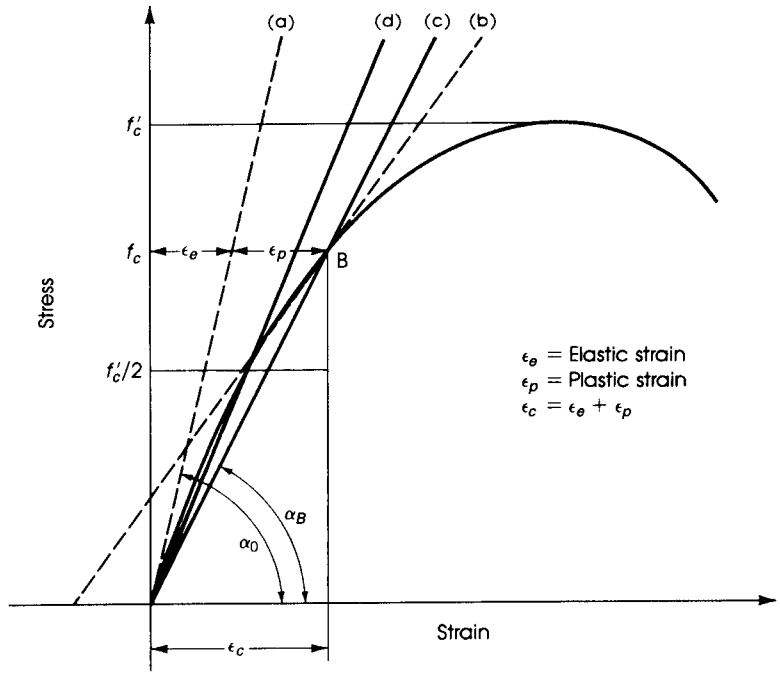


Figure 2.4 Stress–strain curve and modulus of elasticity of concrete. Lines a-d represent (*a*) initial tangent modulus, (*b*) tangent modulus at a stress, f_c, (*c*) secant modulus at a stress, f_c, and (*d*) secant modulus at a stress $f'_c/2$.

The ACI Code allows the use of $E_c = 57{,}000\sqrt{f'_c}$ (psi) $= 4700\sqrt{f'_c}$ MPa. The module of elasticity, E_c, for different values of f'_c are shown in Table A.10.

2.8 POISSON'S RATIO

Poisson's ratio, μ, is the ratio of the transverse to the longitudinal strains under axial stress within the elastic range. This ratio varies between 0.15 and 0.20 for both normal and lightweight concrete. Poisson's ratio is used in structural analysis of flat slabs, tunnels, tanks, arch dams, and other statically indeterminate structures. For isotropic elastic materials, Poisson's ratio is equal to 0.25. An average value of 0.18 can be used for concrete.

2.9 SHEAR MODULUS

The modulus of elasticity of concrete in shear ranges from about 0.4 to 0.6 of the corresponding modulus in compression. From the theory of elasticity, the shear modulus is taken as follows:

$$G_c = \frac{E_c}{2(1+\mu)} \tag{2.7}$$

where μ = Poisson's ratio of concrete. If μ is taken equal to $1/6$, then $G_c = 0.43E_c = 24{,}500\sqrt{f'_c}$.



Test on a standard concrete cylinder to determine the modulus of elasticity of concrete.

2.10 MODULAR RATIO

The modular ratio, n, is the ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete: $n = E_s/E_c$.

Because the modulus of elasticity of steel is considered constant and is equal to 29×10^6 psi and $E_c = 33w^{1.5}\sqrt{f'_c}$,

$$n = \frac{29 \times 10^6}{33w^{1.5}\sqrt{f'_c}} \tag{2.8}$$

For normal-weight concrete, $E_c = 57{,}400\sqrt{f'_c}$; hence, n can be taken as

$$n = \frac{500}{\sqrt{f'_c}}(f'_c \text{ in psi}) = \frac{42}{\sqrt{f'_c}}(f'_c \text{ in N/mm}^2) \tag{2.9}$$

The significance and the use of the modular ratio are explained in Chapter 6.

2.11 VOLUME CHANGES OF CONCRETE

Concrete undergoes volume changes during hardening. If it loses moisture by evaporation, it shrinks, but if the concrete hardens in water, it expands. The causes of the volume changes in concrete can be attributed to changes in moisture content, chemical reaction of the cement with water, variation in temperature, and applied loads.

2.11.1 Shrinkage

The change in the volume of drying concrete is not equal to the volume of water removed [6]. The evaporation of free water causes little or no shrinkage. As concrete continues to dry, water evaporates and the volume of the restrained cement paste changes, causing concrete to shrink, probably due to the capillary tension that develops in the water remaining in concrete. Emptying of the capillaries causes a loss of water without shrinkage, but once the absorbed water is removed, shrinkage occurs.

Many factors influence the shrinkage of concrete caused by the variations in moisture conditions [5]:

1. *Cement and water content*. The more cement or water content in the concrete mix, the greater the shrinkage.
2. *Composition and fineness of cement*. High-early-strength and low-heat cements show more shrinkage than normal portland cement. The finer the cement, the greater the expansion under moist conditions.
3. *Type, amount, and gradation of aggregate*. The smaller the size of aggregate particles, the greater the shrinkage. The greater the aggregate content, the smaller the shrinkage [7].
4. *Ambient conditions, moisture, and temperature*. Concrete specimens subjected to moist conditions undergo an expansion of 200 to 300 $\times$ 10^{-6}, but if they are left to dry in air, they shrink. High temperature speeds the evaporation of water and, consequently, increases shrinkage.
5. *Admixtures*. Admixtures that increase the water requirement of concrete increase the shrinkage value.
6. *Size and shape of specimen*. As shrinkage takes place in a reinforced concrete member, tension stresses develop in the concrete, and equal compressive stresses develop in the steel. These stresses are added to those developed by the loading action. Therefore, cracks may develop in concrete when a high percentage of steel is used. Proper distribution of reinforcement, by producing better distribution of tensile stresses in concrete, can reduce differential internal stresses.

The values of final shrinkage for ordinary concrete vary between 200 and 700 $\times$ 10^{-6}. For normal-weight concrete, a value of 300 $\times$ 10^{-6} may be used. The British Code [8] gives a value of 500 $\times$ 10^{-6}, which represents an unrestrained shrinkage of 1.5 mm in a 3 m length of thin, plain concrete sections. If the member is restrained, a tensile stress of about 10 N/mm^2 (1400 psi) arises. If concrete is kept moist for a certain period after setting, shrinkage is reduced; therefore, it is important to cure the concrete for a period of no fewer than 7 days.

Exposure of concrete to wind increases the shrinkage rate on the upwind side. Shrinkage causes an increase in the deflection of structural members, which in turn increases with time. Symmetrical reinforcement in the concrete section may prevent curvature and deflection due to shrinkage.

Generally, concrete shrinks at a high rate during the initial period of hardening, but at later stages the rate diminishes gradually. It can be said that 15% to 30% of the shrinkage value occurs in 2 weeks, 40% to 80% occurs in 1 month, and 70% to 85% occurs in 1 year.

2.11.2 Expansion Due to Rise in Temperature

Concrete expands with increasing temperature and contracts with decreasing temperature. The coefficient of thermal expansion of concrete varies between 4 and 7 $\times$ 10^{-6} per degree Fahrenheit.

An average value of 5.5×10^{-6} per degree Fahrenheit (12×10^{-6} per degree Celsius) can be used for ordinary concrete. The British code [8] suggests a value of 10^{-5} per degree Celsius. This value represents a change of length of 10 mm in a 30-m member subjected to a change in temperature of 33°C. If the member is restrained and unreinforced, a stress of about 7 N/mm^2 (1000 psi) may develop.

In long reinforced concrete structures, expansion joints must be provided at lengths of 100 to 200 ft (30 to 60 m). The width of the expansion joint is about 1 in. (25 mm). Concrete is not a good conductor of heat, whereas steel is a good one. The ability of concrete to carry load is not much affected by temperature.

2.12 CREEP

Concrete is an elastoplastic material, and beginning with small stresses, plastic strains develop in addition to elastic ones. Under sustained load, plastic deformation continues to develop over a period that may last for years. Such deformation increases at a high rate during the first 4 months after application of the load. This slow plastic deformation under constant stress is called *creep*.

Figure 2.5 shows a concrete cylinder that is loaded. The instantaneous deformation is ε_1, which is equal to the stress divided by the modulus of elasticity. If the same stress is kept for a period of time, an additional strain, ε_2, due to creep effect, can be recorded. If load is then released, the elastic strain, ε_1, will be recovered, in addition to some creep strain. The final permanent plastic strain, ε_3, will be left, as shown in Fig. 2.5. In this case, $\varepsilon_3 = (1 - \alpha)\varepsilon_2$, where α is the ratio of the recovered creep strain to the total creep strain. The ratio α ranges between 0.1 and 0.2. The magnitude of creep recovery varies with the previous creep and depends appreciably upon the period of the sustained load. Creep recovery rate will be less if the loading period is increased, probably due to the hardening of concrete while in a deformed condition.

The ultimate magnitude of creep varies between 0.2×10^{-6} and 2×10^{-6} per unit stress (lb/in.^{2}) per unit length. A value of 1×10^{-6} can be used in practice. The ratio of creep strain to elastic strain may be as high as 4.

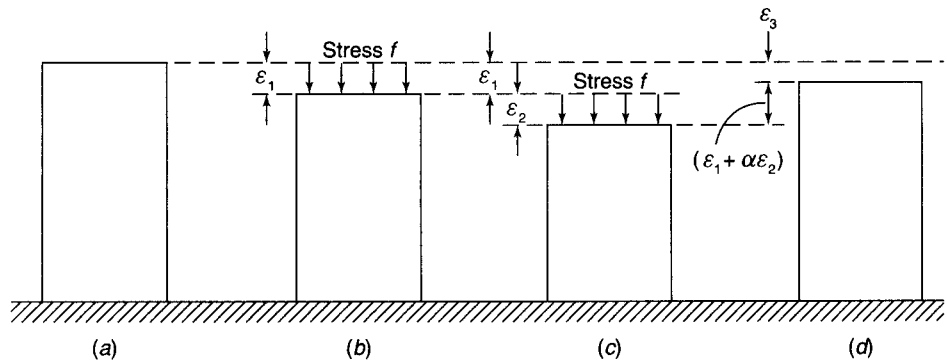


Figure 2.5 Deformation in a loaded concrete cylinder: (*a*) specimen unloaded, (*b*) elastic deformation, (*c*) elastic plus creep deformation, (*d*) permanent deformation after release of load.

Creep takes place in the hardened cement matrix around the strong aggregate. It may be attributed to slippage along planes within the crystal lattice, internal stresses caused by changes in the crystal lattice, and gradual loss of water from the cement gel in the concrete.

The different factors that affect the creep of concrete can be summarized as follows [9]:

1. *Level of stress*. Creep increases with an increase of stress in specimens made from concrete of the same strength and with the same duration of load.
2. *Duration of loading*. Creep increases with the loading period. About 80% of the creep occurs within the first 4 months; 90% occurs after about 2 years.
3. *Strength and age of concrete*. Creep tends to be smaller if concrete is loaded at a late age. Also, creep of 2000 psi ($14\,N/mm^2$)–strength concrete is about 1.41×10^{-6}, whereas that of 4000 psi ($28\,N/mm^2$)–strength concrete is about 0.8×10^{-6} per unit stress and length of time.
4. *Ambient conditions*. Creep is reduced with an increase in the humidity of the ambient air.
5. *Rate of loading*. Creep increases with an increase in the rate of loading when followed by prolonged loading.
6. *Percentage and distribution of steel reinforcement in a reinforced concrete member*. Creep tends to be smaller for higher proportion or better distribution of steel.
7. *Size of the concrete mass*. Creep decreases with an increase in the size of the tested specimen.
8. *Type, fineness, and content of cement*. The amount of cement greatly affects the final creep of concrete, as cement creeps about 15 times as much as concrete.
9. *Water–cement ratio*. Creep increases with an increase in the water–cement ratio.
10. *Type and grading of aggregate*. Well-graded aggregate will produce dense concrete and consequently a reduction in creep.
11. *Type of curing*. High-temperature steam curing of concrete, as well as the proper use of a plasticizer, will reduce the amount of creep.

Creep develops not only in compression, but also in tension, bending, and torsion.

The ratio of the rate of creep in tension to that in compression will be greater than 1 in the first 2 weeks, but this ratio decreases over longer periods [5].

Creep in concrete under compression has been tested by many investigators. Troxell, Raphale, and Davis [10] measured creep strains periodically for up to 20 years and estimated that of the total creep after 20 years, 18% to 35% occurred in 2 weeks, 30% to 70% occurred in 3 months, and 64% to 83% occurred in 1 year.

For normal concrete loaded after 28 days, $C_r = 0.13\sqrt[3]{t}$, where C_r = creep strain per unit stress per unit length. Creep augments the deflection of reinforced concrete beams appreciably with time. In the design of reinforced concrete members, long-term deflection may be critical and has to be considered in proper design. Extensive deformation may influence the stability of the structure.

Sustained loads affect the strength as well as the deformation of concrete. A reduction of up to 30% of the strength of unreinforced concrete may be expected when concrete is subjected to a concentric sustained load for 1 year.

The fatigue strength of concrete is much smaller than its static strength. Repeated loading and unloading cycles in compression lead to a gradual accumulation of plastic deformations. If concrete in compression is subjected to about 2 million cycles, its fatigue limit is about 50% to

60% of the static compression strength. In beams, the fatigue limit of concrete is about 55% of its static strength [11].

2.13 MODELS FOR PREDICTING THE SHRINKAGE AND CREEP OF CONCRETE

2.13.1 The ACI 209 Model

The American Concrete Institute recommend the ACI 209 model [12]. Branson and Christianson [13] first developed this model in 1970. The ACI 209 model was used for many years in the design of concrete structures. The model is simple to use but limited in its accuracy.

Shrinkage calculation. Calculation of shrinkage using the ACI 209 model can be performed if the following parameters and conditions are known: curing method (moist-cured or steam-cured concrete), relative humidity, H, type of cement, specimen shape, ultimate shrinkage strain, ε_{shu}, age of concrete after casting, t, age of the concrete drying commenced, usually taken as the age at the end of moist curing, t_c.

The shrinkage strain is defined as follows:

$$\varepsilon_s(t) = \frac{(t - t_c)}{b + (t - t_c)} K_{ss} K_{sh} \varepsilon_{shu} \tag{2.10}$$

where

t = Age of concrete after casting (days)
t_c = Age of the concrete drying commenced (days)
b = Constant in determining shrinkage strain, depends on curing method according to Table 2.4
K_{ss} = Shape and size correction factor for shrinkage according to the Eq. 2.11
K_{sh} = Relative humidity correction factor for shrinkage according to Eq. 2.12
ε_{shu} (ultimate shrinkage strain) 780×10^{-6} (mm/mm) (for both moist- and steam-cured concrete)

Shape and size correction factor for shrinkage should be calculated as follows:

$$K_{ss} = 1.14 - 0.0035\left(\frac{V}{S}\right) \tag{2.11}$$

where

V = volume of the specimen (mm^3)
S = surface of the specimen (mm^2)

Relative humidity correction factor for shrinkage is

$$K_{sh} = \begin{cases} 1.40 - 0.01H & \text{for } 40\% \le H \le 80\% \\ 3.00 - 0.03H & \text{for } 81\% \le H \le 100\% \end{cases} \tag{2.12}$$

Table 2.4 Values of Constant b as a Function of Curing Method

Moist-Cured Concrete	Steam-Cured Concrete
$b = 35$	$b = 55$

where

H = Relative humidity (in %)

Creep calculation. The total-load dependent strain at time t, ε_{ic} (t, t_0) of a concrete member uniaxially loaded at time t_0 with a constant stress σ may be calculated as follows:

$$\varepsilon_{ic}(t, t_0) = \varepsilon_i(t_0) + \varepsilon_c(t, t_0) \tag{2.13}$$

where

$\varepsilon_i(t_0)$ = The initial elastic strain at loading
$\varepsilon_c(t, t_0)$ = The creep strain at time $t \geq t_0$.

$$\varepsilon_i(t_0) = \frac{\sigma}{E_{\text{cmt}_0}} \tag{2.14}$$

$$\varepsilon_c(t, t_0) = \frac{\sigma}{E_{\text{cmt}_0}} C_c(t) \tag{2.15}$$

where

E_{cmt_0} = Modulus of elasticity at age of loading (MPa) as given in Eq. 2.17
$C_c(t)$ = Creep coefficient at time t, as given in Eq. 2.19

Usually, the total-load dependent strain is presented with compliance function, also called creep function, $J(t, t_0)$, which represent the total-load dependent strain at time t produced by a unit constant stress that has been acting since time t_0.

$$J(t, t_0) = \frac{1 + C_c(t)}{E_{\text{cmt}_0}} \tag{2.16}$$

$$E_{\text{cmt}_0} = 0.043(\gamma)^{3/2}\sqrt{f'_c(t_0)} \tag{2.17}$$

where

γ = Concrete unit weight (kg/m^3)
$f'_c(t_0)$ = Mean concrete compressive strength at age of loading (MPa)

$$f'_c(t_0) = f_{\text{cm}_{28}} \frac{t_0}{b + ct_0} \tag{2.18}$$

where

$f_{\text{cm}_{28}}$ = Average 28-day concrete compressive strength (MPa)
b and $c$are constants according to Table 2.5:

Table 2.5 Constants b and c as a Function of Cement Type and Curing Method

Type of Cement	Moist-Cured Concrete		Steam-Cured Concrete	
I	$b = 4$	$c = 0.85$	$b = 1$	$c = 0.95$
III	$b = 2.30$	$c = 0.92$	$b = 0.70$	$c = 0.98$

Table 2.6 Correction Factors

Curing Method	t_0 (days)	H	K_{ca}	K_{ch}	K_{cs}
Moist Cured	$\geq$ 1 day	$\geq$ 40%	N/A	N/A	N/A
	$\geq$ 7 days	$\geq$ 40%	$1.25(t_0)^{-0.118}$	$1.27 - 0.0067H$	$1.14 - 0.0035(V/S)$
Stem Cured	$\geq$ 1 day	$\geq$ 40%	$1.13(t_0)^{-0.095}$	$1.27 - 0.0067H$	$1.14 - 0.0035(V/S)$
	$\geq$ 7 days	$\geq$ 40%	N/A	N/A	N/A

Creep coefficient, $C_c(t)$, can be determined as follows:

$$C_c(t) = \frac{t^{0.60}}{10 + t^{0.60}} C_{cu} K_{ch} K_{ca} K_{cs} \tag{2.19}$$

where

C_{cu} = Ultimate creep coefficient = 2.35
K_{ch} = Relative humidity correction factor for creep determined from Table 2.6
K_{ca} = Age at loading correction factor determined from Table 2.6
K_{cs} = Shape and size correction factor for creep determined from Table 2.6

2.13.2 The B3 Model

The model was developed by Bazant and Baweja [14].

Shrinkage calculation. Required parameters for calculation of shrinkage strain using the B3 model are concrete mean compressive strength at 28 days, curing conditions, cement type, relative humidity, water content in concrete, and specimen shape.

The shrinkage strain can be estimated using the following equation:

$$\varepsilon_s(t) = (\varepsilon_{shu})(K_h)S(t) \tag{2.20}$$

where

ε_{shu} = Ultimate shrinkage strain according to Eq. 2.21
K_h = Humidity function for shrinkage according to Table 2.9
$S(t)$ = Time function for shrinkage according to Eq. 2.22

Ultimate shrinkage strain can be calculated using the following equation:

$$\varepsilon_{shu} = -\alpha_1 \alpha_2 [0.019(w)^{2.1} (f_{cm_{28}})^{-0.28} + 270] \times 10^{-6} \tag{2.21}$$

where

α_1 = Type of cement correction factor according to Table 2.7
α_2 = Curing condition correction factor according to the Table 2.8
w = Water content (kg/m^3)
$f_{cm_{28}}$ = Mean compressive concrete strength at 28 days (MPa)

Type of cement correction factor α_1 can be determined using Table 2.7.
Curing condition correction factor α_2 can be determined using Table 2.8.
Humidity function for shrinkage, K_h, should be determined according to Table 2.9.

Table 2.7 Correction Factor α_1 as a Function of Cement Type

Type of Cement	α_1
I	1.00
II	0.85
III	1.10

Table 2.8 Correction Factor α_2 as a Function of Type of Curing

Type of Curing	α_2
Steam cured	0.75
Water cured or $H = 100\%$	1.00
Sealed during curing	1.20

Table 2.9 Humidity Function for Shrinkage, K_h

Humidity	K_h
$H \leq 98\%$	$1 - (H/100)^3$
$H = 100\%$	-0.2
$98\% \leq H \leq 100\%$	Linear interpolation

where

H is relative humidity (%)

Time function for shrinkage, $S(t)$, should be calculated according to the following equation:

$$S(t) = \tanh\sqrt{\frac{t - t_c}{T_{\text{sh}}}} \tag{2.22}$$

where

t = Age of concrete after casting (days)
t_c = Age of the concrete drying commenced (days)
T_{sh} = Shrinkage half-time (days) according to the Eq. 2.23

$$T_{\text{sh}} = 0.085(t_c)^{-0.08}(f_{\text{cm}_{28}})^{-0.25}[2K_s(V/S)]^2 \tag{2.23}$$

where K_s = Cross-section shape correction factor according to Table 2.10
K_s can be assumed to be 1 if type of member is not defined.

Creep calculation. The creep function, also called creep compliance, $J(t, t_0)$ is given by Eq. 2.24:

$$J(t, t_0) = q_1 + C_0(t, t_0) + C_d(t, t_0, t_c) \tag{2.24}$$

Table 2.10 Correction Factor, K_s as a Function of Cross-Section Shape

Cross-Section Shape	K_s
Infinite slab	1.00
Infinite cylinder	1.15
Infinite square prism	1.25
Sphere	1.30
Cube	1.55

where

q_1 = The instantaneous strain, given in Eq. 2.25
$C_0(t, t_0)$ = The compliance function for basic creep composed of three terms, an aging viscoelastic term, a nonaging viscoelastic term and an aging flow term given in Eq. 2.27
$C_d(t, t_0, t_c)$ = The compliance function for drying creep, given in Eq. 2.35

$$q_1 = \frac{0.6}{E_{\text{cm}_{28}}} \tag{2.25}$$

where

$E_{\text{cm}_{28}}$ = Modulus of elasticity of concrete at 28 days as given in the following equation:

$$E_{\text{cm}_{28}} = 4735\sqrt{f_{\text{cm}_{28}}} \tag{2.26}$$

The compliance function for basic creep, $C_0(t, t_0)$, should be calculated as follows:

$$C_0(t, t_0) = q_2 Q(t, t_0) + q_3 \ln[1 + (t - t_0)^{0.1}] + q_4 \ln\left(\frac{t}{t_0}\right) \tag{2.27}$$

where

q_2 = Aging viscoelastic compliance parameter
$Q(t, t_0)$ = The binomial integral
q_3 = Nonaging viscoelastic compliance parameter
q_4 = Flow compliance parameter
t_0 = Age of concrete at loading (days)

$$q_2 = 185.4(c)^{0.5}(f_{\text{cm}_{28}})^{-0.9} \times 10^{-6} \tag{2.28}$$

where c is the cement content (kg/m^3).

$$Q(t, t_0) = Q_f(t_0)\left[1 + \frac{Q_f(t_0)^{r(t_0)}}{Z(t, t_0)^{r(t_0)}}\right]^{-1/r(t_0)} \tag{2.29}$$

where

$$Q_f(t_0) = \frac{1}{0.086(t_0)^{2/9} + 1.21(t_0)^{4/9}} \tag{2.30}$$

$$Z(t, t_0) = \frac{\ln[1 + (t - t_0)^{0.1}]}{\sqrt{t_0}} \tag{2.31}$$

$$r(t_0) = 1.7(t_0)^{0.12} + 8 \tag{2.32}$$

$$q_3 = 0.29 q_2 \left(\frac{w}{c}\right)^4 \tag{2.33}$$

$$q_4 = 20.3 \left(\frac{a}{c}\right)^{-0.7} \times 10^{-6} \tag{2.34}$$

The compliance function for drying creep, $C_d(t, t_0, t_c)$, should be calculated as follows:

$$C_d(t, t_0, t_c) = q_5 \sqrt{\exp[-8H(t)] - \exp[-8H(t_0)]} \tag{2.35}$$

where

q_5 = Drying creep compliance parameter that can be calculated from the following equation:

$$q_5 = \frac{0.757|\varepsilon_{shu} \times 10^6|^{-0.6}}{f_{cm_{28}}} \tag{2.36}$$

where

ε_{shu} = Ultimate shrinkage strain, given by Eq. 2.21

$H(t)$ and $H(t_0)$ are spatial averages of pore relative humidity.

$$H(t) = 1 - \left[\left(1 - \frac{H}{100}\right) S(t)\right] \tag{2.37}$$

$$H(t_0) = 1 - \left[\left(1 - \frac{H}{100}\right) S(t_0)\right] \tag{2.38}$$

$S(t)$ is given by Eq. 2.22 and

$$S(t_0) = \tanh\sqrt{\frac{t_0 - t_c}{T_{sh}}} \tag{2.39}$$

T_{sh} is given by Eq. 2.23.

2.13.3 The GL 2000 Model

The GL 2000 Model was developed by Gardner et. al and is described in Ref. 15.

Shrinkage calculation. Parameters required for calculation of shrinkage strain using the GL 2000 model are mean 28-day concrete compressive strength, $f_{cm_{28}}$, relative humidity, H, age of concrete at the beginning of shrinkage, t_c, type of cement, and specimen shape.

The shrinkage strain can be calculated using the following equation:

$$\varepsilon_s(t) = \varepsilon_{shu} \beta(h) \beta(t) \tag{2.40}$$

where

ε_{shu} = Ultimate shrinkage strain according to Eq. 2.41
$\beta(h)$ = Correction term for effect of humidity according to Eq. 2.42
$\beta(t)$ = Correction term for effect of time according to Eq. 2.43

Ultimate shrinkage strain should be calculated from the following equation:

$$\varepsilon_{shu} = (900)K\left(\frac{30}{f_{cm_{28}}}\right)^{1/2} \times 10^{-6} \tag{2.41}$$

where

K = Shrinkage constant, which depends on cement type as shown in Table 2.11
$f_{cm_{28}}$ = Mean 28-day concrete compressive strength (MPa)

Shrinkage constant K can be determined from Table 2.11.

Correction term for effect of humidity, $\beta(h)$, should be calculated as shown:

$$\beta(h) = 1 - 1.18\left(\frac{H}{100}\right)^4 \tag{2.42}$$

where

H = Relative humidity (%)

Correction term for effect of time, $\beta(t)$, should be determined as follows:

$$\beta(t) = \left(\frac{t - t_c}{t - t_c + 0.12(V/S)^2}\right)^{1/2} \tag{2.43}$$

where

t = Age of concrete after casting (days)
t_c = Age of concrete at the beginning of shrinkage (days)
V/S = Volume-to-surface area ratio (mm)

Creep calculation. The creep compliance is composed of two parts: the elastic strain and the creep strain according to the following equation:

$$J(t, t_0) = \frac{1}{E_{cmt_0}} + \frac{\phi(t, t_0)}{E_{cm_{28}}} \tag{2.44}$$

Table 2.11 Shrinkage Constant, K, as a Function of Cement Type

Type of Cement	K
I	1.00
II	0.75
III	1.15

where

E_{cmt_0} = Modulus of elasticity of concrete at loading (MPa)
$E_{cm_{28}}$ = Modulus of elasticity of concrete at 28 days (MPa)
$\phi(t, t_0)$ = Creep coefficient

$$E_{cmt_0} = 3500 + 4300\sqrt{f_{cmt_0}} \tag{2.45}$$

where f_{cmt_0} = Concrete mean compressive strength at loading (MPa), which can be determined as follows:

$$f_{cmt_0} = f_{cm_{28}} \frac{t^{3/4}}{a + bt^{3/4}}$$

Coefficients a and b are related to the cement type as shown in Table 2.12.

$$E_{cm_{28}} = 3500 + 4300\sqrt{f_{cm_{28}}} \tag{2.46}$$

Creep coefficient, $\phi(t, t_0)$, can be calculated as shown:

$$\phi(t, t_0) = \Phi(t_c)2\left(\frac{(t - t_0)^{0.3}}{(t - t_0)^{0.3} + 14}\right) + \left(\frac{7}{t_0}\right)^{0.5}\left(\frac{t - t_0}{t - t_0 + 7}\right)^{0.5} + 2.5(1 - 1.086h^2)\left(\frac{t - t_0}{t - t_0 + 0.12(V/S)^2}\right)^{0.5} \tag{2.47}$$

If $t_0 = t_c$ then $\Phi(t_c) = 1$ (2.48)

If $t_0 > t_c$ then $\Phi(t_c) = \left[1 - \left(\frac{t_0 - t_c}{t_0 - t_c + 0.12(V/S)^2}\right)^{0.5}\right]^{0.5}$ (2.49)

$h = H/100$ (H = Relative humidity(%))

2.13.4 The CEB 90 Model

The CEB 90 Model was developed by Muller and Hillsdorf [16].

Shrinkage calculation. Parameters required for calculation of shrinkage strain using the CEB 90 model are mean 28-day concrete compressive strength, $f_{cm_{28}}$, relative humidity, H, age of concrete at the beginning of shrinkage, t_c, type of cement, and specimen shape.

The strain due to shrinkage may be calculated from the following equation:

$$\varepsilon_s(t, t_c) = \varepsilon_{cs_0}\beta_s(t, t_c) \tag{2.50}$$

Table 2.12 Coefficient *a* and *b* as a Function of Cement Type

Cement Type	*a*	*b*
I	2.8	0.77
II	3.4	0.72
III	1.0	0.92

where

ε_{cs_0} = Notional shrinkage coefficient according to Eq. 2.51
$\beta_s(t, t_c)$ = Coefficient describing development of shrinkage with time according to Eq. 2.54

Notional shrinkage coefficient is

$$\varepsilon_{cs_0} = \varepsilon_s(f_{cm_{28}})\beta_{RH} \tag{2.51}$$

where

$\varepsilon_s(f_{cm_{28}})$ = Concrete strength factor on shrinkage according to Eq. 2.52
β_{RH} = Relative humidity factor on notional shrinkage coefficient according to Table 2.13

Concrete strength factor on shrinkage, $\varepsilon_s(f_{cm_{28}})$, can be calculated as follows:

$$\varepsilon_s(f_{cm_{28}}) = \left[160 + 10(\beta_{sc})\left(9 - \frac{f_{cm_{28}}}{10}\right)\right] \times 10^{-6} \tag{2.52}$$

where

β_{sc} = Coefficient that depends on type of cement according to Table 2.14.
$f_{cm_{28}}$ = Mean 28-day concrete compressive strength (MPa)

Coefficient β_{sc} dependent on humidity, β_{RH}, should be determined according to Table 2.14, where

$$\beta_{arh} = 1 - \left(\frac{H}{100}\right)^3 \tag{2.53}$$

The development of shrinkage with time is given by

$$\beta_s(t - t_c) = \sqrt{\frac{(t - t_c)}{0.56(h_e/4)^2 + (t - t_c)}} \tag{2.54}$$

Table 2.13 Determination of Coefficient β_{RH}

Humidity	β_{RH}
$40\% \leq H < 99\%$	$-1.55 \times \beta_{arh}$
$H \geq 99\%$	0.25

Table 2.14 Coefficient β_{sc}

Type of Cement	European Type	American Type	β_{sc}
Slow hardening	SL	II	4
Normal/rapid hardening	R	I	5
Rapid hardening, high strength	RS	III	8

where

t = Age of concrete (days)
t_c = Age of concrete at the beginning of shrinkage (days)
h_e = Effective thickness to account for volume/surface ratio (mm)

Effective thickness, h_e, can be determined as follows:

$$h_e = \frac{2A_c}{u} \tag{2.55}$$

where

A_c = Cross-section of the structural member (mm^2)
u = Perimeter of the structural member in contact with the atmosphere (mm)

Creep calculation. Creep compliance represents the total stress dependent strain per unit stress. It can be calculated as

$$J(t, t_0) = \frac{1}{E_{\text{cmt}_0}} + \frac{\phi(t, t_0)}{E_{\text{cm}_{28}}} \tag{2.56}$$

where

E_{cmt_0} = Modulus of elasticity at age of loading (MPa)
$E_{\text{cm}_{28}}$ = Modulus of elasticity at 28 days (MPa)
$\phi(t, t_0)$ = Creep coefficient

$$E_{\text{cmt}_0} = E_{\text{cm}_{28}} \exp\left[0.5S\left(1 - \sqrt{\frac{28}{t}}\right)\right] \tag{2.57}$$

S is the coefficient that depends on cement type and can be determined from Table 2.15.

$$E_{\text{cm}_{28}} = 21500\sqrt[3]{\frac{f_{\text{cm}_{28}}}{10}} \tag{2.58}$$

Creep coefficient, $\phi(t, t_0)$, can be evaluated from the given equation:

$$\phi(t, t_0) = \phi_0 \beta_c(t, t_0) \tag{2.59}$$

Table 2.15 Coefficient S as a Function of Cement Type

Cement Type	European Type	U.S. Type	S
Slow hardening	SL	II	0.38
Normal/rapid hardening	R	I	0.25
Rapid hardening high strength	RS	III	0.20

where

ϕ_0 = Notional creep coefficient
$\beta_c(t, t_0)$ = Equation describing development of creep with time after loading

$$\phi_0 = \phi_{RH}\beta(f_{cm_{28}})\beta(t_0) \tag{2.60}$$

where ϕ_{RH} = Relative humidity factor on the notional creep coefficient, which is given by

$$\phi_{RH} = 1 + \frac{1 - H/100}{0.16\sqrt[3]{h_e/4}} \tag{2.61}$$

$\beta(f_{cm_{28}})$ = Concrete strength factor on the notional creep coefficient, which is given by

$$\beta(f_{cm_{28}}) = \frac{5.3}{\sqrt{f_{cm_{28}}/10}} \tag{2.62}$$

$\beta\ (t_0)$ = Age of concrete at loading factor on the notional creep coefficient, which is given by

$$\beta(t_0) = \frac{1}{0.1 + t_0^{0.2}} \tag{2.63}$$

An equation describing development of creep with time after loading, $\beta_c(t, t_0)$, can be calculated using the following equation:

$$\beta_c(t, t_0) = \left(\frac{t - t_0}{\beta_H + t - t_0}\right)^{0.3} \tag{2.64}$$

$$\beta_H = 1.5h_e[1 + (0.012H)^{18}] + 250 \leq 1500 \text{ days} \tag{2.65}$$

2.13.5 The CEB 90-99 Model

The CEB 90-99 is a modification of the CEB 90 and is described in Ref. 17.

Shrinkage calculation. In this new model, total shrinkage contains of autogenous and drying shrinkage component. In high-performance concrete, autogenous shrinkage is significant and needs to be considered in prediction of shrinkage. This new approach was necessary so that shrinkage of normal as well as high-performance concrete can be predicted with sufficient accuracy [1].

Total shrinkage strain can be calculated using the following equation:

$$\varepsilon_s(t, t_c) = \varepsilon_{as}(t) + \varepsilon_{ds}(t, t_c) \tag{2.66}$$

where

$\varepsilon_{as}(t)$ = Autogenous shrinkage at time t
$\varepsilon_{ds}(t, t_c)$ = Drying shrinkage at time t

Autogenous shrinkage, $\varepsilon_{as}(t)$, should be calculated as follows:

$$\varepsilon_{as}(t) = \varepsilon_{as_0}(f_{cm_{28}})\beta_{as}(t) \tag{2.67}$$

where

$\varepsilon_{as_0}(f_{cm_{28}})$ = National autogenous shrinkage coefficient according to Eq. 2.68
$\beta_{as}(t)$ = Function to describe the time-development of autogenous shrinkage, from Eq. 2.69

National autogenous shrinkage coefficient, $\varepsilon_{cas_0}(f_{cm})$, can be calculated as follows:

$$\varepsilon_{as_0}(f_{cm_{28}}) = -\alpha_{as}\left(\frac{f_{cm_{28}}/10}{6 + f_{cm_{28}}/10}\right)^{2.5} \times 10^{-6} \tag{2.68}$$

where

α_{as} = Coefficient that depends on type of cement
= 800 for slowly hardening cements
= 700 for normal or rapidly hardening cements
= 600 for rapidly hardening high-strength cements
$f_{cm_{28}}$ = Mean compressive strength of concrete at an age of 28 days (MPa)

Function $\beta_{as}(t)$ should be calculated using the following equation:

$$\beta_{as}(t) = 1 - \exp[-0.2(t)^{0.5}] \tag{2.69}$$

where t = Age of concrete (days)

Drying shrinkage, $\varepsilon_{ds}(t, t_c)$, can be estimated by the following equation:

$$\varepsilon_{ds}(t, t_c) = \varepsilon_{ds_0}(f_{cm_{28}})\beta_{RH}(H)\beta_{ds}(t - t_c) \tag{2.70}$$

where

$\varepsilon_{ds_0}(f_{cm_{28}})$ = Notional drying shrinkage coefficient according to Eq. 2.71
$\beta_{RH}(H)$ = Coefficient to take into account the effect of relative humidity on drying shrinkage according to Eq. 2.72
$\beta_{ds}(t - t_c)$ = Function to describe the time development of drying shrinkage according to Eq. 2.74

Notional drying shrinkage coefficient, $\varepsilon_{ds_0}(f_{cm_{28}})$, may be calculated from the following equation:

$$\varepsilon_{ds_0}(f_{cm_{28}}) = [(220 + 110\alpha_{ds_1})\exp(-\alpha_{ds_2} f_{cm_{28}}/10)] \times 10^{-6} \tag{2.71}$$

where

α_{ds_1} = Coefficient that depends on type of cement
= 3 for slowly hardening cements
= 4 for normal or rapidly hardening cements
= 6 for rapidly hardening high-strength cements
α_{ds_2} = Coefficient that depends on type of cement
= 0.13 for slowly hardening cements
= 0.11 for normal or rapidly hardening cements
= 0.12 for rapidly hardening high-strength cements

Coefficient $\beta_{RH}(H)$ should be calculated as follows:

$$\beta_{RH} = \begin{cases} -1.55\left[1 - \left(\dfrac{H}{100}\right)^3\right] & \text{for } 40\% \le H < 99\% \times \beta_{s_1} \\ 0.25 & \text{for } H \ge 99\% \times \beta_{s_1} \end{cases} \tag{2.72}$$

where

H = Ambient relative humidity (%)

β_{s_1} = Coefficient to take into account the self-desiccation in high-performance concrete. It can be determined as follows:

$$\beta_{s_1} = \left(\frac{35}{f_{cm_{28}}}\right)^{0.1} \le 1.0 \tag{2.73}$$

Function $\beta_{ds}(t - t_c)$ may be estimated as follows:

$$\beta_{ds}(t - t_c) = \left(\frac{(t - t_c)}{0.56(h_e/4)^2 + (t - t_c)}\right)^{0.5} \tag{2.74}$$

where

t_c = Concrete age at the beginning of drying (days)

$h_e = \dfrac{2A_c}{u}$ = notional size of member (mm), where A_c is the cross-section (mm^2) and u is the perimeter of the member in contact with the atmosphere (mm)

Creep calculation. Total stress-dependent strain per unit stress, also called creep compliance or creep function can be determined as follows:

$$J(t, t_0) = \frac{1}{E_{cmt_0}} + \frac{\phi(t, t_0)}{E_{cm_{28}}} \tag{2.75}$$

where

E_{cmt_0} = Modulus of elasticity at age of loading (MPa)

$E_{cm_{28}}$ = Modulus of elasticity at day 28 (MPa)

$\phi(t, t_0)$ = Creep coefficient

$$E_{cmt_0} = E_{cm_{28}} \exp\left[0.5S\left(1 - \sqrt{\left(\frac{28}{t_0}\right)}\right)\right] \tag{2.76}$$

S is the coefficient that depends on cement type and compressive strength and can be determined from Table 2.16.

$$E_{cm_{28}} = 21500\sqrt[3]{\frac{f_{cm_{28}}}{10}} \tag{2.77}$$

Creep coefficient, $\phi(t, t_0)$, can be evaluated from the given equation:

$$\phi(t, t_0) = \phi_0 \beta_c(t, t_0) \tag{2.78}$$

Table 2.16 Coefficient S as a Function of Cement Type and Compressive Strength

$f_{cm_{28}}$ (MPa)	Type of Cement	S
≤60	Rapidly hardening high strength	0.20
	Normal and rapidly hardening	0.25
	Slow hardening	0.38
>60	All types	0.20

where

ϕ_0 = Notional creep coefficient

$\beta_c(t, t_0)$ = Equation describing development of creep with time after loading

$$\phi_0 = \phi_{RH}\beta(f_{cm_{28}})\beta(t_0) \tag{2.79}$$

ϕ_{RH} = Relative humidity factor on the notional creep coefficient

$$\phi_{RH} = \left[1 + \frac{1 - H/100}{0.16\sqrt[3]{h_e/4}}\alpha_1\right]\alpha_2 \tag{2.80}$$

where

$$\alpha_1 = \left[\frac{35}{f_{cm_{28}}}\right]^{0.7} \tag{2.81}$$

$$\alpha_2 = \left[\frac{35}{f_{cm_{28}}}\right]^{0.2} \tag{2.82}$$

$\beta(f_{cm_{28}})$ = Concrete strength factor on the notional creep coefficient,

$$\beta(f_{cm_{28}}) = \frac{5.3}{\sqrt{f_{cm_{28}}/10}} \tag{2.83}$$

$\beta(t_0)$ = Age of concrete at loading factor on the notional creep coefficient

$$\beta(t_0) = \frac{1}{0.1 + t_0^{0.2}} \tag{2.84}$$

where

$$t_0 = t_{0,T}\left[\frac{9}{2 + t_{0,T}^{1.2}} + 1\right]^{\alpha} \geq 0.5 \text{ days} \tag{2.85}$$

t_0 = Age of concrete at loading (days)

$t_{0,T}$ = Age of concrete at loading adjusted according to the concrete temperature; for $T = 20\,^{\circ}\text{C}$, $t_{0,T}$ corresponds to t_0

α = Coefficient that depends on type of cement

= −1 for slowly hardening cement

= 0 for normal or rapidly hardening cement

= 1 for rapidly hardening high-strength cement

An equation describing development of creep with time after loading, $\beta_c(t, t_0)$, can be calculated using the following equation:

$$\beta_c(t, t_0) = \left(\frac{t - t_0}{\beta_H + t - t_0}\right)^{0.3} \tag{2.86}$$

$$\beta_H = 1.5h_e[1 + (0.012H)^{18}] + 250\alpha_3 \leq 1500\alpha_3 \tag{2.87}$$

$$\alpha_3 = \left[\frac{35}{f_{cm_{28}}}\right]^{0.5} \tag{2.88}$$

2.13.6 The AASHTO Model

Shrinkage calculation. Parameters required for calculation of shrinkage strain using the AASHTO model are: curing method (moist-cured or steam-cured concrete), 28-day concrete compressive strength, $f_{cm_{28}}$, relative humidity, H, drying time of concrete, t, type of cement, and specimen shape.

The strain due to shrinkage may be calculated from the following equation:

- For moist-cured concrete:

$$\varepsilon_{sh} = -k_s k_h \left(\frac{t}{35.0 + t}\right) 0.51 \times 10^{-3} \tag{2.89}$$

- For steam-cured concrete:

$$\varepsilon_{sh} = -k_s k_h \left(\frac{t}{55.0 + t}\right) 0.56 \times 10^{-3} \tag{2.90}$$

where

t = drying time (day)
k_s = size factor for shrinkage specified in Eq. 2.91
k_h = humidity factor for shrinkage specified in Eq. 2.92

Size factor for shrinkage should be calculated as follows:

$$k_s = \left[\frac{\dfrac{t}{26e^{0.36(V/S)} + t}}{\dfrac{t}{45 + t}}\right]\left[\frac{1064 - 94(V/S)}{923}\right] \tag{2.91}$$

where

V = Volume of the specimen (in.^{3})
S = Surface of the specimen (in.^{2})

Humidity factor for shrinkage is:

$$k_h = \frac{140 - H}{70} \quad \text{for } H < 80\%$$
$$k_h = \frac{3(100 - H)}{70} \quad \text{for } H \geq 80\% \tag{2.92}$$

where

H = Relative humidity (%)

Creep calculation. The creep compliance represents the total stress dependent strain per unit stress. It can be calculated as:

$$J(t, t_0) = \frac{1}{E_c} + \frac{\psi(t, t_0)}{E_c} \tag{2.93}$$

where

$\Psi(t, t_0)$ = Creep coefficient as given in Eq. 2.94

E_c = Modulus of elasticity at 28 days (ksi) as given in Eq. 2.97

The creep coefficient may be calculated from the following equation:

$$\psi(t, t_0) = 3.5 k_c k_f \left(1.58 - \frac{H}{120}\right) t_0^{-0.118} \frac{(t - t_0)^{0.6}}{10.0 + (t - t_0)^{0.6}} \tag{2.94}$$

where

t = Maturity of concrete (day)

t_0 = Age of concrete when load is initially applied (day)

H = Relative humidity (%)

k_f = Factor for the effect of concrete strength as given in Eq. 2.95

k_c = Factor for the effect of the volume-to-surface ratio of the component as given in Eq. 2.96

The factor for the effect of concrete strength should be calculated as follows:

$$k_f = \frac{1}{0.67 + \frac{f_{cm28}}{9}} \tag{2.95}$$

where

f_{cm28} = Specified concrete compressive strength at 28 days (Ksi)

The factor for the effect of the volume-to-surface ratio of the component should be calculated as follows:

$$k_c = \left[\frac{\frac{t}{26e^{0.36(V/S)} + t}}{\frac{t}{45 + t}}\right] \left[\frac{1.80 + 1.77e^{-0.54(V/S)}}{2.587}\right] \tag{2.96}$$

where

V = Volume of the specimen (in.^{3})

S = Surface of the specimen (in.^{2})

The modulus of elasticity at 28 days should be calculated as follows:

$$E_c = 33000 \omega_c^{1.5} \sqrt{f_c'} \tag{2.97}$$

where

ω_c = Concrete unit weight (Kcf)

f_c' = Specified concrete compressive strength at 28 days (Ksi)

Example 2.1

Calculate shrinkage strain and creep compliance for the concrete specimen given below. Use the ACI 209 model.

Given factors:

Humidity = 75%
$h_e = 2V/S = 2A_c/u = 76\,\text{mm}$
$f_{\text{cm}_{28}} = 45.2$ MPa
$w = 207.92\,\text{kg/m}^3$
$w/c = 0.46$
$a/c = 3.73$
$t = 35$ days
$t_0 = 28$ days
$t_c = 8$ days
$\gamma = 2405\,\text{kg/m}^3$
Cement type III
Moist-cured concrete

Solution

Shrinkage calculation

$$\varepsilon_s(t) = \frac{(t - t_c}{b + (t - t_c)} K_{\text{ss}} K_{\text{sh}} \varepsilon_{\text{shu}}$$

$$\varepsilon_{\text{shu}} = 780 \times 10^{-6}\ \text{mm/mm}$$

According to Table 2.4, $b = 35$

$$V/S = 38\ \text{mm}$$

$$K_{\text{ss}} = 1.14 - 0.0035\left(\frac{V}{S}\right) = 1.14 - 0.0035(38) = 1.007$$

For $H = 75\%$,

$$K_{\text{sh}} = 1.40 - 0.01H = 1.40 - 0.01(75) = 0.65$$

$$\varepsilon_s(t) = \frac{(t - t_c)}{b + (t - t_c)} K_{\text{ss}} K_{\text{sh}} \varepsilon_{\text{shu}}$$

$$= \frac{(35 - 8)}{35 + (35 - 8)}(1.007)(0.65)(780 \times 10^{-6}) = 222.3 \times 10^{-6}\ \text{mm/mm}$$

Creep calculation

$$J(t, t_0) = \frac{1 + C_c(t)}{E_{\text{cmt}_0}} \qquad \text{(Eq. 2.7)}$$

Determination of E_{cmt_0}

$b = 2.30$, $c = 0.92$ (Table 2.5)

$$f_c'(t_0) = f_{\text{cm}_{28}} \frac{t_0}{b + ct_0} = 45.2 \frac{28}{2.3 + 0.92 \times 28} = 45.1\ \text{MPa}$$

$$E_{\text{cmt}_0} = 0.043(\gamma)^{3/2}\sqrt{f_c'(t_0)} = 0.043(2405)^{3/2}\sqrt{45.1} = 34058.8\ \text{MPa}$$

Determination of $C_c(t)$

$$C_{cu} = 2.35$$

$$K_{ch} = 1.27 - 0.0067(H) = 1.27 - 0.0067(75) = 0.767$$

$$K_{ca} = 1.25(t_0)^{-0.118} = 1.25(28)^{-0.118} = 0.844$$

$$K_{cs} = 1.14 - 0.0035(V/S) = 1.14 - 0.0035(38) = 1.007$$

$$C_c(t) = \frac{t^{0.60}}{10 + t^{0.60}} C_{cu} K_{ch} K_{ca} K_{cs} = \frac{35^{0.60}}{10 + 35^{0.60}} 2.35 \times 0.767 \times 0.844 \times 1.00 = 0.702$$

$$J(t, t_0) = \frac{1 + C_c(t)}{E_{cmt_0}} = \frac{1 + 0.702}{34058.8} = 49.9 \times 10^{-6} \frac{1}{\text{MPa}}$$

Example 2.2

Using the B3 model, calculate shrinkage strain and creep function for the specimen given in Example 2.1.

Solution

Shrinkage calculation

$$\varepsilon_s(t) = (\varepsilon_{shu})(K_h)S(t)$$

Determination of ε_{shu}

$$\alpha_1 = 1.10 \text{ (Table 2.7)}$$

$$\alpha_2 = 1.0 \text{ (Table 2.8)}$$

$$\varepsilon_{shu} = \alpha_1 \alpha_2 [0.019(w)^{2.1}(f_{cm_{28}})^{-0.28} + 270] \times 10^{-6}$$

$$= (1.10)(1.0)[0.019(207.92)^{2.1}(45.2)^{-0.28} + 270] \times 10^{-6} = 827 \times 10^{-6} \text{ mm/mm}$$

Determination of K_h
According to the Table 2.6, for $H = 75\%$

$$K_h = 1 - \left(\frac{H}{100}\right)^3 = 1 - \left(\frac{75}{100}\right)^3 = 0.578$$

Determination of $S(t)$

$$K_s = 1.0, \text{ since the type of member is not defined}$$

$$T_{sh} = 0.085(t_c)^{-0.08}(f_{cm_{28}})^{-0.25}[2K_s(V/S)]^2$$

$$= 0.085(8)^{-0.08}(45.2)^{-0.25}[2(1.0)(38)]^2 = 160.3$$

$$S(t) = \tanh\sqrt{\frac{t - t_c}{T_{sh}}} = \tanh\sqrt{\frac{35 - 8}{160.3}} = 0.389$$

$$\varepsilon_s(t) = (\varepsilon_{shu})(K_h)S(t) = (827 \times 10^{-6})(0.578)(0.389) = 185.9 \times 10^{-6} \text{ mm/mm}$$

Creep calculation

$$J(t, t_0) = q_1 + C_0(t, t_0) + C_d(t, t_0, t_c) \tag{2.15}$$

Determination of q_1

$$E_{cm_{28}} = 4735\sqrt{f_{cm_{28}}} = 4735\sqrt{45.2} = 31833.9 \text{ MPa}$$

$$q_1 = \frac{0.6}{E_{cm_{28}}} = \frac{0.6}{31833.9} = 18.85 \times 10^{-6}\frac{1}{\text{MPa}}$$

Calculation of $C_0(t, t_0)$

$$c = \frac{w}{w/c} = \frac{207.92}{0.46} = 452 \text{ kg/m}^3$$

$$q_2 = 185.4(c)^{0.5}(f_{cm_{28}})^{-0.9} \times 10^{-6} = 185.4(452)^{0.5}(45.2)^{-0.9} \times 10^{-6}$$
$$= 127.6 \times 10^{-6}$$

$$Q_f(t_0) = \frac{1}{0.086(t_0)^{2/9} + 1.21(t_0)^{4/9}} = \frac{1}{0.086(28)^{2/9} + 1.21(28)^{4/9}} = 0.182$$

$$Z(t, t_0) = \frac{\ln[1 + (t - t_0)^{0.1}]}{\sqrt{t_0}} = \frac{\ln[1 + (35 - 28)^{0.1}]}{\sqrt{28}} = 0.150$$

$$r(t_0) = 1.7(t_0)^{0.12} + 8 = 1.7(28)^{0.12} + 8 = 10.54$$

$$Q(t, t_0) = Q_f(t_0)\left[1 + \frac{Q_f(t_0)^{r(t_0)}}{Z(t, t_0)^{r(t_0)}}\right]^{-1/r(t_0)} = 0.182\left[1 + \frac{0.182^{10.54}}{0.150^{10.54}}\right]^{-1/10.54} = 0.148$$

$$q_3 = 0.29q_2\left(\frac{w}{c}\right)^4 = 0.29(127.6 \times 10^{-6})(0.46)^4 = 1.66 \times 10^{-6}$$

$$q_4 = 20.3\left(\frac{a}{c}\right)^{-0.7} \times 10^{-6} = 20.3(3.73)^{-0.7} \times 10^{-6} = 8.08 \times 10^{-6}$$

$$C_0(t, t_0) = q_2 Q(t, t_0) + q_3 \ln[1 + (t - t_0)^{0.1}] + q_4 \ln\left(\frac{t}{t_0}\right)$$
$$= (127.6 \times 10^{-6})(0.148) + (1.66 \times 10^{-6}) \ln[1 + (35 - 28)^{0.1}] + (8.08 \times 10^{-6}) \ln\left(\frac{35}{28}\right)$$
$$= 22.01 \times 10^{-6}\frac{1}{\text{MPa}}$$

Calculation of $C_d(t, t_0, t_c)$:

$$q_5 = \frac{0.757|\varepsilon_{shu} \times 10^6|^{-0.6}}{f_{cm_{28}}} = \frac{0.757|827 \times 10^{-6} \times 10^6|^{-0.6}}{45.2} = 297.5 \times 10^{-6}$$

$$S(t) = 0.389$$

$$S(t_0) = \tanh\sqrt{\frac{t_0 - t_c}{T_{sh}}} = \tanh\sqrt{\frac{28 - 8}{160.3}} = 0.339$$

$$H(t) = 1 - \left[\left(1 - \frac{H}{100}\right)S(t)\right] = 1 - \left(1 - \frac{75}{100}\right)0.389] = 0.903$$

$$H(t_0) = 1 - \left[\left(1 - \frac{H}{100}\right)S(t_0)\right] = 1 - \left[\left(1 - \frac{75}{100}\right)0.339\right] = 0.915$$

$$C_d(t, t_0, t_c) = q_5\sqrt{\exp[-8H(t)] - \exp[-8H(t_0)]}$$

$$= (297 \times 10^{-6})\sqrt{\exp[-8 \times 0.903] - \exp[-8 \times 0.915]} = 2.43 \times 10^{-6}\frac{1}{\text{MPa}}$$

$$J(t, t_0) = q_1 + C_0(t, t_0) + C_d(t, t_0, t_c)$$

$$= (18.85 \times 10^{-6}) + (22.01 \times 10^{-6}) + (2.43 \times 10^{-6}) = 43.3 \times 10^{-6}\frac{1}{\text{MPa}}$$

Example 2.3

Using the GL 2000 model, calculate the shrinkage strain and creep function for the specimen given in Example 2.1.

Solution

Shrinkage calculation

$$\varepsilon_s(t) = \varepsilon_{\text{shu}}\beta(h)\beta(t)$$

Calculation of ε_{shu}

$K = 1.15$ (Table 2.11)

$$\varepsilon_{\text{shu}} = (900)K\left(\frac{30}{f_{\text{cm}_{28}}}\right)^{1/2} \times 10^{-6} = (900)(1.15)\left(\frac{30}{45.2}\right)^{1/2} \times 10^{-6} = 843.2 \times 10^{-6} \text{ mm/mm}$$

Calculation of $\beta(h)$

$$\beta(h) = 1 - 1.18\left(\frac{H}{100}\right)^4 = 1 - 1.18\left(\frac{75}{100}\right)^4 = 0.627$$

Calculation of $\beta(t)$:

$$\beta(t) = \left(\frac{t - t_c}{t - t_c + 0.12(V/S)^2}\right)^{1/2} = \left(\frac{35 - 8}{35 - 8 + 0.12(38)^2}\right)^{1/2} = 0.367$$

$$\varepsilon_s(t) = \varepsilon_{\text{shu}}\beta(h)\beta(t) = (843.2 \times 10^{-6})(0.627)(0.367) = 194 \times 10^{-6} \text{ mm/mm}$$

Creep calculation

$$J(t, t_0) = \frac{1}{E_{\text{cmt}_0}} + \frac{\phi(t, t_0)}{E_{\text{cm}_{28}}}$$

Calculation of E_{cmt_0} and $E_{\text{cm}_{28}}$

$$t_0 = 28 \text{ days} \Rightarrow E_{\text{cmt}_0} = E_{\text{cm}_{28}}$$

$$E_{\text{cm}_{28}} = 3500 + 4300\sqrt{f_{\text{cm}_{28}}} = 3500 + 4300\sqrt{45.2} = 32409.3 \text{ MPa}$$

Calculation of $\phi(t, t_0)$

$$t_0 = 28 > t_c = 8 \text{ days}$$

$$\Phi(t_c) = \left[1 - \left(\frac{t_0 - t_c}{t_0 - t_c + 0.12(V/S)^2}\right)^{0.5}\right]^{0.5} = \left[1 - \left(\frac{28 - 8}{28 - 8 + 0.12(38)^2}\right)^{0.5}\right]^{0.5} = 0.824$$

$$h = H/100 = 75/100 = 0.75$$

$$\phi(t, t_0) = \Phi(t_c)\left[2\left(\frac{(t-t_0)^{0.3}}{(t-t_0)^{0.3}+14}\right) + \left(\frac{7}{t_0}\right)^{0.5}\left(\frac{t-t_0}{t-t_0+7}\right)^{0.5} + 2.5(1-1.086h^2)\left(\frac{t-t_0}{t-t_0+0.12(V/S)^2}\right)^{0.5}\right]$$

$$= 0.824\left[2\left(\frac{(35-28)^{0.3}}{(35-28)^{0.3}+14}\right) + \left(\frac{7}{28}\right)^{0.5}\left(\frac{35-28}{35-28+7}\right)^{0.5} + 2.5(1-1.086(0.75)^2)\left(\frac{35-28}{35-28+0.12(38)^2}\right)^{0.5}\right] = 0.636$$

$$J(t, t_0) = \frac{1}{E_{\text{cmt}_0}} + \frac{\phi(t, t_0)}{E_{\text{cm}_{28}}} = \frac{1}{32409.3} + \frac{0.636}{32409.3} = 50.5 \times 10^{-6}\frac{1}{\text{MPa}}$$

Example 2.4

Using the CEB 90 model, calculate shrinkage strain and creep function for the specimen given in Example 2.1.

Solution

Shrinkage calculation

$$\varepsilon_s(t, t_c) = (\varepsilon_{\text{cs}_0})\beta_s(t, t_c)$$

Calculation of $\varepsilon_{\text{cs}_0}$

$$\beta_{\text{sc}} = 8$$

$$\varepsilon_s(f_{\text{cm}_{28}}) = \left[160 + 10(\beta_{\text{sc}})\left(9 - \frac{f_{\text{cm}_{28}}}{10}\right)\right] \times 10^{-6}$$

$$= \left[160 + 10(8)\left(9 - \frac{45.2}{10}\right)\right] \times 10^{-6} = 518.4 \times 10^{-6} \text{ mm/mm}$$

For $H = 75\%$,

$$\beta_{\text{RH}} = -1.55\beta_{\text{arh}}$$

$$\beta_{\text{arh}} = 1 - (H/100)^3 = 1 - (75/100)^3 = 0.578$$

$$\beta_{\text{RH}} = -1.55\beta_{\text{arh}} = -1.55 \times 0.578 = -0.896$$

$$\varepsilon_{\text{cs}_0} = \varepsilon_s(f_{\text{cm}_{28}})(\beta_{\text{RH}}) = (518.4 \times 10^{-6})(-0.896) = -464.2 \times 10^{-6} \text{ mm/mm}$$

Calculation of $\beta_s(t - t_c)$

$$h_e = \frac{2A_c}{u} = 76 \text{ mm}$$

$$\beta_s(t - t_c) = \sqrt{\frac{(t-t_c)}{0.56(h_e/4)^2 + (t-t_c)}} = \sqrt{\frac{(35-8)}{0.56(76/4)^2 + (35-8)}} = 0.343$$

$$\varepsilon_s(t, t_c) = (\varepsilon_{\text{cs}_0})\beta_s(t - t_c) = (-464.2 \times 10^{-6})(0.343) = -159.3 \times 10^{-6} \text{ mm/mm}$$

Creep calculation

$$J(t, t_0) = \frac{1}{E_{\text{cmt}_0}} + \frac{\phi(t, t_0)}{E_{\text{cm}_{28}}}$$

Calculation of E_{cmt_0} and $E_{\text{cm}_{28}}$

$$t_0 = 28 \text{ days} \Rightarrow E_{\text{cmt}_0} = E_{\text{cm}_{28}}$$

$$E_{\text{cm}_{28}} = 21500\sqrt[3]{\frac{f_{\text{cm}_{28}}}{10}} = 21500\sqrt[3]{\frac{45.2}{10}} = 35548 \text{ MPa}$$

Calculation of $\phi(t, t_0)$

$$\phi_{\text{RH}} = 1 + \frac{1 - H/100}{0.16\sqrt[3]{h_e/4}} = 1 + \frac{1 - 75/100}{0.16\sqrt[3]{76/4}} = 1.586$$

$$\beta(f_{\text{cm}_{28}}) = \frac{5.3}{\sqrt{f_{\text{cm}_{28}}/10}} = \frac{5.3}{\sqrt{45.2/10}} = 2.49$$

$$\beta(t_0) = \frac{1}{0.1 + t_0^{0.02}} = \frac{1}{0.1 + 28^{0.2}} = 0.488$$

$$\phi_0 = \phi_{\text{RH}}\beta(f_{\text{cm}_{28}})\beta(t_0) = (1.586)(2.49)(0.488) = 1.927$$

$$\beta_H = 1.5h_e[1 + (0.012H)^{18}] + 250 = 1.5(76)[1 + (0.012 \times 75)^{18}] + 250$$

$$= 379 \leq 1500 \text{ days}$$

$$\beta_c(t, t_0) = \left(\frac{t - t_0}{\beta_H + t - t_0}\right)^{0.3} = \left(\frac{35 - 28}{379 + 35 - 28}\right)^{0.3} = 0.3$$

$$\phi(t, t_0) = \phi_0\beta_c(t, t_0) = 1.927 \times 0.3 = 0.578$$

$$J(t, t_0) = \frac{1}{E_{\text{cmt}_0}} + \frac{\phi(t, t_0)}{E_{\text{cm}_{28}}} = \frac{1}{35548} + \frac{0.578}{35548} = 44.4 \times 10^{-6}\frac{1}{\text{MPa}}$$

Example 2.5

Using the new CEB 90–99 model to calculate shrinkage strain and creep function for the specimen given in Example 2.1.

Solution

Shrinkage calculation

$$\varepsilon_s(t, t_c) = \varepsilon_{\text{as}}(t) + \varepsilon_{\text{ds}}(t, t_c)$$

Calculation of $\varepsilon_{\text{as}}(t)$

$$\alpha_{\text{as}} = 600 \text{ for rapidly hardening high-strength cements}$$

$$\varepsilon_{\text{as}_0}(f_{\text{cm}_{28}}) = -\alpha_{\text{as}}\left(\frac{f_{\text{cm}_{28}}/10}{6 + f_{\text{cm}_{28}}/10}\right)^{2.5} \times 10^{-6}$$

$$= -600\left(\frac{45.2/10}{6 + 45.2/10}\right)^{2.5} \times 10^{-6} = -72.6 \times 10^{-6} \text{ mm/mm}$$

$$\beta_{\text{as}}(t) = 1 - \exp(-0.2(t)^{0.5}) = 1 - \exp(-0.2(35)^{0.5}) = 0.694$$

$$\varepsilon_{\text{as}}(t) = \varepsilon_{\text{as}_0}(f_{\text{cm}_{28}})\beta_{\text{as}}(t) = (-72.6 \times 10^{-6})(0.694) = -50.4 \times 10^{-6} \text{ mm/mm}$$

Calculation of $\varepsilon_{ds}(t, t_c)$

$$\alpha_{ds_1} = 6 \text{ for rapidly hardening high-strength cements}$$

$$\alpha_{ds_2} = 0.12 \text{ for rapidly hardening high-strength cements}$$

$$\varepsilon_{ds_0}(f_{cm_{28}}) = [(220 + 110\alpha_{ds_1})\exp(-\alpha_{ds_2} f_{cm_{28}}/10)] \times 10^{-6}$$

$$= [(220 + 110 \times 6)\exp(-0.12 \times 45.2/10)] \times 10^{-6} = 511.6 \times 10^{-6} \text{ mm/mm}$$

$$\beta_{s1} = \left(\frac{35}{f_{cm_{28}}}\right)^{0.1} = \left(\frac{35}{45.2}\right)^{0.1} = 0.97 \leq 1.0$$

For 40% < H = 75% < 99% (0.97) = 96.5%,

$$\beta_{RH} = -1.55\left[1 - \left(\frac{H}{100}\right)^3\right] = -1.55\left[1 - \left(\frac{75}{100}\right)^3\right] = -0.896$$

$$\beta_{ds}(t - t_c) = \left(\frac{(t - t_c)}{0.56(h/4)^2 + (t - t_c)}\right)^{0.5}$$

$$= \left(\frac{(35 - 8)}{0.56(76/100)^2 + (35 - 8)}\right)^{0.5} = 0.343$$

$$\varepsilon_{ds}(t, t_c) = \varepsilon_{ds_0}(f_{cm_{28}})\beta_{RH}(H)\beta_{ds}(t - t_c)$$

$$= (511.6 \times 10^{-6})(-0.896)(0.343) = -157.2 \times 10^{-6} \text{ mm/mm}$$

$$\varepsilon_s(t, t_c) = \varepsilon_{as}(t) + \varepsilon_{ds}(t, t_c) = (-50.4 \times 10^{-6}) + (-157.2 \times 10^{-6}) = -207.6 \times 10^{-6} \text{ mm/mm}$$

Creep calculation

$$J(t, t_0) = \frac{1}{E_{cmt_0}} + \frac{\phi(t, t_0)}{E_{cm_{28}}}$$

Calculation of E_{cmt_0} and $E_{cm_{28}}$

$$t_0 = 28 \text{ days} \Rightarrow E_{cmt_0} = E_{cm_{28}}$$

$$E_{cm_{28}} = 21{,}500\sqrt[3]{\frac{f_{cm_{28}}}{10}} = 21{,}500\sqrt[3]{\frac{45.2}{10}} = 35{,}548 \text{ MPa}$$

Calculation of $\phi(t, t_0)$

$$\alpha_1 = \left[\frac{35}{f_{cm_{28}}}\right]^{0.7} = \left[\frac{35}{45.2}\right]^{0.7} = 0.836$$

$$\alpha_2 = \left[\frac{35}{f_{cm_{28}}}\right]^{0.2} = \left[\frac{35}{45.2}\right]^{0.2} = 0.950$$

$$\phi_{RH} = \left[1 + \frac{1 - H/100}{0.16\sqrt[3]{h_e/4}}\alpha_1\right]\alpha_2 = \left[1 + \frac{1 - 75/100}{0.16\sqrt[3]{76/4}}0.836\right]0.950 = 1.419$$

$$\beta(f_{cm_{28}}) = \frac{5.3}{\sqrt{f_{cm_{28}}/10}} = \frac{5.3}{\sqrt{45.2/10}} = 2.49$$

$$t_0 = t_{0.T}\left[\frac{9}{2 + t_{0.T}^{1.2}} + 1\right]^{\alpha} = 28\left[\frac{9}{2 + 28^{1.2}} + 1\right] = 32.5 \geq 0.5 \text{ days}$$

$$\beta(t_0) = \frac{1}{0.1 + t_0^{0.2}} = \frac{1}{0.1 + 32.5^{0.2}} = 0.475$$

$$\phi_0 = \phi_{RH}\beta(f_{cm_{28}})\beta(t_0) = 1.415 \times 2.49 \times 0.475 = 1.674$$

$$\alpha_3 = \left[\frac{35}{f_{cm_{28}}}\right]^{0.5} = \left[\frac{35}{45.2}\right]^{0.5} = 0.880$$

$$\beta_H = 1.5h_e[1 + (0.012H)^{18}] + 250\alpha_3$$

$$= 1.5 \times 76 \times [1 + (0.012 \times 75)^{18}] + 250 \times 0.88 = 351 \leq 1500 \times 0.880 = 1320$$

$$\beta_c(t, t_0) = \left(\frac{t - t_0}{\beta_H + t - t_0}\right)^{0.3} = \left(\frac{35 - 28}{351 + 35 - 28}\right)^{0.3} = 0.307$$

$$\phi(t, t_0) = \phi_0\beta_c(t, t_0) = 1.674 \times 0.307 = 0.514$$

$$J(t, t_0) = \frac{1}{E_{cmt_0}} + \frac{\phi(t, t_0)}{E_{cm_{28}}} = \frac{1}{35{,}548} + \frac{0.514}{35{,}548} = 42.6 \times 10^{-6}\frac{1}{\text{MPa}}$$

Example 2.6

Using the AASHTO model, calculate shrinkage strain and creep function for the specimen given in Example 2.1.

Solution

Shrinkage calculation

For moist-cured concrete, ε_{sh} should be taken as:

$$\varepsilon_{sh} = -K_s K_h \left(\frac{t}{35.0 + t}\right) 0.51 \times 10^{-3}$$

Determination of K_s:

$$\text{V/S} = 38 \text{ mm} = 1.5 \text{ in.}$$

$$K_s = \left[\frac{t}{\frac{26e^{0.36(V/S)} + t}{45 + t}}\right]\left[\frac{1064 - 94(V/S)}{923}\right]$$

$$K_s = \left[\frac{t}{\frac{26e^{0.36(1.5)} + t}{45 + t}}\right]\left[\frac{1064 - 94(1.5)}{923}\right] = 1$$

Determination of K_h:

For $H = 75\%$,

$$K_h = \left[\frac{140 - H}{70}\right] = \frac{140 - 75}{70} = 0.93$$

Calculation of ε_{sh}:

$$\varepsilon_{sh} = -1 \times 0.93 \times \left(\frac{35}{35.0 + 35}\right) 0.51 \times 10^{-3} = -237.15 \times 10^{-6} \text{ in/in.}$$

Creep calculation
The creep coefficient should be taken as:

$$\psi(t, t_0) = 3.5 K_c K_f \left(1.58 - \frac{H}{120}\right) t_0^{-0.118} \frac{(t - t_0)^{0.6}}{10.0 + (t - t_0)^{0.6}}$$

Determination of k_c:

$$k_c = \left[\frac{\frac{t}{26e^{0.36(V/S)} + t}}{\frac{t}{45 + t}}\right]\left[\frac{1.80 - 1.77e^{-0.54(V/S)}}{2.587}\right]$$

$$k_c = \left[\frac{\frac{t}{26e^{0.36(1.5)} + t}}{\frac{t}{45 + t}}\right]\left[\frac{1.80 + 1.77e^{-0.54\times 1.5}}{2.587}\right] = 1$$

Determination of k_f:

$$f_{cm_{28}} = 45.2 \text{ MPa} = 6.55 \text{ Ksi}$$

$$k_f = \left[\frac{1}{0.67 + \frac{f_{cm_{28}}}{9}}\right] = \frac{1}{0.67 + \frac{6.55}{9}} = 0.715$$

Calculation of $\psi\ (t, t_0)$:

$$\psi(t, t_0) = 3.5 \times 1 \times 0.715\left(1.58 - \frac{75}{120}\right) \times 28^{-0.118} \times \frac{(35 - 28)^{0.6}}{10.0 + (35 - 28)^{0.6}}$$

$$\psi(t, t_0) = 0.3923$$

Determination of E_c:

$$\omega_c = 2405 \text{ Kg/m}^3 = 0.15 \text{ Kcf}$$

$$E_c = 33000\omega_c^{1.5}\sqrt{f'_c}$$

$$E_c = 33000 \times 0.15^{1.5}\sqrt{6.55} = 4906.5 \text{ Ksi}$$

Calculation of $J(t, t_0)$:

$$J(t, t_0) = \frac{1}{E_c} + \frac{\psi(t, t_0)}{E_c}$$

$$J(t, t_0) = \frac{1}{4906.5} + \frac{0.3923}{4906.5} = 284 \times 10^{-6}\frac{1}{\text{Ksi}}$$

$$J(t, t_0) = 284 \times 10^{-6}\frac{1}{\text{Ksi}} = 41.2 \times 10^{-6}\frac{1}{\text{MPa}}$$

2.14 UNIT WEIGHT OF CONCRETE

The unit weight, w, of hardened normal concrete ordinarily used in buildings and similar structures depends on the concrete mix, maximum size and grading of aggregates, water–cement ratio, and strength of concrete. The following values of the unit weight of concrete may be used:

1. Unit weight of plain concrete using maximum aggregate size of $\frac{3}{4}$ in. (20 mm) varies between 145 and 150 lb/ft^3 (2320 to 2400 kg/m^3). For concrete of strength less than 4000 psi (280 kg/cm^2), a value of 145 lb/ft^3 (2320 kg/m^3) can be used, whereas for higher-strength concretes, w can be assumed to be equal to 150 lb/ft^3 (2400 kg/m^3).
2. Unit weight of plain concrete of maximum aggregate size of 4 to 6 in. (100 to 150 mm) varies between 150 and 160 lb/ft^3 (2400 to 2560 kg/m^3). An average value of 155 lb/ft^3 may be used.
3. Unit weight of reinforced concrete, using about 0.7% to 1.5% of steel in the concrete section, may be taken as 150 lb/ft^3 (2400 kg/m^3). For higher percentages of steel, the unit weight, w, can be assumed to be 155 lb/ft^3 (2500 kg/m^3).
4. Unit weight of lightweight concrete used for fireproofing, masonry, or insulation purposes varies between 20 and 90 lb/ft^3 (320 and 1440 kg/m^3). Concrete of upper values of 90 pcf or greater may be used for load-bearing concrete members.

The unit weight of heavy concrete varies between 200 and 270 lb/ft^3 (3200 and 4300 kg/m^3). Heavy concrete made with natural barite aggregate of $1\frac{1}{2}$ in. maximum size (38 mm) weighs about 225 lb/ft^3 (3600 kg/m^3). Iron ore sand and steel-punchings aggregate produce a unit weight of 270 lb/ft^3 (4320 kg/m^3). [18].

2.15 FIRE RESISTANCE

Fire resistance of a material is its ability to resist fire for a certain time without serious loss of strength, distortion, or collapse [19]. In the case of concrete, fire resistance depends on the thickness, type of construction, type and size of aggregates, and cement content. It is important to consider the effect of fire on tall buildings more than on low or single-story buildings, because occupants need more time to escape.

Reinforced concrete is a much better fire-resistant material than steel. Steelwork heats rapidly, and its strength drops appreciably in a short time. Concrete itself has low thermal conductivity. The effect of temperatures below 250°C is small on concrete, but definite loss is expected at higher temperatures.

2.16 HIGH-PERFORMANCE CONCRETE

High-performance concrete may be assumed to imply that the concrete exhibits combined properties of strength, toughness, energy absorption, durability, stiffness, and a relatively higher ductility than normal concrete. This improvement in concrete quality may be achieved by using a new generation of additives and superplasticizers, which improves the workability of concrete and, consequently, its strength. Also, the use of active microfillers such as silica fume, fly ash, and polymer improves the strength, porosity, and durability of concrete. The addition of different



Casting and finishing precast concrete wall panels.

types of fiber to the concrete mix enhances many of its properties, including ductility, strength, toughness, and many other properties.

Because it is difficult to set a limit to measure high-performance concrete, one approach is to define a lower-bound limit based on the shape of its stress-strain response in tension [20]. If the stress-strain relationship curve shows a quasi strain-hardening behavior—or, in other words, a postcracking strength larger than the cracking strength with an elastic-plastic behavior—then high performance is achieved [20]. In this behavior, multicracking stage is reached with high energy-absorption capacity. Substantial progress has been made recently in understanding the behavior and practical application of high-performance concrete.

2.17 LIGHTWEIGHT CONCRETE

Lightweight concrete is a concrete that has been made lighter than conventional normal-weight concrete and, consequently, it has a relatively lower density. Basically, reducing the density

requires the inclusion of air in the concrete composition. This, however, can be achieved in four distinct ways:

1. By omitting the finer sizes from the aggregate grading, thereby creating what is called *no-fines* concrete. It is a mixture of cement, water, and coarse aggregate only ($\frac{3}{4} - \frac{3}{8}$), mixed to produce concrete with many uniformly distributed voids.
2. By replacing the gravel or crushed rock aggregate with a hollow cellular or porous aggregate, which includes air in the mix. This type is called *lightweight aggregate concrete*. Lightweight aggregate may be natural, such as pumice, pozzolans, and volcanic slags; artificial (from industrial by-products), such as furnace clinker and foamed slag; or industrially produced, such as perlite, vermiculite, expanded clay, shale, and slate.
3. By creating gas bubbles in a cement slurry, which, when it sets, leaves a spongelike structure. This type is called *aerated concrete*.
4. By forming air cells in the slurry by chemical reaction or by vigorous mixing of the slurry with a preformed stable foam, which is produced by using special foam concentrate in a high-speed mixer. This type is called *cellular concrete*.

Structural lightweight concrete has a unit weight that ranges from 90 to 115 lb/ft^3, compared with 145 lb/ft^3 for normal-weight concrete. It is used in the design of floor slabs in buildings and other structural members where high-strength concrete is not required. Structural lightweight concrete can be produced with a compressive strength of 2500 to 5000 psi for practical applications.

2.18 FIBROUS CONCRETE

Fibrous concrete is made primarily of concrete constituents and discrete reinforcing fibers. The brittle nature of concrete and its low flexural tensile strength are major reasons for the growing interest in the performance of fibers in concrete technology. Various types of fibers—mainly steel, glass, and organic polymers—have been used in fibrous concrete. Generally, the length and diameter of the fibers do not exceed 3 in. (75 mm) and 0.04 in. (1 mm), respectively. The addition of fibers to concrete improves its mechanical properties, such as ductility, toughness, shear, flexural strength, impact resistance, and crack control. A convenient numerical parameter describing a fiber is its aspect ratio, which is the fiber length divided by an equivalent fiber diameter. Typical aspect ratios range from about 30 to 150, with the most common ratio being about 100. More details on fibrous concrete are given in [21].

2.19 STEEL REINFORCEMENT

Reinforcement, usually in the form of steel bars, is placed in the concrete member, mainly in the tension zone, to resist the tensile forces resulting from external load on the member. Reinforcement is also used to increase the member's compression resistance. Steel costs more than concrete, but it has a yield strength about 10 times the compressive strength of concrete. The function and behavior of both steel and concrete in a reinforced concrete member are discussed in Chapter 3.

Longitudinal bars taking either tensile or compression forces in a concrete member are called *main reinforcement*. Additional reinforcement in slabs, in a direction perpendicular to the

main reinforcement, is called *secondary*, or *distribution, reinforcement*. In reinforced concrete beams, another type of steel reinforcement is used, transverse to the direction of the main steel and bent in a box or U shape. These are called *stirrups*. Similar reinforcements are used in columns, where they are called *ties*. Refer to Figure 8.8 and Figure 10.3.

2.19.1 Types of Steel Reinforcement

Different types of steel reinforcement are used in various reinforced concrete members. These types can be classified as follows:

Round bars. Round bars are used most widely for reinforced concrete. Round bars are available in a large range of diameters, from $\frac{1}{4}$ (6 mm) to $1\frac{3}{8}$ (36 mm), plus two special types, $1\frac{3}{4}$ (45 mm) and $2\frac{1}{4}$ (57 mm). Round bars, depending on their surfaces, are either plain or deformed bars. Plain bars are used mainly for secondary reinforcement or in stirrups and ties. Deformed bars have projections or deformations on the surface for the purpose of improving the bond with concrete and reducing the width of cracks opening in the tension zone.

The diameter of a plain bar can be measured easily, but for a deformed bar, a nominal diameter is used that is the diameter of a circular surface with the same area as the section of the deformed bar. Requirements of surface projections on bars are specified by ASTM Specification A 305, or A 615. The bar sizes are designated by numbers 3 through 11, corresponding to the diameter in one-eighths of an inch. For instance, a no. 7 bar has a nominal diameter of $\frac{7}{8}$ in. and a no. 4 bar has a nominal diameter of $\frac{1}{2}$ in. The two largest sizes are designated no. 14 and no. 18, respectively. American standard bar marks are shown on the steel reinforcement to indicate the initial of the producing mill, the bar size, and the type of steel (Fig. 2.6). The grade of the reinforcement is indicated on the bars by either the continuous-line system or the number system. In the first system, one longitudinal line is added to the bar, in addition to the main ribs, to indicate the high-strength grade of 60 ksi (420 N/mm^2), according to ASTM Specification A 617. If only the main ribs are shown on the bar, without any additional lines, the steel is of the ordinary grade according to ASTM A 615 for the structural grade (f_y = 40 ksi, or 280 N/mm^2). In the number system, the yield strength of the high-strength grades is marked clearly on every bar. For ordinary grades, no strength marks are indicated. The two types are shown in Fig. 2.6.

Welded fabrics and mats. Welded fabrics and mats consist of a series of longitudinal and transverse cold-drawn steel wires, generally at right angles and welded together at all points of intersection. Steel reinforcement may be built up into three-dimensional cages before being placed in the forms.

Prestressed concrete wires and strands. Prestressed concrete wires and strands use special high-strength steel (see Chapter 20). High-tensile steel wires of diameters 0.192 in. (5 mm) and 0.276 in. (7 mm) are used to form the prestressing cables by winding six steel wires around a seventh wire of slightly larger diameter. The ultimate strength of prestressed strands is 250 ksi or 270 ksi.

2.19.2 Grades and Strength

Different grades of steel are used in reinforced concrete. Limitations on the minimum yield strength, ultimate strength, and elongation are explained in ASTM specifications for reinforcing steel bars (Table 2.17). The properties and grades of metric reinforcing steel are shown in Tables 2.18 and 2.19.

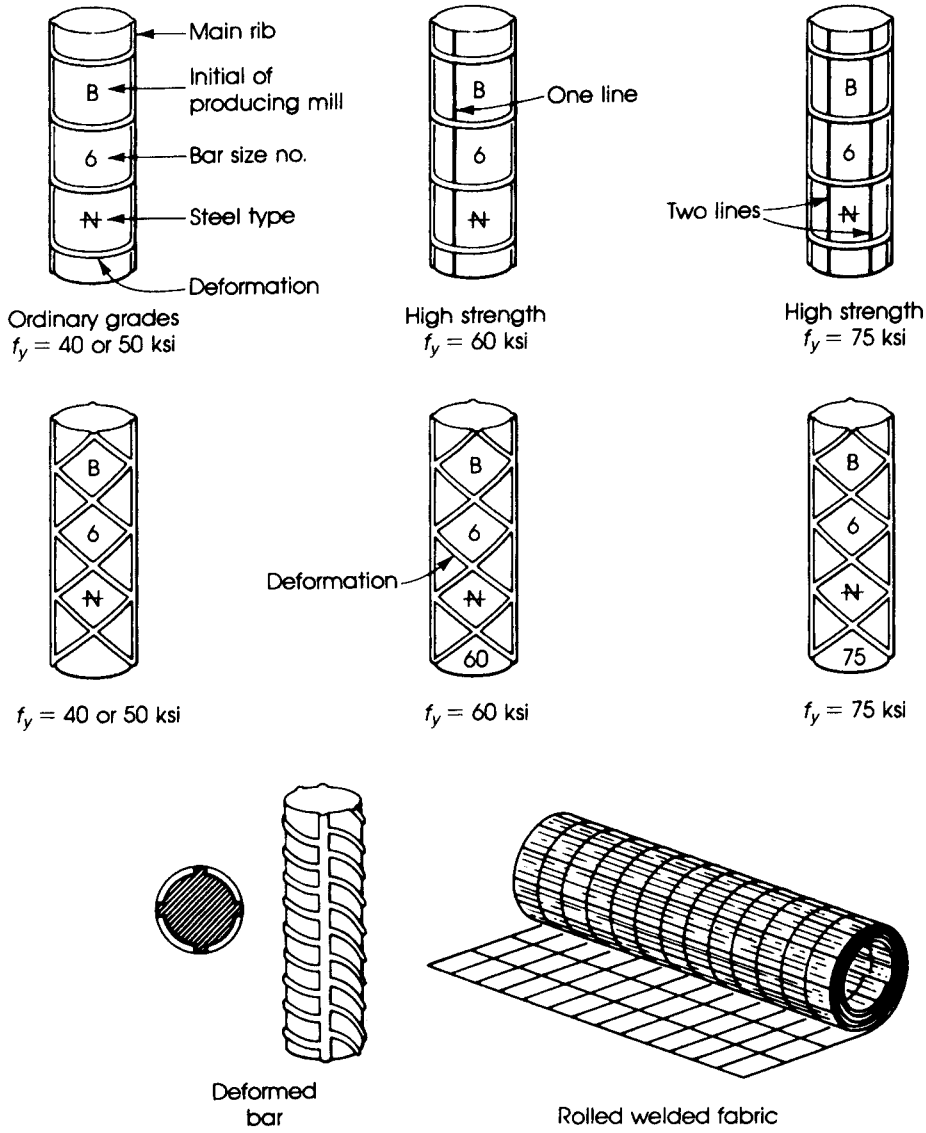


Figure 2.6 Some types of deformed bars and American standard bar marks.

2.19.3 Stress–Strain Curves

The most important factor affecting the mechanical properties and stress–strain curve of the steel is its chemical composition. The introduction of carbon and alloying additives in steel increases its strength but reduces its ductility. Commercial steel rarely contains more than 1.2% carbon; the proportion of carbon used in structural steels varies between 0.2% and 0.3%.

Two other properties are of interest in the design of reinforced concrete structures; the first is the modulus of elasticity, E_s. It has been shown that the modulus of elasticity is constant for all types of steel. The ACI Code has adopted a value of $E_s = 29 \times 10^6$ psi (2.0×10^5 MPa).

Table 2.17 Grade of ASTM Reinforcing Steel Bars

	Minimum Yield Strength f_y		Ultimate Strength f_{su}	
Steel	**ksi**	**MPa**	**ksi**	**MPa**
Billet steel				
Grade 40	40	276	70	483
60	60	414	90	621
75	75	518	100	690
Rail steel				
Grade 50	50	345	80	551
60	60	414	90	621
Deformed wire				
Reinforcing	75	518	85	586
Fabric	70	483	80	551
Cold-drawn wire				
Reinforcing	70	483	80	551
Fabric	65	448	75	518
Fabric	56	386	70	483

Table 2.18 ASTM 615 M (Metric) for Reinforcing Steel Bars

Bar No.	Diameter (mm)	Area (mm^2)	Weight (kg/m)
10 M	11.3	100	0.785
15 M	16.0	200	1.570
20 M	19.5	300	2.355
25 M	25.2	500	3.925
30 M	29.9	700	5.495
35 M	35.7	1000	7.850
45 M	43.7	1500	11.770
55 M	56.4	2500	19.600

Table 2.19 ASTM Metric Specifications

		Grade	
ASTM	**Bar size no.**	**MPa**	**ksi**
A615 M	10, 15, 20	300	43.5
Billet steel	10–55	400	58.0
	35, 45, 55	500	72.5
A616 M	10–35	350	50.75
Rail steel	10–35	400	58.0
A617 M	10–35	300	43.5
Axle steel	10–35	400	58.0
A706	10–55	400	58.0
Low alloy			

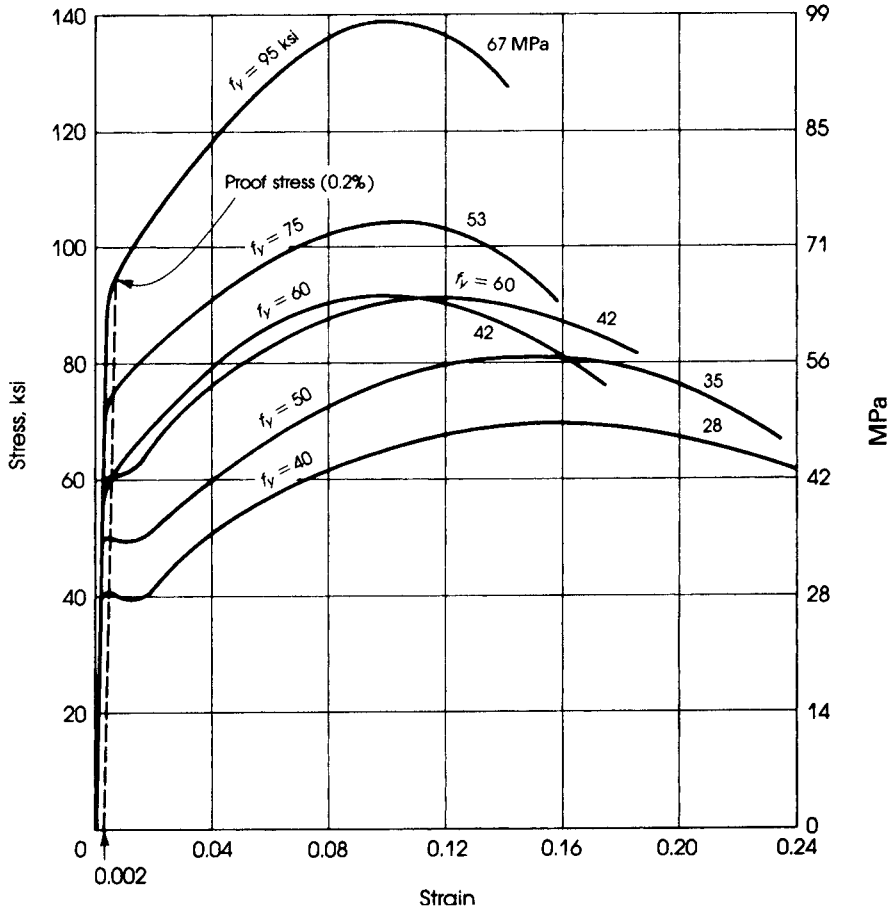


Figure 2.7 Typical stress-strain curves for some reinforcing steel bars of different grades. Note that 60-ksi steel may or may not show a definite yield point.

The modulus of elasticity is the slope of the stress-strain curve in the elastic range up to the proportional limit; E_s = stress/strain. Second is the yield strength, f_y. Typical stress—strain curves for some steel bars are shown in Fig. 2.7. In high-tensile steel, a definite yield point may not show on the stress-strain curve. In this case, ultimate strength is reached gradually under an increase of stress (Fig. 2.7). The yield strength or proof stress is considered the stress that leaves a residual strain of 0.2% on the release of load, or a total strain of 0.5% to 0.6% under load.

SUMMARY

Section 2.1

The main factors that affect the strength of concrete are the water—cement ratio, properties and proportions of materials, age of concrete, loading conditions, and shape of tested specimen.

$$f'_c \text{ (cylinder)} = 0.85 f'_c \text{ (cube)} = 1.10 f'_c \text{ (prism)}$$

Sections 2.2–2.6

1. The usual specimen used to determine the compressive strength of concrete at 28 days is a 6- by 12-in. (150- by 300-mm) cylinder. Compressive strength between 3000 and 6000 psi is usually specified for reinforced concrete structures. Maximum stress, f'_c, is reached at an estimated strain of 0.002, whereas rupture occurs at a strain of about 0.003.
2. Tensile strength of concrete is measured indirectly by a splitting test performed on a standard cylinder using formula $f'_{sp} = 2P/\pi LD$. Tensile strength of concrete is approximately $0.1f'_c$.
3. Flexural strength (modulus of rupture, f_r) of concrete is calculated by testing a 6- by 6- by 28-in. plain concrete beam, $f_r = 7.5\lambda\sqrt{f'_c}$ (psi), where λ is a modification factor related to unit weight of concrete.
4. Nominal shear stress is $2\lambda\sqrt{f'_c}$ (psi).

Sections 2.7–2.9

The modulus of elasticity of concrete, E_c for unit weight w between 90 and 160 pcf, is $E_c = 33w^{1.5}\sqrt{f'_c}$ (psi) $= 0.043w^{1.5}\sqrt{f'_c}$ MPa.

For normal-weight concrete, $w = 145$ pcf.

$$E_c = 57{,}600\sqrt{f'_c} \quad \text{or} \quad E_c = 57{,}000\sqrt{f'_c} = 4700\sqrt{f'_c} \text{ MPa}$$

The shear modulus of concrete is $G_c = E_c/2(1 + \mu) = 0.43\ E_c$ for a Poisson's ratio $\mu = \frac{1}{6}$. Poisson's ratio, μ, varies between 0.15 and 0.20, with an average value of 0.18.

Section 2.10

Modular ratio is $n = E_s/E_c = 500/\sqrt{f'_c}$, where f'_c is in psi.

Section 2.11

1. Values of shrinkage for normal concrete fall between 200×10^{-6} and 700×10^{-6}. An average value of 300×10^{-6} may be used.
2. The coefficient of expansion of concrete falls between 4×10^{-6} and 7×10^{-6}/°F.

Section 2.12–2.13

The ultimate magnitude of creep varies between 0.2×10^{-6} and 2×10^{-6} per unit stress per unit length. An average value of 1×10^{-6} may be adopted in practical problems. Of the ultimate (20-year) creep, 18% to 35% occurs in 2 weeks, 30% to 70% occurs in 3 months, and 64% to 83% occurs in 1 year.

Section 2.14

The unit weight of normal concrete is 145 pcf for plain concrete and 150 pcf for reinforced concrete.

Section 2.15

Reinforced concrete is a much better fire-resistant material than steel. Concrete itself has a low thermal conductivity. An increase in concrete cover in structural members such as walls, columns, beams, and floor slabs will increase the fire resistance of these members.

Sections 2.16–2.18

1. High-performance concrete implies that concrete exhibits properties of strength, toughness, energy absorption, durability, stiffness, and ductility higher than normal concrete.
2. Concrete is made lighter than normal-weight concrete by inclusion of air in the concrete composition. Types of lightweight concrete are no-fines concrete, lightweight aggregate concrete, aerated concrete, and cellular concrete.
3. Fibrous concrete is made of concrete constituents and discrete reinforcing fibers such as steel, glass, and organic polymers.

Section 2.19

The grade of steel mainly used is grade 60 ($f_y = 60$ ksi). The modulus of elasticity of steel is $E_s = 29 \times 10^6$ psi (2×10^{-5} MPa).

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PROBLEMS

2.1 Explain the modulus of elasticity of concrete in compression and the shear modulus.

2.2 Determine the modulus of elasticity of concrete by the ACI formula for a concrete cylinder that has a unit weight of 120 pcf (1920 kg/m^3) and a compressive strength of 3000 psi (21 MPa).

2.3 Estimate the modulus of elasticity and the shear modulus of a concrete specimen with a dry density of 150 pcf (2400 kg/m^3) and compressive strength of 4500 psi (31 MPa) using Poisson's ratio, $\mu = 0.18$.

2.4 What is meant by the modular ratio and Poisson's ratio? Give approximate values for concrete.

2.5 What factors influence the shrinkage of concrete?

2.6 What factors influence the creep of concrete?

2.7 What are the types and grades of the steel reinforcement used in reinforced concrete?

2.8 On the stress-strain diagram of a steel bar, show and explain the following: proportional limit, yield stress, ultimate stress, yield strain, and modulus of elasticity.

2.9 Calculate the modulus of elasticity of concrete, E_c, for the following types of concrete:

$$E_c = 33W^{1.5}\sqrt{f'_c} \text{ (ft)},$$

$$E_c = 0.043W^{1.5}\sqrt{f'_c} \text{ (SI)}$$

Density	Strength f'_c
160 pcf	5000 psi
145 pcf	4000 psi
125 pcf	2500 psi
2400 kg/m^3	35 MPa
2300 kg/m^3	30 MPa
2100 kg/m^3	25 MPa

2.10 Determine the modular ratio, n, and the modulus of rupture for each case of Problem 2.9. Tabulate your results.

$$f_r = 7.5\lambda\sqrt{f'_c} \text{ (psi)} \quad f_r = 0.62\lambda\sqrt{f'_c} \text{ (MPa)}$$

2.11 A standard normal 6 × 12-in. concrete cylinder was tested to failure, and the following loads and strains were recorded.

Load, kips	Strain x 10^{-4}	Load, kips	Strain x 10^{-4}
0.0	0.0	72	10.0
12	1.2	84	13.6
24	2.0	96	18.0
36	3.2	108	30.0
48	5.2	95	39.0
60	7.2	82	42.0

a. Draw the stress–strain diagram of concrete and determine the maximum stress and corresponding strain.

b. Determine the initial modulus and secant modulus.

c. Calculate the modulus of elasticity of concrete using the ACI formula for normal-weight concrete and compare results.

$$E_c = 57{,}000\sqrt{f'_c} \text{ psi}$$

$$E_c = 4730\sqrt{f'_c} \text{ MPa}$$

CHAPTER 3

FLEXURAL ANALYSIS OF REINFORCED CONCRETE BEAMS



Apartment building, Fort Lauderdale, Florida.

3.1 INTRODUCTION

The analysis and design of a structural member may be regarded as the process of selecting the proper materials and determining the member dimensions such that the design strength is equal or greater than the required strength. The required strength is determined by multiplying the actual applied loads, the dead load, the assumed live load, and other loads, such as wind, seismic, earth pressure, fluid pressure, snow, and rain loads, by load factors. These loads develop external forces such as bending moments, shear, torsion, or axial forces depending on how these loads are applied to the structure.

In proportioning reinforced concrete structural members, three main items can be investigated:

1. The safety of the structure, which is maintained by providing adequate internal design strength.
2. Deflection of the structural member under service loads. The maximum value of deflection must be limited and is usually specified as a factor of the span, to preserve the appearance of the structure.
3. Control of cracking conditions under service loads. Visible cracks spoil the appearance of the structure and also permit humidity to penetrate the concrete, causing corrosion of steel and consequently weakening the reinforced concrete member. The ACI Code implicitly limits crack widths to 0.016 in. (0.40 mm) for interior members and 0.013 in. (0.33 mm) for exterior members. Control of cracking is achieved by adopting and limiting the spacing of the tension bars (see Chapter 6).

It is worth mentioning that the strength design approach was first permitted in the United States in 1956 and in Britain in 1957. The latest ACI Code emphasizes the strength concept based on specified strain limits on steel and concrete that develop tension-controlled, compression-controlled, or transition conditions.

3.2 ASSUMPTIONS

Reinforced concrete sections are heterogeneous (nonhomogeneous), because they are made of two different materials, concrete and steel. Therefore, proportioning structural members by ultimate-strength design is based on the following assumptions:

1. Strain in concrete is the same as in reinforcing bars at the same level, provided that the bond between the steel and concrete is adequate.
2. Strain in concrete is linearly proportional to the distance from the neutral axis.
3. The modulus of elasticity of all grades of steel is taken as $E_s = 29 \times 10^6$ lb/in.^{2} (200,000 MPa or N/mm^2). The stress in the elastic range is equal to the strain multiplied by E_s.
4. Plane cross-sections continue to be plane after bending.
5. Tensile strength of concrete is neglected because (1) concrete's tensile strength is about 10% of its compressive strength, (2) cracked concrete is assumed to be not effective, and (3) before cracking, the entire concrete section is effective in resisting the external moment.
6. The method of elastic analysis, assuming an ideal behavior at all levels of stress, is not valid. At high stresses, nonelastic behavior is assumed, which is in close agreement with the actual behavior of concrete and steel.
7. At failure the maximum strain at the extreme compression fibers is assumed equal to 0.003 by the ACI Code provision.
8. For design strength, the shape of the compressive concrete stress distribution may be assumed to be rectangular, parabolic, or trapezoidal. In this text, a rectangular shape will be assumed (ACI Code, Section 10.2).

3.3 BEHAVIOR OF A SIMPLY SUPPORTED REINFORCED CONCRETE BEAM LOADED TO FAILURE

Concrete being weakest in tension, a concrete beam under an assumed working load will definitely crack at the tension side, and the beam will collapse if tensile reinforcement is not provided. Concrete cracks occur at a loading stage when its maximum tensile stress reaches the modulus of rupture of concrete. Therefore, steel bars are used to increase the moment capacity of the beam; the steel bars resist the tensile force, and the concrete resists the compressive force.

To study the behavior of a reinforced concrete beam under increasing load, let us examine how two beams were tested to failure. Details of the beams are shown in Fig. 3.1. Both beams had a section of 4.5 in. by 8 in. (110 mm by 200 mm), reinforced only on the tension side by two no. 5 bars. They were made of the same concrete mix. Beam 1 had no stirrups, whereas beam 2 was provided with no. 3 stirrups spaced at 3 in. The loading system and testing procedure were the same for both beams. To determine the compressive strength of the concrete and its modulus of elasticity, E_c, a standard concrete cylinder was tested, and strain was measured at different load increments. The following observations were noted at different distinguishable stages of loading.

Stage 1. At zero external load, each beam carried its own weight in addition to that of the loading system, which consisted of an I-beam and some plates. Both beams behaved similarly at this stage. At any section, the entire concrete section, in addition to the steel reinforcement, resisted

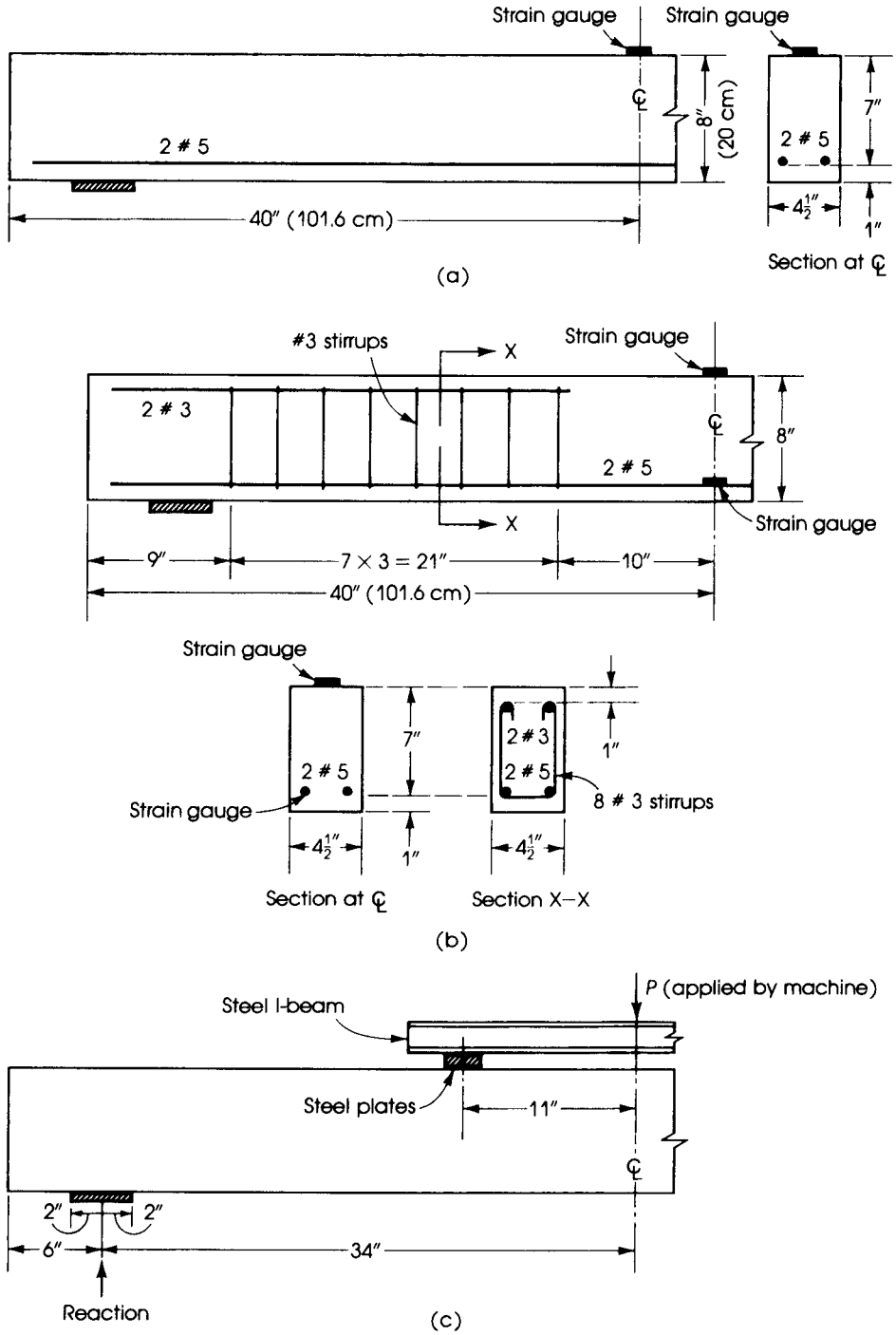
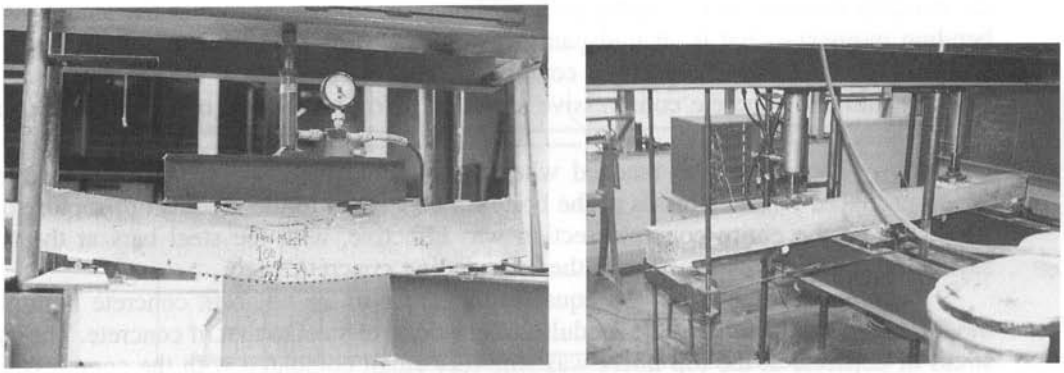
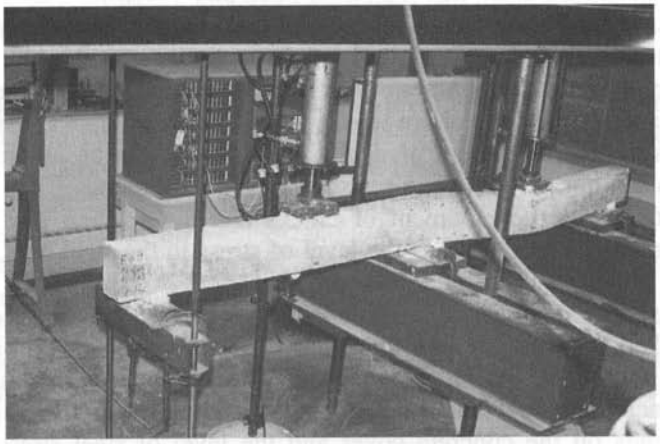


Figure 3.1 Details of tested beams: (*a*) beam 1, (*b*) beam 2, and (*c*) loading system. All beams are symmetrical about the centerline.



Test on a simply supported beam and a two-span continuous beam loaded to failure.



Two-span continuous reinforced concrete beam loaded to failure.



Failure conditions at the positive- and negative-moment sections in a continuous reinforced concrete beam.

the bending moment and shearing forces. Maximum stress occurred at the section of maximum bending moment—that is, at midspan. Maximum tension stress at the bottom fibers was much less than the modulus of rupture of concrete. Compressive stress at the top fibers was much less than the ultimate concrete compressive stress, f'_c. No cracks were observed at this stage.

Stage 2. This stage was reached when the external load, P, was increased from 0 to P_1, which produced tensile stresses at the bottom fibers equal to the modulus of rupture of concrete. At this stage the entire concrete section was effective, with the steel bars at the tension side sustaining a strain equal to that of the surrounding concrete.

Stress in the steel bars was equal to the stress in the adjacent concrete multiplied by the modular ratio, n, the ratio of the modulus of elasticity of steel to that of concrete. The compressive stress of concrete at the top fibers was still very small compared with the compressive strength, f'_c. The behavior of beams was elastic within this stage of loading.

Stage 3. When the load was increased beyond P_1, tensile stresses in concrete at the tension zone increased until they were greater than the modulus of rupture, f_r, and cracks developed. The neutral axis shifted upward, and cracks extended close to the level of the shifted neutral axis. Concrete in the tension zone lost its tensile strength, and the steel bars started to work effectively and to resist the entire tensile force. Between cracks, the concrete bottom fibers had tensile stresses, but they were of negligible value. It can be assumed that concrete below the neutral axis did not participate in resisting external moments.

In general, the development of cracks and the spacing and maximum width of cracks depend on many factors, such as the level of stress in the steel bars, distribution of steel bars in the section, concrete cover, and grade of steel used.

At this stage, the deflection of the beams increased clearly, because the moment of inertia of the cracked section was less than that of the uncracked section. Cracks started about the midspan of the beam, but other parts along the length of the beam did not crack. When load was again increased, new cracks developed, extending toward the supports. The spacing of these cracks depends on the concrete cover and the level of steel stress. The width of cracks also increased. One or two of the central cracks were most affected by the load, and their crack widths increased appreciably, whereas the other crack widths increased much less. It is more important to investigate those wide cracks than to consider the larger number of small cracks.

If the load were released within this stage of loading, it would be observed that permanent fine cracks of no significant magnitude were left. On reloading, cracks would open quickly, because the tensile strength of concrete had already been lost. Therefore, it can be stated that the second stage, once passed, does not happen again in the life of the beam. When cracks develop under working loads, the resistance of the entire concrete section and gross moment of inertia are no longer valid.

At high compressive stresses, the strain of the concrete increased rapidly, and the stress of concrete at any strain level was estimated from a stress—strain graph obtained by testing a standard cylinder to failure for the same concrete. As for the steel, the stresses were still below the yield stress, and the stress at any level of strain was obtained by multiplying the strain of steel, ε_s, by the modulus of elasticity of steel, E_s.

Stage 4. In beam 1, at a load value of 9500 lb (42.75 kN), shear stress at a distance of about the depth of the beam from the support increased and caused diagonal cracks at approximately 45° from horizontal in the direction of principal stresses resulting from the combined action of bending moment and shearing force. The diagonal crack extended downward to the level of

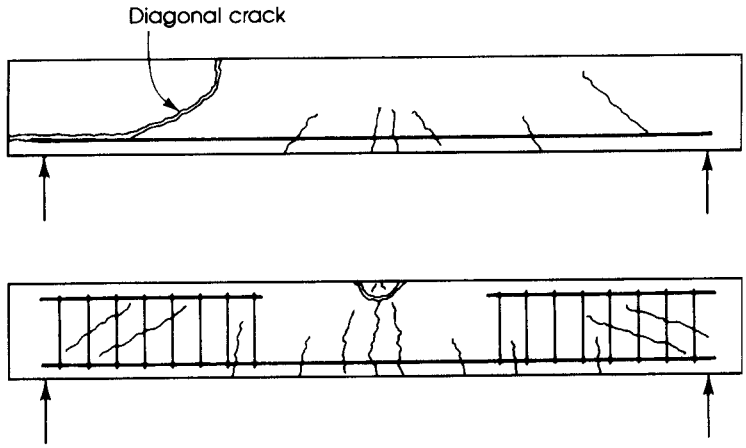


Figure 3.2 Shape of beam 1 at shear failure (*top*) and beam 2 at bending moment failure (*bottom*).

the steel bars and then extended horizontally at that level toward the support. When the crack, which had been widening gradually, reached the end of the beam, a concrete piece broke off and failure occurred suddenly (Fig. 3.2). The failure load was 13,600 lb (61.2 kN). Stresses in concrete and steel at the midspan section did not reach their failure stresses. (The shear behavior of beams is discussed in Chapter 8.)

In beam 2, at a load of 11,000 lb (49.5 kN), a diagonal crack developed similar to that of beam 1; then other parallel diagonal cracks appeared, and the stirrups started to take an effective part in resisting the principal stresses. Cracks did not extend along the horizontal main steel bars, as in beam 1. On increasing the load, diagonal cracks on the other end of the beam developed at a load of 13,250 lb (59.6 kN). Failure did not occur at this stage because of the presence of stirrups.

Stage 5. When the load on beam 2 was further increased, strains increased rapidly until the maximum carrying capacity of the beam was reached at ultimate load, $P_u = 16{,}200$ lb (72.9 kN).

In beam 2, the amount of steel reinforcement used was relatively small. When reached, the yield strain can be considered equal to yield stress divided by the modulus of elasticity of steel, $\varepsilon_y = f_y/E_s$; the strain in the concrete, ε_c, was less than the strain at maximum compressive stress, f'_c. The steel bars yielded, and the strain in steel increased to about 12 times that of the yield strain without increase in load. Cracks widened sharply, deflection of the beam increased greatly, and the compressive strain on the concrete increased. After another very small increase of load, steel strain hardening occurred, and concrete reached its maximum strain, ε'_c, and it started to crush under load; then the beam collapsed. Figure 3.2 shows the failure shapes of the two beams.

3.4 TYPES OF FLEXURAL FAILURE AND STRAIN LIMITS

3.4.1 Flexural Failure

Three types of flexural failure of a structural member can be expected depending on the percentage of steel used in the section.

1. Steel may reach its yield strength before the concrete reaches its maximum strength, Fig. 3.3a. In this case, the failure is due to the yielding of steel reaching a high strain equal to or greater than 0.005. The section contains a relatively small amount of steel and is called a tension-controlled section.
2. Steel may reach its yield strength at the same time as concrete reaches its ultimate strength, Fig. 3.3*b*. The section is called a balanced section.
3. Concrete may fail before the yield of steel, Fig. 3.3*c*, due to the presence of a high percentage of steel in the section. In this case, the concrete strength and its maximum strain of 0.003 are reached, but the steel stress is less than the yield strength, that is, f_s is less than f_y. The strain in the steel is equal to or less than 0.002. This section is called a compression-controlled section.

It can be assumed that concrete fails in compression when the concrete strain reaches 0.003. A range of 0.0025 to 0.004 has been obtained from tests and the ACI Code assumes a strain of 0.003.

In beams designed as tension-controlled sections, steel yields before the crushing of concrete. Cracks widen extensively, giving warning before the concrete crushes and the structure collapses. The ACI Code adopts this type of design. In beams designed as balanced or compression-controlled sections, the concrete fails suddenly, and the beam collapses immediately without warning. The ACI Code does not allow this type of design.

3.4.2 Strain Limits for Tension and Tension-Controlled Sections

The design provisions for both reinforced and prestressed concrete members are based on the concept of tension or compression-controlled sections, ACI Code, Section 10.3. Both are defined in terms of net tensile strain (NTS), (ε_t, in the extreme tension steel at nominal strength, exclusive of prestress strain. Moreover, two other conditions may develop: (1) the balanced strain condition and (2) the transition region condition. These four conditions are defined as follows:

1. Compression-controlled sections are those sections in which the net tensile strain, NTS, in the extreme tension steel at nominal strength is equal to or less than the compression-controlled strain limit at the time when concrete in compression reaches its assumed strain limit of 0.003, ($\varepsilon_c = 0.003$). For grade 60 steel, ($f_y = 60$ ksi), the compression-controlled strain limit may be taken as a net strain of 0.002, Fig. 3.4*a*. This case occurs mainly in columns subjected to axial forces and moments.
2. Tension-controlled sections are those sections in which the NTS, ε_t, is equal to or greater than 0.005 just as the concrete in the compression reaches its assumed strain limit of 0.003, Fig. 3.4*c*.
3. Sections in which the NTS in the extreme tension steel lies between the compression-controlled strain limit (0.002 for $f_y = 60$ ksi) and the tension-controlled strain limit of 0.005 constitute the transition region, Fig. 3.4*b*.
4. The balanced strain condition develops in the section when the tension steel, with the first yield, reaches a strain corresponding to its yield strength, f_y or $\varepsilon_s = f_y/E_s$, just as the maximum strain in concrete at the extreme compression fibers reaches 0.003, Fig. 3.5.

In addition to the above four conditions, Section 10.3.5 of the ACI Code indicates that the net tensile strain, ε_t, at nominal strength, within the transition region, shall not be less than 0.004 for reinforced concrete flexural members without or with an axial load less than $0.10 f'_c A_g$, where A_g = gross area of the concrete section.

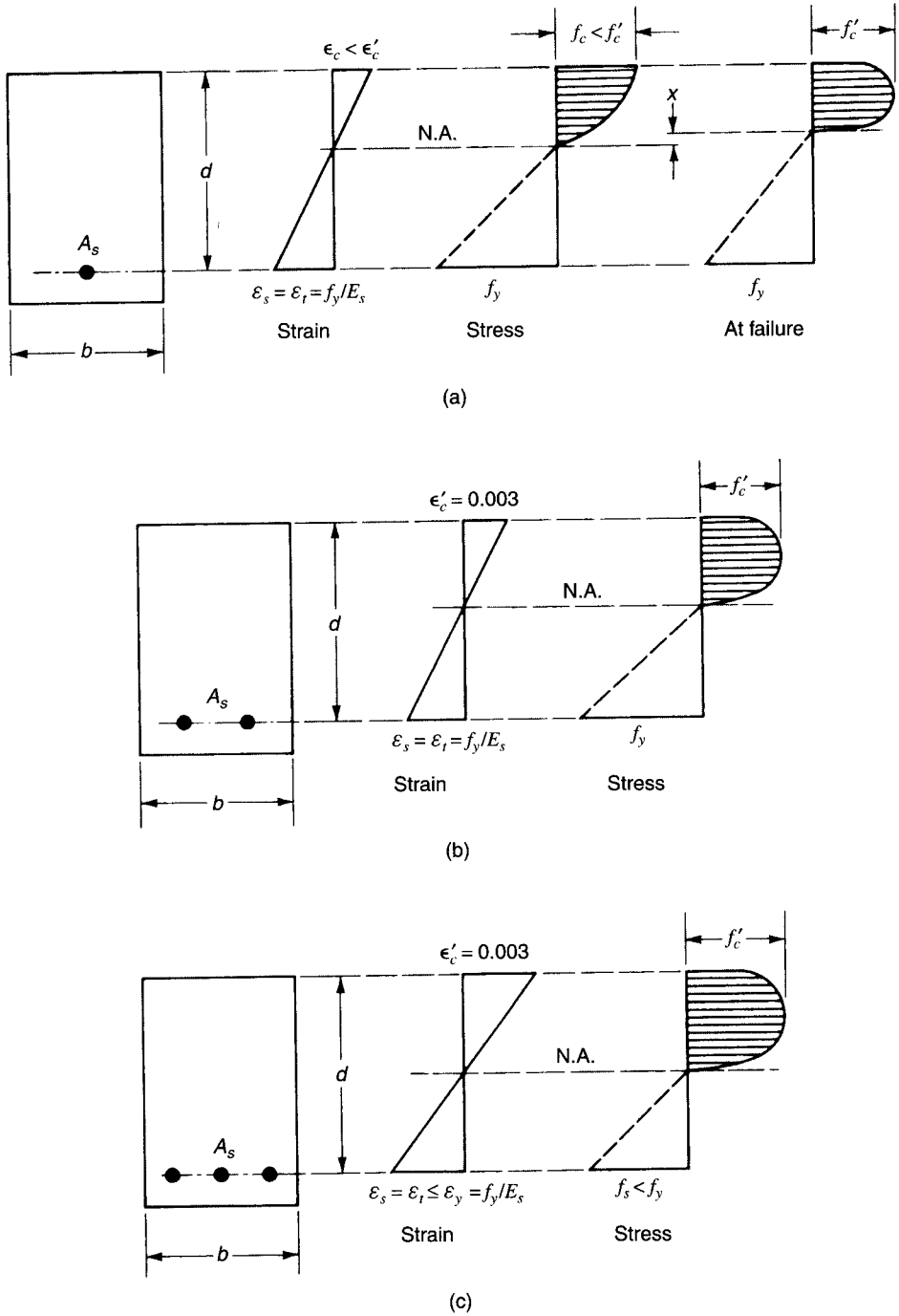


Figure 3.3 Stress and strain diagrams for (*a*) tension-controlled, (*b*) balanced, and (*c*) compression-controlled sections.

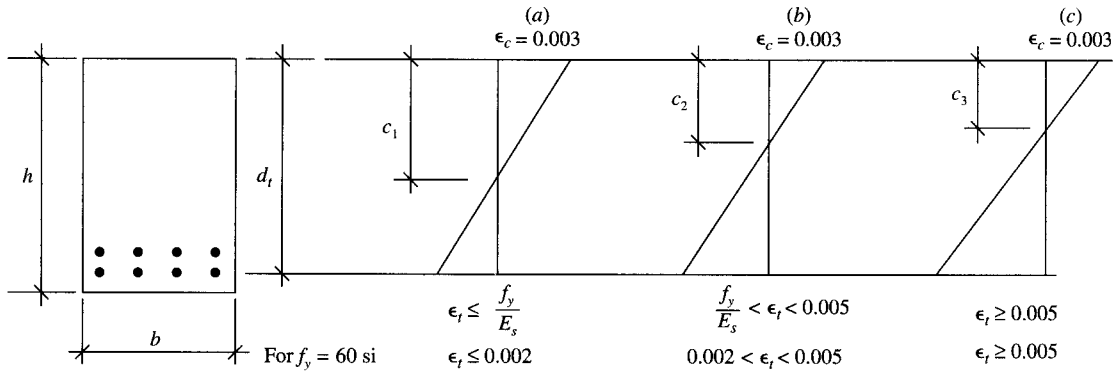


Figure 3.4 Strain limit distribution, $c_1 > c_2 > c_3$: (*a*) compression-controlled section, (*b*) transition region, and (*c*) tension-controlled section.

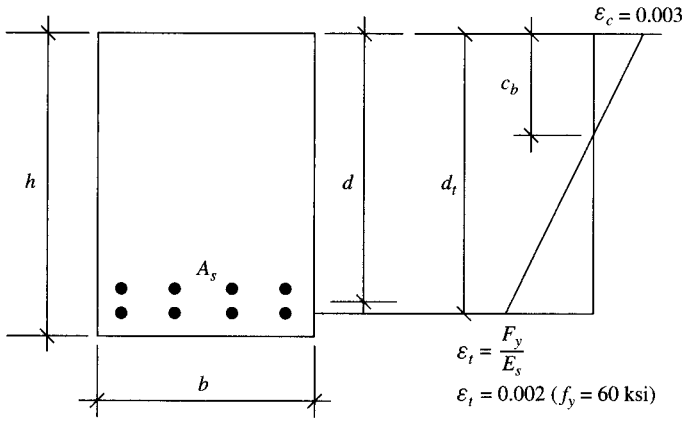


Figure 3.5 Balanced strain section (occurs at first yield or at distance d_t).

Note that d_t in Fig. 3.4, is the distance from the extreme concrete compression fiber to the extreme tension steel, while the effective depth, d, equals the distance from the extreme concrete compression fiber to the centroid of the tension reinforcement, Fig. 3.5. These cases are summarized in Table 3.1.

Table 3.1 Strain Limits of Fig. 3.4

Section Condition	Concrete Strain	Steel Strain	Notes (f_y = 60 ksi)
Compression-controlled	0.003	$\varepsilon_t \le f_y/E_s$	$\varepsilon_t \le 0.002$
Tension-controlled	0.003	$\varepsilon_t \ge 0.005$	$\varepsilon_t \ge 0.005$
Transition region	0.003	$f_y/E_s < \varepsilon_t < 0.005$	$0.002 < \varepsilon_t < 0.005$
Balanced strain	0.003	$\varepsilon_s = f_y/E_s$	$\varepsilon_s = 0.002$
Transition region (flexure)	0.003	$0.004 \le \varepsilon_t < 0.005$	$0.004 \le \varepsilon_t < 0.005$

3.5 LOAD FACTORS

The types of loads and the safety provisions were explained earlier in Sections 1.7 and 1.8.

For the design of structural members, the factored design load is obtained by multiplying the dead load by a load factor and the specified live load by another load factor. The magnitude of the load factor must be adequate to limit the probability of sudden failure and to permit an economical structural design. The choice of a proper load factor or, in general, a proper factor of safety depends mainly on the importance of the structure (whether a courthouse or a warehouse), the degree of warning needed prior to collapse, the importance of each structural member (whether a beam or column), the expectation of overload, the accuracy of artisanry, and the accuracy of calculations.

Based on historical studies of various structures, experience, and the principles of probability, the ACI Code adopts a load factor of 1.2 for dead loads and 1.6 for live loads. The dead load factor is smaller, because the dead load can be computed with a greater degree of certainty than the live load. Moreover, the choice of factors reflects the degree of the economical design as well as the degree of safety and serviceability of the structure. It is also based on the fact that the performance of the structure under actual loads must be satisfactorily within specific limits.

If the required strength is denoted by U (ACI Code, Section 9.2), and those due to wind and seismic forces are W and E, respectively, according to the ACI Code, the required strength U, shall be the most critical of the following factors (based on the ASCE 7-05):

1. In the case of dead, live, and wind loads,

$$U = 1.4D \tag{3.1a}$$

$$U = 1.2D + 1.6L \tag{3.1b}$$

$$U = 1.2D + 1.0L + 1.6W \tag{3.1c}$$

$$U = 0.9D + 1.6W \tag{3.1d}$$

2. In the case of dead, live, and seismic (earthquake) forces, E,

$$U = 1.2D + 1.0(L + E) \tag{3.2a}$$

$$U = 0.9D + 1.0E \tag{3.2b}$$

3. When the earth pressure load, H, is included,

$$U = 1.2D + 1.6(L + H) \tag{3.3a}$$

$$U = 0.9D + 1.6(W + H) \tag{3.3b}$$

$$U = 0.9D + 1.0E + 1.6H \tag{3.3c}$$

4. When pressure loads from fluids, F, are included,

$$U = 1.4(D + F) \tag{3.4a}$$

$$U = 1.2(D + F) + 1.6(L + H) \tag{3.4b}$$

5. For load combination due to roof live load, L_r, rain load, R, snow load, S, the effect of temperature T (including the effect of creep, shrinkage, and differential settlement) in addition to the above loads,

$$U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \quad (3.5a)$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W) \quad (3.5b)$$

$$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \quad (3.5c)$$

$$U = 1.2D + 1.0E + 1.0L + 0.2S \quad (3.5d)$$

It is to noted that

1. The load factor L in Eqs. 3.1*c*, 3.2*a*, and 3.5*a*, *b*, *c*, and *d* shall be permitted to be reduced to $0.5L$, except for garages, areas occupied as places of public assembly, and all areas where the live load, L, is greater than 100 pounds per square foot (psf).
2. When the wind load, W, is not reduced by a directionality factor, it is permitted to use $1.3W$ in place of $1.6W$ in Eqs. 3.1*d* and 3.3*b*.
3. If the service level of the seismic load E is used, $1.4E$ shall be used in place of $1.0E$ in Eqs. 3.2*a* and *b* and 3.3*c*.
4. If the structural action due to H counteracts that due to W or E, the load factor of H shall be set to 0.
5. In a flood zone area, the flood load or load combinations of ASCE shall be used.
6. Impact effects shall be included with the live load, L.

The ACI Code does not specify a value for impact, but AASHTO specifications give a simple factor for impact, I, as a percentage of the live load, L, as follows:

$$I = 50/(125 + S) \leq 30\% \quad (3.6)$$

where I = percentage of impact, S = part of the span loaded, and live load including impact $= L(1 + I)$.

When a better estimation is known from experiments or experience, the adjusted value shall be used.

The above equations indicate that the dead load factor is 1.2, whereas the live load factor is 1.6. These values are less than those specified by the 1999 ACI Code of 1.4 for the dead load and 1.7 for the live load. The new factors are based on the ASCE specifications ASCE 7-05.

For applied concentrated dead and live loads, P_D, P_L, the factored concentrated load $P_U = 1.2P_D + 1.6P_L$; also $M_U = 1.2\ M_D + 1.6\ M_L$, where M_D and M_L are the service dead-load and live-load moments, respectively.

3.6 STRENGTH-REDUCTION FACTOR ϕ

The nominal strength of a section, say M_n, for flexural members, calculated in accordance with the requirements of the ACI Code provisions must be multiplied by the strength reduction factor, ϕ, which is always less than 1. The strength reduction factor has several purposes:

1. To allow for the probability of under-strength sections due to variations in dimensions, material properties, and inaccuracies in the design equations
2. To reflect the importance of the member in the structure
3. To reflect the degree of ductility and required reliability under the applied loads

The ACI Code, Section 9.3, specifies the following values to be used:

For tension-controlled sections,	$\phi = 0.90$
For compression-controlled section	
a. with spiral reinforcement,	$\phi = 0.75$
b. other reinforced members,	$\phi = 0.65$
For plain concrete,	$\phi = 0.60$
For shear and torsion,	$\phi = 0.75$
For bearing on concrete,	$\phi = 0.65$
For strut and tie models,	$\phi = 0.75$

A higher ϕ factor is used for tension-controlled sections than for compression-controlled sections, because the latter sections have less ductility and they are more sensitive to variations in concrete strength. Also, spirally reinforced compression members have a ϕ value of 0.75 compared to 0.65 for tied compression members; this variation reflects the greater ductility behavior of spirally reinforced concrete members under the applied loads. In the ACI Code provisions, the ϕ factor is based on the behavior of the cross-section at nominal strength, (P_n, M_n), defined in terms of the NTS, ε_t, in the extreme tensile strains, as given in Table 3.1. For tension-controlled members, $\phi = 0.9$. For compression-controlled members, $\phi = 0.75$ (with spiral reinforcement) and $\phi = 0.65$ for other members.

For the transition region, ϕ may be determined by linear interpolation between 0.65 (or 0.75) and 0.9. Figure 3.6*a* shows the variation of ϕ for grade 60 steel. The linear equations are as follows:

$$\phi = 0.75 + (\varepsilon_t - 0.002)(50) \text{ (for spiral members)} \tag{3.7}$$

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) \text{ (for other members)} \tag{3.8}$$

Alternatively, ϕ may be determined in the transition region, as a function of (c/d_t) for grade 60 steel as follows:

$$\phi = 0.75 + 0.15\left[\frac{1}{c/d_t} - \frac{5}{3}\right] \text{ (for spiral members)} \tag{3.9}$$

$$\phi = 0.65 + 0.25\left[\frac{1}{c/d_t} - \frac{5}{3}\right] \text{ (for other members)} \tag{3.10}$$

where c = the depth of the neutral axis at nominal strength (c_2 in Fig. 3.4). At the limit strain of 0.002 for grade 60 steel and from the triangles of Fig. 3.4*a*, $c/d_t = 0.003/(0.002 + 0.003) = 0.6$. Similarly, at a strain, $\varepsilon_t = 0.005$, $c/d_t = 0.003/(0.005 + 0.003) = 0.375$. Both values are shown in Fig. 3.6.

For reinforced concrete flexural members, the NTS, ε_t, should be equal to or greater than 0.004 (ACI Code, Section 10.3). In this case,

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) = 0.82 \tag{3.11}$$

Figure 3.6*b* shows the range of ϕ for flexural members. For grade 60 steel, the range varies between 0.9 for $\varepsilon_t \geq 0.005$ and 0.82 for $\varepsilon_t = 0.004$. Other values of ϕ can be obtained from Eq. 3.11 or by interpolation.

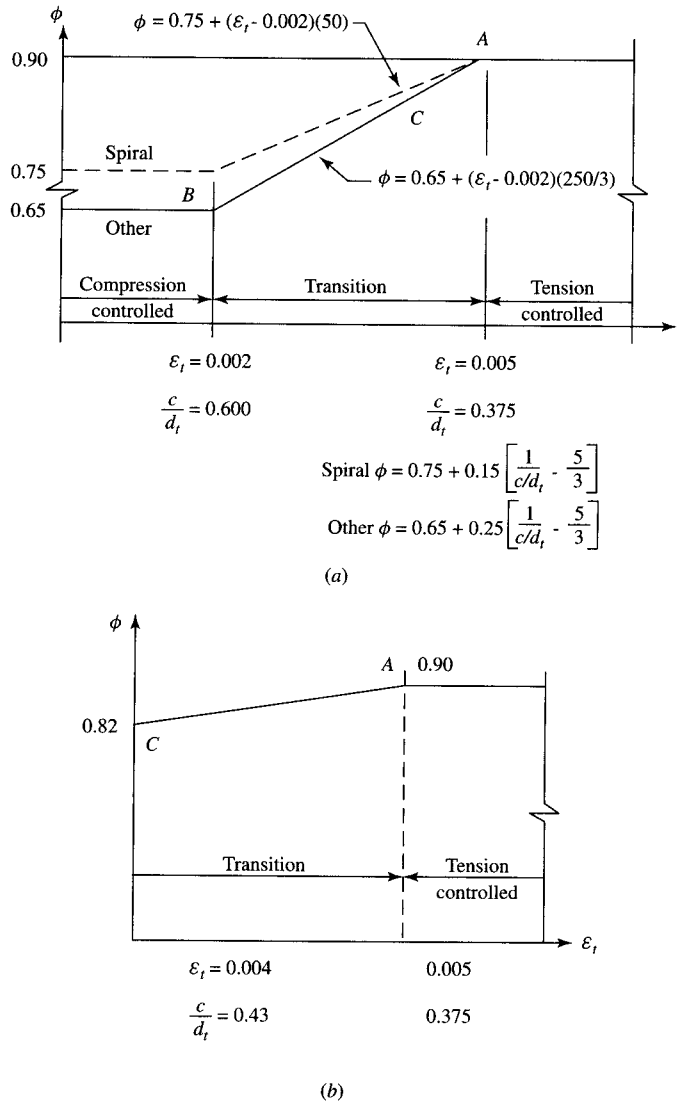


Figure 3.6 (*a*) Variation of ϕ, with the net tensile strain for grade 60 steel and for prestressed steel, [1]; (*b*) variation of ϕ and strain limit in flexural member with f_y = 60 ksi.

3.7 SIGNIFICANCE OF ANALYSIS AND DESIGN EXPRESSIONS

Two approaches for the investigations of a reinforced concrete member will be used in this book:

Analysis of a section implies that the dimensions and steel used in the section (in addition to concrete strength and steel yield strength) are given, and it is required to calculate the internal design moment capacity of the section so that it can be compared with the applied external required moment.

Design of a section implies that the external required moment is known from structural analysis, and it is required to compute the dimensions of an adequate concrete section and the amount of steel reinforcement. Concrete strength and yield strength of steel used are given.

3.8 EQUIVALENT COMPRESSIVE STRESS DISTRIBUTION

The distribution of compressive concrete stresses at failure may be assumed to be a rectangle, trapezoid, parabola, or any other shape that is in good agreement with test results.

When a beam is about to fail, the steel will yield first if the section is under-reinforced, and in this case the steel is equal to the yield stress. If the section is over-reinforced, concrete crushes first and the strain is assumed to be equal to 0.003, which agrees with many tests of beams and columns. A compressive force, C, develops in the compression zone and a tension force, T, develops in the tension zone at the level of the steel bars. The position of force T is known, because its line of application coincides with the center of gravity of the steel bars. The position of compressive force C is not known unless the compressive volume is known and its center of gravity is located. If that is done, the moment arm, which is the vertical distance between C and T, will consequently be known.

In Fig. 3.7, if concrete fails, $\varepsilon_c = 0.003$, and if steel yields, as in the case of a balanced section, $f_s = f_y$.

The compression force, C, is represented by the volume of the stress block, which has the nonuniform shape of stress over the rectangular hatched area of bc. This volume may be considered equal to $C = bc(\alpha_1 f'_c)$, where $\alpha_1 f'_c$ is an assumed average stress of the nonuniform stress block.

The position of compression force C is at a distance z from the top fibers, which can be considered as a fraction of the distance c (the distance from the top fibers to the neutral axis), and z can be assumed to be equal to $\alpha_2 c$, where $\alpha_2 < 1$. The values of α_1 and α_2 have been estimated from many tests, and their values, as suggested by Mattock, Kriz, and Hognestad [3], are as follows:

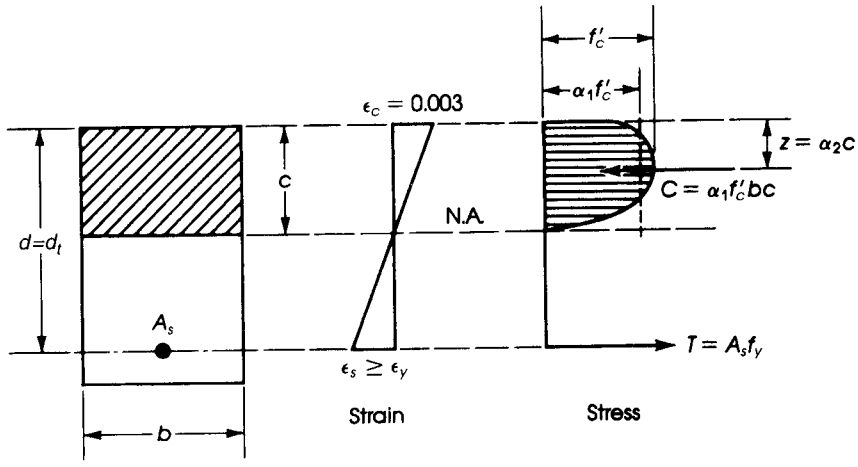


Figure 3.7 Ultimate forces in a rectangular section.

$\alpha_1 = 0.72$ for $f'_c \leq 4000$ psi (27.6 MPa); it decreases linearly by 0.04 for every 1000 psi (6.9 MPa) greater than 4000 psi

$\alpha_2 = 0.425$ for $f'_c < 4000$ psi (27.6 MPa); it decreases linearly by 0.025 for every 1000 psi greater than 4000 psi

The decrease in the value of α_1 and α_2 is related to the fact that high-strength concretes show more brittleness than low-strength concretes [2].

To derive a simple rational approach for calculations of the internal forces of a section, the ACI Code adopted an equivalent rectangular concrete stress distribution, which was first proposed by C. S. Whitney and checked by Mattock and others [3]. A concrete stress of 0.85 f'_c is assumed to be uniformly distributed over an equivalent compression zone bounded by the edges of the cross-section and a line parallel to the neutral axis at a distance $a = \beta_1 c$ from the fiber of maximum compressive strain, where c is the distance between the top of the compressive section and the neutral axis (Fig. 3.8). The fraction β_1 is 0.85 for concrete strengths $f'_c \leq 4000$ psi (27.6 MPa) and is reduced linearly at a rate of 0.05 for each 1000 psi (6.9 MPa) of stress greater than 4000 psi (Fig. 3.9), with a minimum value of 0.65.

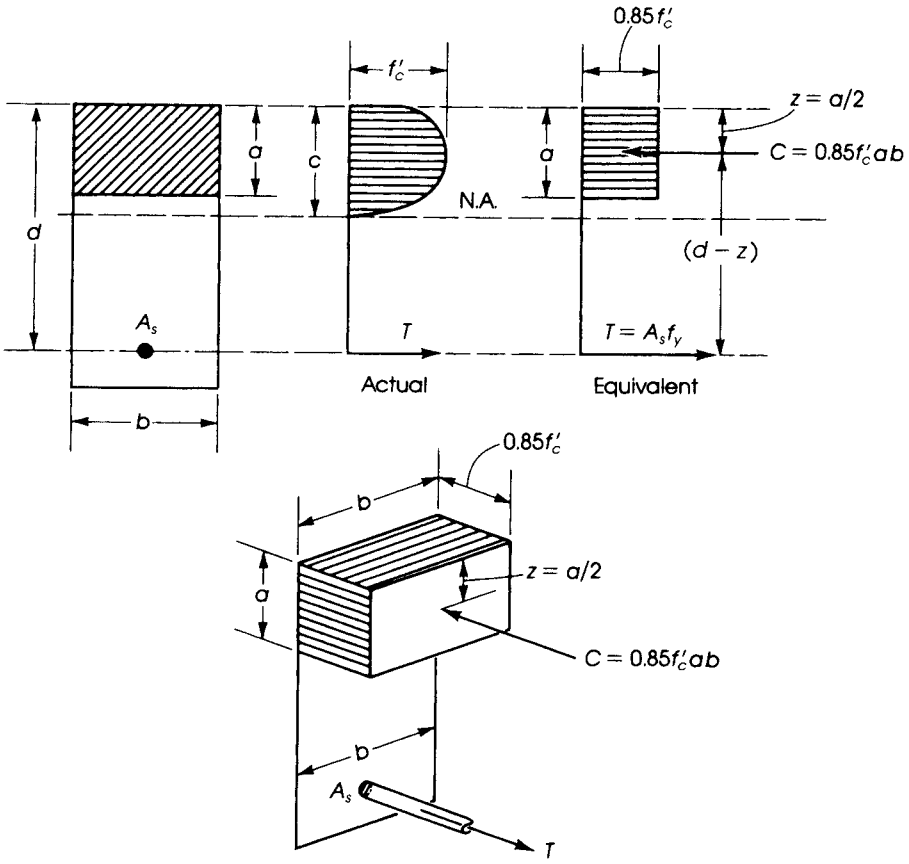


Figure 3.8 Actual and equivalent stress distributions at failure.

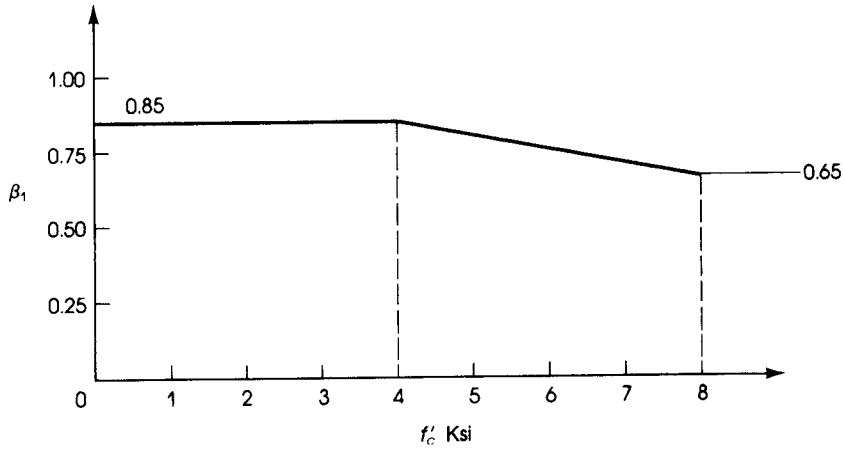


Figure 3.9 Values of β_1 for different compressive strengths of concrete, f'_c.

The preceding discussion applies in general to any section, and it is not confined to a rectangular shape. In the rectangular section, the area of the compressive zone is equal to ba, and every unit area is acted on by a uniform stress equal to $0.85f'_c$, giving a total stress volume equal to $0.85f'_cab$, which corresponds to the compressive force, C. For any other shape, the force C is equal to the area of the compressive zone multiplied by a constant stress equal to $0.85f'_c$.

For example, in the section shown in Fig. 3.10, the force C is equal to the shaded area of the cross-section multiplied by $0.85f'_c$:

$$C = 0.85f'_c(6 \times 3 + 10 \times 2) = 32.3f'_c \text{ lb}$$

The position of the force C is at a distance z from the top fibers, at the position of the resultant force of all small-element forces of the section. As in the case when the stress is uniform and equals $0.85f'_c$, the resultant force C is located at the center of gravity of the compressive zone, which has a depth of a.

In this example, z is calculated by taking moments about the top fibers:

$$z = \frac{\left(6 \times 3 \times \frac{3}{2}\right) + 10 \times 2(1+3)}{6 \times 3 + 10 \times 2} = \frac{107}{38} = 2.82 \text{ in.}$$

3.9 SINGLY REINFORCED RECTANGULAR SECTION IN BENDING

We explained previously that a balanced condition is achieved when steel yields at the same time as the concrete fails, and that failure usually happens suddenly. This implies that the yield strain in the steel is reached ($\varepsilon_y = f_y/E_s$) and that the concrete has reached its maximum strain of 0.003. The percentage of reinforcement used to produce a balanced condition is called the *balanced steel ratio*, ρ_b. This value is equal to the area of steel, A_s, divided by the effective cross-section, bd:

$$\rho_b = \frac{A_s(\text{balanced})}{bd}$$

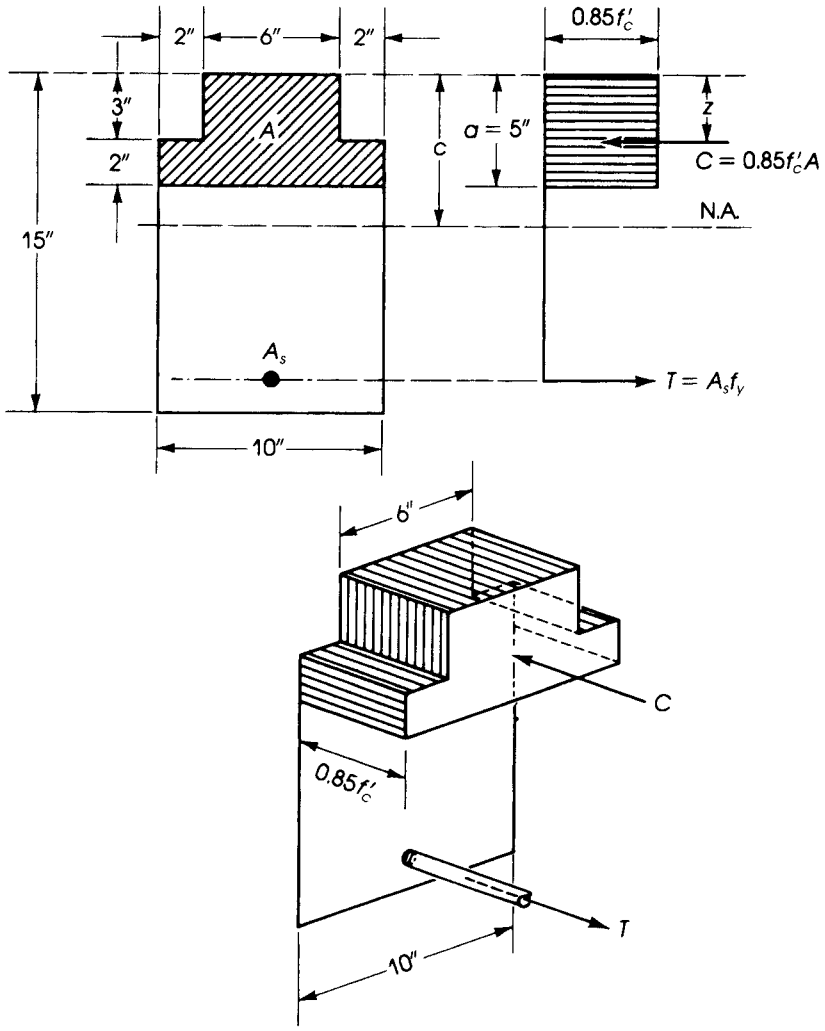


Figure 3.10 Ultimate forces in a nonrectangular section.

where

b = width of the compression face of the member

d = distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement

Two basic equations for the analysis and design of structural members are the two equations of equilibrium that are valid for any load and any section:

1. The compression force should be equal to the tension force; otherwise, a section will have linear displacement plus rotation:

$$C = T \tag{3.12}$$

2. The internal nominal bending moment, M_n, is equal to either the compressive force, C, multiplied by its arm or the tension force, T, multiplied by the same arm:

$$M_n = C(d - z) = T(d - z)$$

$$(M_u = \phi M_n \text{ after reduction by the factor } \phi) \tag{3.13}$$

The use of these equations can be explained by considering the case of a rectangular section with tension reinforcement (Fig. 3.8). The section may be balanced, under-reinforced, or over-reinforced, depending on the percentage of steel reinforcement used.

3.9.1 The Balanced Section

Let us consider the case of a balanced section, which implies that at ultimate load the strain in concrete equals 0.003 and that of steel equals the first yield stress at distance d_t divided by the modulus of elasticity of steel, f_y/E_s. This case is explained by the following steps.

Step 1. From the strain diagram of Fig. 3.11,

$$\frac{c_b}{d - c_b} = \frac{0.003}{f_y/E_s}$$

From triangular relationships (where is c_b is c for a balanced section) and by adding the numerator to the denominator,

$$\frac{c_b}{d} = \frac{0.003}{0.003 + f_y/E_s}$$

Substituting $E_s = 29 \times 10^3$ ksi,

$$c_b = \left(\frac{87}{87 + f_y}\right) d \tag{3.14}$$

where f_y is in ksi.

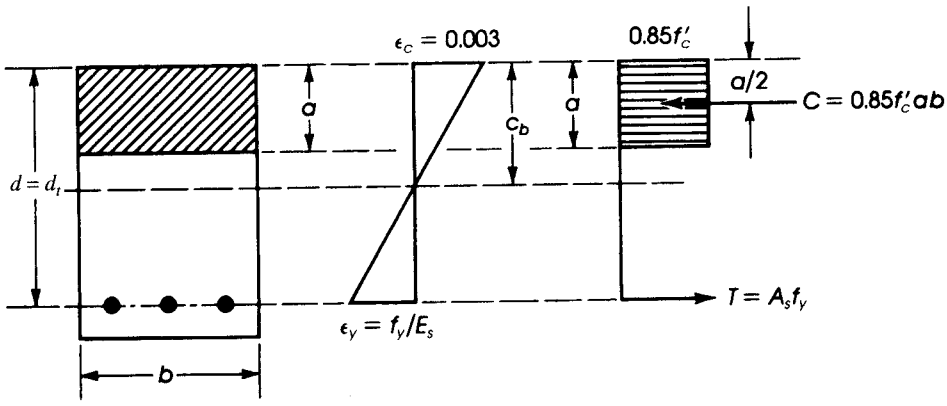


Figure 3.11 Rectangular balanced section.

Step 2. From the equilibrium equation,

$$C = T$$

$$0.85 f'_c ab = A_s f_y \tag{3.15}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} \tag{3.16}$$

Here a is the depth of the compressive block, equal to $\beta_1 c$, where $\beta_1 = 0.85$ for $f'_c \leq 4000$ psi (27.6 MPa) and decreases linearly by 0.05 per 1000 psi (6.9 MPa) for higher concrete strengths (Fig. 3.9). Because the balanced steel reinforcement ratio is used,

$$\rho_b = \frac{A_s(\text{balanced})}{bd} = \frac{A_{sb}}{bd} \tag{3.17}$$

and substituting the value of A_{sb} in Eq. 3.15,

$$0.85 f'_c ab = f_y \rho_b bd$$

Therefore,

$$\rho_b = \frac{0.85 f'_c}{f_y d} a = \frac{0.85 f'_c}{f_y d} (\beta_1 c_b)$$

Substituting the value of c_b from Eq. 3.14, the general equation of the balanced steel ratio becomes

$$\rho_b = 0.85 \beta_1 \frac{f'_c}{f_y} \left(\frac{87}{87 + f_y} \right) \tag{3.18}$$

Step 3. The internal nominal moment, M_n, is calculated by multiplying either C or T by the distance between them:

$$M_n = C(d - z) = T(d - z) \tag{3.13}$$

For a rectangular section, the distance $z = a/2$ as the line of application of the force C lies at the center of gravity of the area ab, where

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$M_n = C\left(d - \frac{a}{2}\right) = T\left(d - \frac{a}{2}\right)$$

For a balanced or an under-reinforced section, $T = A_s f_y$. Then

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) \tag{3.19}$$

To get the usable design moment ϕM_n, the previously calculated M_n must be reduced by the capacity reduction factor, ϕ,

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) = \phi A_s f_y \left(d - \frac{A_s f_y}{1.7 f'_c b}\right) \tag{3.19a}$$

Equation 3.19*a* can be written in terms of the steel percentage ρ:

$$\rho = \frac{A_s}{bd} \quad A_s = \rho bd$$

$$\phi M_n = \phi f_y \rho bd \left(d - \frac{\rho bd f_y}{1.7 f_c' b}\right) = \phi \rho f_y b d^2 \left(1 - \frac{\rho f_y}{1.7 f_c'}\right) \tag{3.20}$$

Equation 3.20 can be written as

$$\phi M_n = R_u b d^2 \tag{3.21}$$

where

$$R_u = \phi \rho f_y \left(1 - \frac{\rho f_y}{1.7 f_c'}\right) \tag{3.22}$$

The ratio of the equivalent compressive stress block depth, a, to the effective depth of the section, d, can be found from Eq. 3.15:

$$0.85 f_c' a b = \rho b d f_y \tag{3.23}$$

$$\frac{a}{d} = \frac{\rho f_y}{0.85 f_c'}$$

3.9.2 Upper Limit of Steel Percentage

The upper limit or the maximum steel percentage, $\rho_{\max}$, that can be used in a singly reinforced concrete section in bending is based on the net tensile strain in the tension steel, the balanced steel ratio, and the grade of steel used. The relationship between the steel percentage, ρ, in the section, and the net tensile strain, ε_t, is as follows:

$$\varepsilon_t = \left(\frac{0.003 + f_y/E_s}{\rho/\rho_b}\right) - 0.003 \tag{3.24}$$

For $f_y = 60$ ksi, and assuming $f_y/E_s = 0.002$,

$$\varepsilon_t = \left(\frac{0.005}{\rho/\rho_b}\right) - 0.003 \tag{3.25}$$

These expressions are obtained by referring to Fig. 3.12. For a balanced section,

$$c_b = \frac{a_b}{\beta_1} = \frac{A_{sb} f_y}{0.85 f_c' b \beta_1} = \frac{\rho_b f_y d}{0.85 f_c' \beta_1}$$

Similarly, for any steel ratio, ρ,

$$c = \frac{\rho f_y d}{0.85 f_c' \beta_1} \quad \text{and} \quad \frac{c}{c_b} = \frac{\rho}{\rho_b}$$

Divide both sides by d to get

$$\frac{c}{d} = \left(\frac{\rho}{\rho_b}\right)\left(\frac{c_b}{d}\right) \tag{3.26}$$

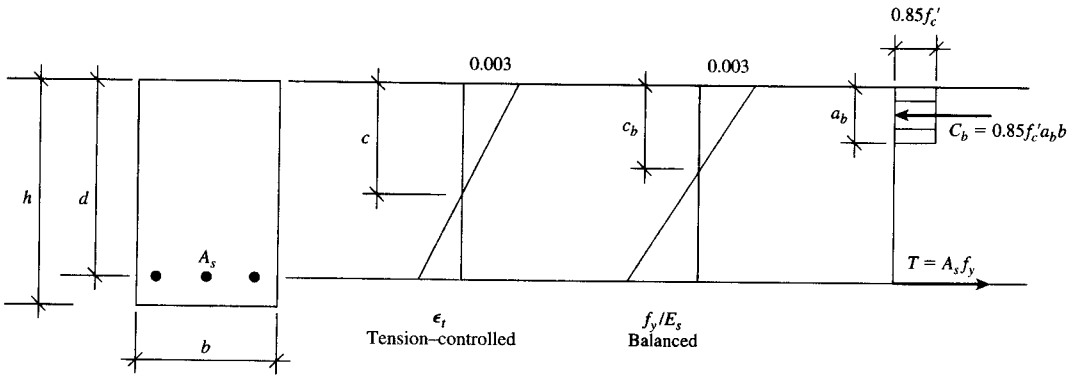


Figure 3.12 Strains in tension-controlled and balanced conditions.

From the triangles of the strain diagrams,

$$\frac{c}{d} = \frac{0.003}{0.003 + \varepsilon_t}$$
$$\varepsilon_t = \frac{0.003}{(c/d)} - 0.003 \tag{3.27}$$

Similarly,

$$\frac{c_b}{d} = \frac{0.003}{0.003 + f_y/E_s} \tag{3.28}$$

From Eqs. 3.26 and 3.28,

$$\frac{c}{d} = \left(\frac{\rho}{\rho_b}\right)\left(\frac{c_b}{d}\right) = \left(\frac{\rho}{\rho_b}\right)\left(\frac{0.003}{0.003 + f_y/E_s}\right)$$

Substitute this value in Eq. 3.27 to get

$$\varepsilon_t = \frac{0.003}{(c/d)} - 0.003 = \left[\frac{0.003 + f_y/E_s}{\rho/\rho_b}\right] - 0.003 \tag{3.24}$$

For grade 60 steel, $f_y = 60$ ksi, $E_s = 29{,}000$ ksi, and $f_y/E_s = 0.00207$, then

$$\varepsilon_t = \left(\frac{0.00507}{\rho/\rho_b}\right) - 0.003 \tag{3.25}$$

To determine the upper limit or the maximum steel percentage, ρ, in a singly reinforced concrete section, refer to Fig. 3.6. It can be seen that concrete sections subjected to flexure or axial load and bending moment may lie in compression-controlled, transition, or tension-controlled zones. When $\varepsilon_t \leq 0.002$ (or $c/d_t \geq 0.6$), compression controls, whereas when $\varepsilon_t \geq 0.005$ (or $c/d_t \leq 0.375$), tension controls. The transition zone occurs when $0.002 < \varepsilon_t < 0.005$ or $0.6 > c/d_t > 0.375$.

For members subjected to flexure, the relationship between the steel ratio, ρ, was given in Eq. 3.24:

$$\varepsilon_t + 0.003 = \frac{0.003 + f_y/E_s}{\rho/\rho_b} \tag{3.24}$$

or

$$\frac{\rho}{\rho_b} = \frac{0.003 + f_y/E_s}{0.003 + \varepsilon_t} \tag{3.29}$$

For $f_y = 60$ ksi and $E_s = 29{,}000$ ksi, f_y/E_s may be assumed to be 0.00207.

$$\frac{\rho}{\rho_b} = \frac{0.00507}{0.003 + \varepsilon_t} \tag{3.30}$$

The limit for tension to control is $\varepsilon_t \geq 0.005$. For $\varepsilon_t = 0.005$, Eq. 3.30 becomes

$$\frac{\rho}{\rho_b} = \frac{0.005}{0.008} = \frac{5}{8} = 0.625 \tag{3.30a}$$

or $\rho \leq 0.63375\rho_b$ for tension-controlled sections if $\varepsilon_t = 0.00507 = f_y/E_s$. Both values can be used for practical analysis and design. The small increase in ρ will slightly increase the moment capacity of the section. For example, if $f'_c = 4$ ksi and $f_y = 60$ ksi, $\rho_b = 0.0285$ and $\rho \leq 0.01806$ for tension to control (as in the case of flexural members). The ϕ factor in this case is 0.9. This value is less than $\rho_{\max} = 0.75\rho_b = 0.0214$ allowed by the ACI Code for flexural members when $\phi = 0.9$ can be used.

Design of beams and other flexural members can be simplified using the limit of $\varepsilon_t = 0.005$.

$$\frac{\rho}{\rho_b} = \frac{0.003 + f_y/E_s}{0.008} \tag{3.31}$$

In this case, $\rho = \rho_{\max}$ = upper limit for tension-controlled sections.

$$\rho_{\max} = \left(\frac{0.003 + f_y/E_s}{0.008}\right)\rho_b \tag{3.31a}$$

Note that when ρ used $\leq \rho_{\max}$, tension controls and $\phi = 0.9$. When $\rho > \rho_{\max}$, section will be in the transition region with $\phi < 0.9$.
And for $f_y = 60$ ksi and $f_y/E_s = 0.00207$,

$$\frac{\rho_{\max}}{\rho_b} = 0.63375 \tag{3.32}$$

This steel ratio will provide adequate ductility before beam failure.

Similarly,

$$\text{for } f_y = 40 \text{ ksi}, \quad \rho_{\max} = 0.5474\rho_b \tag{3.32a}$$

$$\text{for } f_y = 50 \text{ ksi}, \quad \rho_{\max} = 0.5905\rho_b \tag{3.32b}$$

$$\text{for } f_y = 75 \text{ ksi}, \quad \rho_{\max} = 0.6983\rho_b \tag{3.32c}$$

It was established that $\phi M_n = R_u bd^2$ (Eq. 3.21), where $R_u = \phi\rho f_y(1 - \rho f_y/1.7f'_c)$ (Eq. 3.22). Once f'_c and f_y are known, then ρ_b, ρ, R_u, and bd^2 can be calculated. For example, for $f'_c = 4$ ksi, $f_y = 60$ ksi, $\phi = 0.9$, $\varepsilon_t = 0.005$, and one row of bars in the section,

$$\rho_b = 0.0285 \quad \rho = 0.01806 \quad R_u = 820 \text{ psi}$$

Note that for one row of bars in the section, it can be assumed that $d = d_t = h - 2.5$ in., whereas for two rows of bars, $d = h - 3.5$ in., and $d_t = h - 2.5$ in. $= d + 1.0$ in. (Refer to Figs. 3.4 and 3.5 and Section 4.3.3.)

Table 3.2 Values of ρ_{max} and $R_u = M_u/bd^2$ for Flexural Tension-Controlled Sections with One Row of Bars, $\varepsilon_t = 0.005$

f'_c (ksi)	f_y (ksi)	ρ_b	$\rho_{max} = 0.63375\,\rho_b$	R_u (psi) (Eq. 3.22)
3	60	0.0214	0.01356	615
4	60	0.0285	0.01806	820
5	60	0.0335	0.02123	975
6	60	0.0377	0.02389	1109

Table 3.2 gives the values of ρ, ρ_b, and $R_u = M_u/bd^2$ for flexural tension-controlled sections with one row of bars.

For reinforced concrete flexural members with $\rho > \rho_{max}$, ε_t will be less than 0.005. Section 10.3 of the ACI Code specifies that ε_t should not be less than 0.004 in the transition region to maintain adequate ductility and warning before failure.

For this limitation of $\varepsilon_t = 0.004$, the general equation (3.29) becomes

$$\frac{\rho}{\rho_b} = \frac{0.003 + f_y/E_s}{0.007} \tag{3.33}$$

For $f_y = 60$ ksi,

$$\frac{\rho}{\rho_b} = \frac{0.003 + 0.00207}{0.007} = 0.724 \tag{3.34}$$

and the limit in the transition region is

$$\rho_{max\,t} = 0.724\,\rho_b \tag{3.34a}$$

Note that the t here refers to the transition region. In this case, limit of ϕ is

$$\phi_t = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) = 0.817 < 0.9 \tag{3.35}$$

For $f_y = 60$ ksi and $f'_c = 4$ ksi, $\rho_b = 0.0285$, $\rho_{max\,t} = 0.02063$, $R_n = 1012$ psi (from Eq.3.22, and $R_u = \phi R_n = 0.817(1012) = 826$ psi.

This steel ratio in Eq. 3.33 is the upper limit ($\rho_{max\,t}$) for a singly reinforced concrete section in the transition region with $\phi < 0.9$.

It can be noticed that the aforementioned $R_u = 826$ psi calculated for $\varepsilon_t = 0.004$, is very close to $R_u = 820$ psi for $\rho_{max} = 0.63375\rho_b$ and $\phi = 0.9$. Therefore, adding reinforcement beyond ρ_{max} (for $\varepsilon_t = 0.005$, Table 3.2) reduces ϕ because of the reduced ductility resulting in little or nonsubstantial gain in design strength. Adding compression reinforcement in the section is a better solution to increase the design moment, keeping the section in the tension-controlled region with $\phi = 0.9$. (Refer to Section 3.14.)

Table 3.3 gives the values of ρ_t(limit), ρ_b, and R_u for flexural members in the transition region for $f_y = 60$ ksi and $\varepsilon_t = 0.004$ and one row of bars. In this case $\phi = 0.817$ (Eq. 3.35) and $\rho/\rho_b = 0.724$. It is clear that for $f_y = 60$ ksi, the design R_u in both cases, when $\varepsilon_t = 0.005$ with $\phi = 0.9$ and when $\varepsilon_{max} = 0.004$ with $\phi = 0.816$, are quite close.

Table 3.3 Values of ρ_t and R_u for Sections in the Transition Region with $\varepsilon_t = 0.004$, $f_y = 60$ ksi, and One Row of Bars ($\phi = 0.817$)

f'_c (ksi)	ρ_b	ρ_t (limit)	R_u (psi)
3	0.0214	0.0155	617
4	0.0285	0.0206	822
5	0.0335	0.0243	980
6	0.0377	0.0273	1116

Example 3.1

For the section shown in Fig. 3.13, calculate

a. The balanced steel reinforcement

b. The maximum reinforcement area allowed by the ACI Code for tension-controlled section and in the transition region

c. The position of the neutral axis and the depth of the equivalent compressive stress block for the tension-controlled section in *b*.

Given: $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

a. $\rho_b = 0.85\,\beta_1 \frac{f'_c}{f_y}\left(\frac{87}{87 + f_y}\right)$

Because $f'_c = 4000$ psi, $\beta_1 = 0.85$:

$$\rho_b = (0.85)^2\left(\frac{4}{60}\right)\left(\frac{87}{87+60}\right) = 0.0285$$

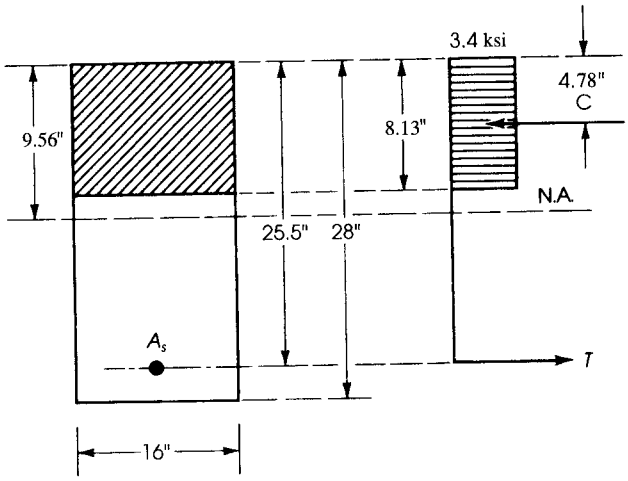


Figure 3.13 Example 3.1.

The area of steel reinforcement to provide a balanced condition is

$$A_{sb} = \rho_b bd = 0.0285 \times 16 \times 25.5 = 11.63 \text{ in.}^2$$

b. For a tension-controlled section, $\rho_{max} = 0.63375\ \rho_b = 0.63375 \times 0.0285 = 0.01806$ or, from Eq. 3.32,

$$A_{s\,max} = \rho_{max} bd = 0.01806 \times 16 \times 25.5 = 7.37 \text{ in.}^2 \text{ for } \phi = 0.9.$$

For the transition region, $\rho_{max\,t} = 0.724\ \rho_b = 0.0206$. For the case of $\varepsilon_t = 0.004$, $A_{s\,max\,t} = 0.0206(16 \times 25.5) = 8.41 \text{ in.}^2$ for $\phi = 0.817$

c. The depth of the equivalent compressive block using $A_{s\,max}$ is

$$a_{max} = \frac{A_{s\,max} f_y}{0.85 f'_c b} = \frac{7.37 \times 60}{0.85 \times 4 \times 16} = 8.13 \text{ in.}$$

The distance from the top fibers to the neutral axis is $c = \alpha/\beta_1$. Because $f'_c = 4000$ psi, $\beta_1 = 0.85$; thus,

$$c = \frac{8.13}{0.85} = 9.56 \text{ in.}$$

or $c/d = 0.375$ and $c = 0.375(25.5) = 9.56$ in.

Example 3.2

Determine the design moment strength and the position of the neutral axis of the rectangular section shown in Fig. 3.14 if the reinforcement used is three no. 9 bars. Given: $f'_c = 3$ ksi and $f_y = 60$ ksi.

Solution

1. The area of three no. 9 bars is 3.0 in.^2

$$\rho = \frac{A_s}{bd} = \frac{3.0}{21 \times 12} = 0.0119$$

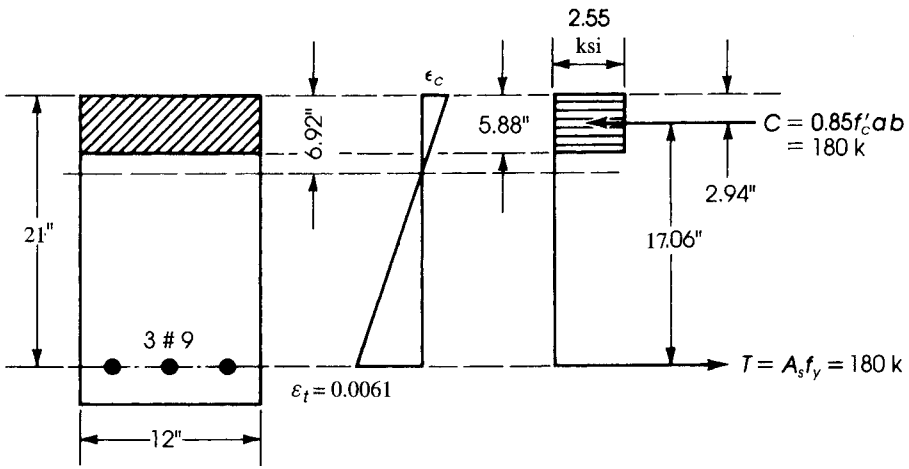


Figure 3.14 Example 3.2.

2. $\rho_{max} = 0.01356 > \rho$, tension-controlled section, $\phi = 0.9$ or check ε_t:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3(60)}{0.85 \times 3 \times 12} = 5.88 \text{ in.}$$

$$c = \frac{a}{0.85} = 6.92 \text{ in.}$$

$$d_t = d = 21 \text{ in.}$$

$$\varepsilon_t = \left(\frac{21 - 6.92}{6.92}\right) 0.003$$

$$= 0.0061 > 0.005, \quad \phi = 0.9$$

$$\text{or } \frac{c}{d_t} = 0.33 < 0.375 \quad \text{(o.k.)}$$

3. $\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.0 \times 60}{0.85 \times 3 \times 12} = 5.88 \text{ in.}$$

$$\phi M_n = 0.9 \times 3.0 \times 60 \left(21 - \frac{5.88}{2}\right) = 2926 \text{ K·in.} = 243.8 \text{ K·ft}$$

Discussion

In this example, the section is tension-controlled, which implies that the steel will yield before the concrete reaches its ultimate strain. A simple check can be made from the strain diagram (Fig. 3.14). From similar triangles,

$$\frac{\varepsilon_c}{\varepsilon_y} = \frac{c}{(d - c)} \text{ and } \varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29000} = 0.00207$$

$$\varepsilon_c = \frac{6.92}{(21 - 6.92)} \times 0.00207 = 0.00102$$

which is much less than 0.003. Therefore, steel yields before concrete reaches its limiting strain of 0.003.

Example 3.3

Repeat Example 3.2 using three no. 10 bars as the tension steel (Fig. 3.15).

Solution

1. Check ε_t:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.81(60)}{0.85 \times 3 \times 12} = 7.47 \text{ in.}$$

$$c = \frac{a}{0.85} = 8.79 \text{ in.} \quad d_t = d = 21 \text{ in.} \quad \frac{c}{d_t} = 0.419 > 0.375$$

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003 = \left(\frac{21 - 8.79}{8.79}\right) 0.003 = 0.004168$$

This value is less than 0.005 but greater than 0.004 (transition region), $\phi < 0.9$.

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) = 0.831$$

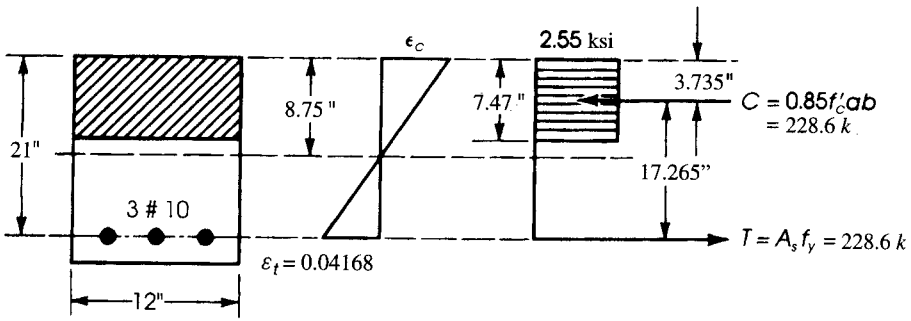


Figure 3.15 Example 3.3.

2. Calculate ϕM_n:

$$\phi M_n = 0.831(3.81)(60)\left[21 - \frac{7.47}{2}\right] = 3278 \text{ K·in.} = 273 \text{ K·ft}$$

Discussion

For a tension-controlled section, $\varepsilon_t = 0.005$ and $\rho = 0.63375\ \rho_b = 0.01356$ (Table 3.2), $\phi = 0.9$.

$$A_s \max = 0.01356(12 \times 21) = 3.417 \text{ in.}^2 < 3.81 \text{ in.}^2$$

$$a = \frac{3.417 \times 60}{0.85 \times 3 \times 12} = 6.7 \text{ in.}$$

$$\phi M_n = 0.9 \times 3.417 \times 60\left(21 - \frac{6.7}{2}\right) = 271.4 \text{ K·ft}$$

which is close to the above ϕM_n. This is a somewhat conservative approach.

3.10 LOWER LIMIT OR MINIMUM PERCENTAGE OF STEEL

If the factored moment applied on a beam is very small and the dimensions of the section are specified (as is sometimes required architecturally) and are larger than needed to resist the factored moment, the calculation may show that very small or no steel reinforcement is required. In this case, the maximum tensile stress due to bending moment may be equal to or less than the modulus of rupture of concrete: $f_r = \lambda 7.5\sqrt{f'_c}$. If no reinforcement is provided, sudden failure will be expected when the first crack occurs, thus giving no warning. The ACI Code, 10.5, specifies a minimum steel area, A_s,

$$A_{s\,\min} = \left(\frac{3\sqrt{f'_c}}{f_y}\right) b_w d \geq \left(\frac{200}{f_y}\right) b_w d$$

or the minimum steel ratio, $\rho_{\min} = (3\sqrt{f'_c}/f_y) \geq 200/f_y$, where the units of f'_c and f_y are in psi. This ρ ratio represents the lower limit. The first term of the preceding equation was specified

to accommodate a concrete strength higher than 5 ksi. The two minimum ratios are equal when $f'_c = 4440$ psi. This indicates that

$$\rho_{\min} = \frac{200}{f_y} \text{ when } f'_c < 4500 \text{ psi}$$

$$\rho_{\min} = \frac{3\sqrt{f'_c}}{f_y} \text{ when } f'_c \geq 4500 \text{ psi}$$

For example, if $f_y = 60$ ksi, $\rho_{\min} = 0.00333$ when $f'_c < 4500$ psi, whereas $\rho_{\min} = 0.00353$ when $f'_c = 5000$ psi and 0.00387 when $f'_c = 6000$ psi.

In the case of a rectangular section, use $b = b_w$ in the preceding expressions. For statically determinate T-sections with the flange in tension, as in the case of cantilever beams, the value of $A_{s\,\min}$ should be equal to or greater than the *smaller* of (a) and (b):

(a) $$A_{s\,\min} = \left(\frac{6\sqrt{f'_c}}{f_y}\right) b_w d$$

(b) $$A_{s\,\min} = \left(\frac{3\sqrt{f'_c}}{f_y}\right) b_w d \geq \left(\frac{200}{f_y}\right) b_w d$$

where b_w and b are the width of the beam web and flange, respectively, and f'_c and f_y are in psi. For example, if $b = 48$ in., $b_w = 16$ in., $f'_c = 4000$ psi, and $f_y = 60{,}000$ psi, than $A_{s\,\min} = 2.02$ in.^{2} in (a) controls, which is smaller than the value of $A_{s\,\min}$ in (b) (3.2 in.^{2}).

3.11 ADEQUACY OF SECTIONS

A given section is said to be *adequate* if the internal moment strength of the section is equal to or greater than the externally applied factored moment, M_u, or $\phi M_n \geq M_u$. The procedure can be summarized as follows:

1. Calculate the external applied factored moment, M_u.

$$M_u = 1.2M_D + 1.6M_L$$

2. Calculate ϕM_n for the basic singly reinforced section:
   a. Check that $\rho_{\min} \leq \rho \leq \rho_{\max}$.
   b. Calculate $a = A_s f_y/(0.85 f'_c b)$ and check ε_t for ϕ.
   c. Calculate $\phi M_n = \phi\ A_s f_y(d - a/2)$.
3. If $\phi M_n \geq M_u$, then the section is adequate; Fig. 3.16 shows a typical tension-controlled section.

Example 3.4

An 8-ft-span cantilever beam has a rectangular section and reinforcement as shown in Fig. 3.17. The beam carries a dead load, including its own weight, of 1.5 K/ft and a live load of 0.9 K/ft. Using $f'_c = 4$ ksi and $f_y = 60$ ksi, check if the beam is safe to carry the above loads.

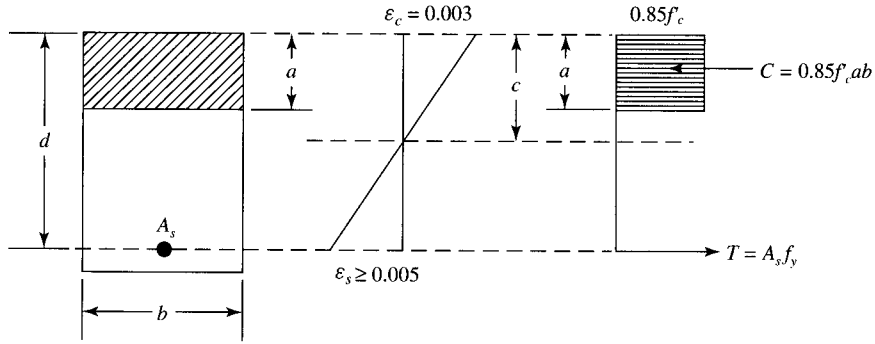


Figure 3.16 Tension-controlled rectangular section.

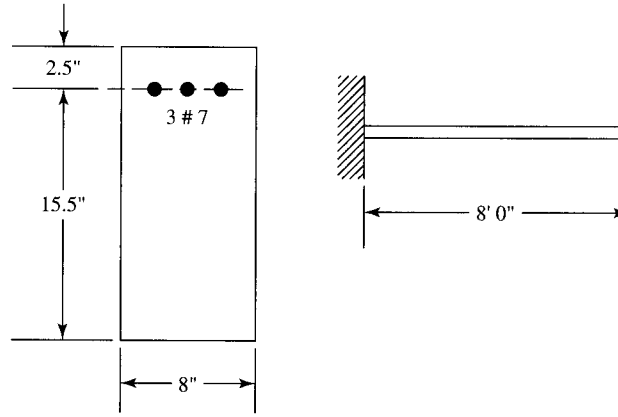


Figure 3.17 Example 3.4.

Solution

1. Calculate the external factored moment:

$$W_u = 1.2D + 1.6L = 1.2(1.5) + 1.6(0.9) = 3.24 \text{ K/ft}$$

$$M_u = W_u \frac{L^2}{2} = 3.24\frac{8^2}{2} = 103.68 \text{ K·ft} = 1244 \text{ K·in.}$$

2. Check ε_t:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.8 \times 60}{0.85 \times 4 \times 8} = 3.97 \text{ in.}$$

$$c = \frac{a}{0.85} = 4.67 \text{ in.} \quad d_t = d = 15.5 \text{ in.} \quad \frac{c}{d_t} = 0.3 < 0.375$$

Also,

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003 = \left(\frac{15.5 - 4.67}{4.67}\right) 0.003 = 0.007 > 0.005, \quad \phi = 0.9$$

or check

$$\rho = \frac{A_s}{bd} = \frac{1.8}{8 \times 15.5} = 0.0145 < \rho_{\max} = 0.01806$$

(from Table 3.2). Therefore, it is a tension-controlled section and $\phi = 0.9$.

3. Calculate ϕM_n:

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$= 0.9(1.8)(60)\left(15.5 - \frac{3.97}{2}\right) = 1312 \text{ K·in.} > M_u$$

Then section is adequate.

Example 3.5

A simply supported beam has a span of 20 ft. If the cross section of the beam is as shown in Fig. 3.18, $f'_c = 3$ ksi, and $f_y = 60$ ksi, determine the allowable uniformly distributed service live load on the beam assuming the dead load is that due to beam weight. Given: $b = 12$ in., $d = 17$ in., total depth $h = 20$ in., and reinforced with three no. 8 bars ($A_s = 2.37$ in.^{2}).

Solution

1. Determine the design moment strength:

$$\rho = \frac{A_s}{bd} = \frac{3 \times 0.79}{12 \times 17} = 0.0116$$

$$\rho_{\max} = 0.01356 \text{ (Table 3.2)}$$

$$\rho < \rho_{\max}$$

Therefore it is a tension-controlled section and $\phi = 0.9$

Also, $\rho > \rho_{\min} = \dfrac{200}{f_y} = 0.00333$.

2. $$\phi M_n = \phi A_s f_y \left(d - \frac{A_s f_y}{1.7 f'_c b}\right)$$

$$= 0.9 \times 2.37 \times 60\left(17 - \frac{2.37 \times 60}{1.7 \times 3 \times 12}\right) = 1878 \text{ K·in.} = 156.5 \text{ K·ft}$$

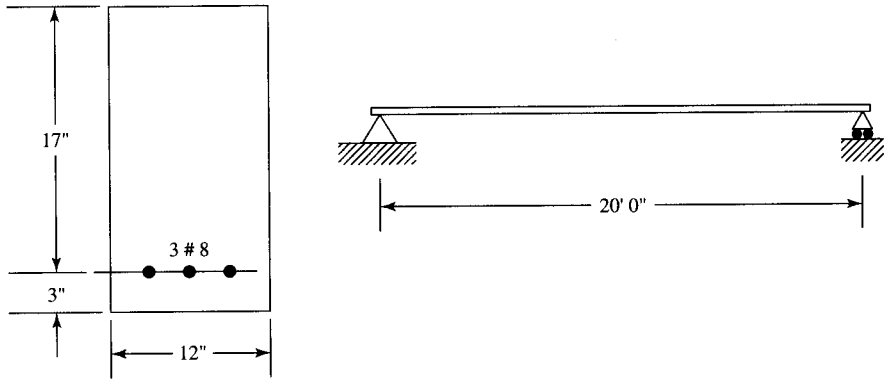


Figure 3.18 Example 3.5.

3. The dead load acting on the beam is self-weight (assumed):

$$w_D = \frac{12 \times 20}{144} \times 150 = 250 \text{ lb/ft} = 0.25 \text{ K/ft}$$

where 150 is the weight of reinforced concrete in pcf.

4. The external factored moment is

$$M_u = 1.2M_D + 1.6M_L$$

$$= 1.2\left(\frac{0.25}{8} \times 20^2\right) + 1.6\left(\frac{w_L}{8} \times 20^2\right) = 15.0 + 80w_L$$

where w_L = uniform service live load on the beam in K/ft.

5. Internal design moment equals the external factored moment:

$$156.5 = 15.0 + 80w_L \quad \text{and} \quad w_L = 1.77 \text{ K/ft}$$

The allowable uniform service live load on the beam is 1.77 K/ft.

Example 3.6: Minimum Steel Reinforcement

Check the design adequacy of the section shown in Fig. 3.19 to resist a factored moment $M_u = 30$ K·ft, using $f'_c = 3$ ksi and $f_y = 40$ ksi.

Solution

1. Check ρ provided in the section:

$$\rho = \frac{A_s}{bd} = \frac{3 \times 0.2}{10 \times 18} = 0.00333$$

2. Check $\rho_{\min}$ required according to the ACI Code:

$$\rho_{\min} = \frac{200}{f_y} = 0.005 > \rho = 0.00333$$

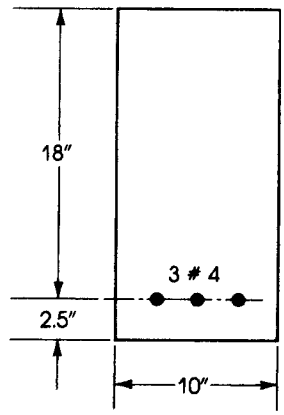


Figure 3.19 Example 3.6.

Therefore, use $\rho = \rho_{min} = 0.005$.

$$A_{s\,min} = \rho_{min} bd = 0.005 \times 10 \times 18 = 0.90 \text{ in.}^2$$

Use three no. 5 bars ($A_s = 0.91 \text{ in.}^2$), because three no. 4 bars are less than the minimum specified by the code.

3. Check moment strength: $\phi M_n = \phi A_s f_y (d - a/2)$.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.91 \times 40}{0.85 \times 3 \times 10} = 1.43 \text{ in.}$$

$$\phi M_n = 0.9 \times 0.91 \times 40 \left(18 - \frac{1.43}{2}\right) = 566 \text{ K·in.} = 47.2 \text{ K·ft}$$

4. An alternative solution, according to the ACI Code, Section 10.5, for three no. 4 bars, $A_s = 0.6 \text{ in.}^2$ is

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.6 \times 40}{0.85 \times 3 \times 10} = 0.94 \text{ in.}$$

$$\phi M_n = \frac{0.9}{12} \times 0.6 \times 40 \left(18 - \frac{0.94}{2}\right) = 31.55 \text{ K·ft}$$

$$A_s \text{ required for } 30 \text{ K·ft} = \frac{30}{31.55} \times 0.6 = 0.57 \text{ in.}^2$$

The minimum A_s required according to the ACI Code, Section 10.5, is at least one-third greater than 0.57 in.^2:

$$\text{Minimum } A_s \text{ required} = 1.33 \times 0.57 = 0.76 \text{ in.}^2$$

which exceeds the 0.6 in.^2 provided by the no. 4 bars. Use three no. 5 bars, because $A_s = 0.91 \text{ in.}^2$ is greater than the 0.76 in.^2 required.

3.12 BUNDLED BARS

When the design of a section requires the use of a large amount of steel—for example, when ρ_{max} is used—it may be difficult to fit all bars within the cross-section. The ACI Code, 7.6, allows the use of parallel bars placed in a bundled form of two, three, or four bars, as shown in Fig. 3.20. Up to four bars (no. 11 or smaller) can be bundled when they are enclosed by stirrups.

The same bundled bars can be used in columns, provided that they are enclosed by ties. All bundled bars may be treated as a single bar for checking the spacing and concrete cover requirements. The single bar diameter shall be derived from the equivalent total area of the bundled bars.

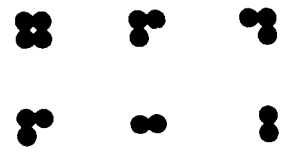


Figure 3.20 Bundled bar arrangement.

Summary: Singly Reinforced Rectangular Section

The procedure for determining the design moment of a singly reinforced rectangular section according to the ACI Code limitations can be summarized as follows:

1. Calculate the steel ratio in the section, $\rho = A_s/bd$.
2. Calculate the balanced and maximum steel ratios, Eqs. 3.18 and 3.31 or Table 3.2, for tension-controlled section. Also, calculate $\rho_{min} = 200/f_y$ when $f'_c < 4500$ psi (f'_c and f_y are in psi units) and $\rho_{min} = 3\sqrt{f'_c}/f_y$ when $f'_c \geq 4500$ psi.
3. If $\rho_{min} \leq \rho \leq \rho_{max}$, then the section meets the ACI Code limitations for tension-controlled section. If $\rho \leq \rho_{min}$, the section is not acceptable (unless a steel ratio $\rho \geq \rho_{min}$ is used). If $\rho \geq \rho_{max}$, $\phi = 0.9$.
4. Calculate $a = \dfrac{A_s f_y}{0.85 f'_c b}$, c, ε_t, and ϕ.
5. Calculate $\phi M_n = \phi A_s f_y \left(d - \dfrac{a}{2}\right)$.

Flow charts representing this section and other sections are given on www.wiley.com/college/hassoun.

3.13 SECTIONS IN THE TRANSITION REGION ($\phi < 0.9$)

In the case when the NTS, ε_t in the extreme tension steel lies between the compression-controlled strain limit (0.002 for $f_y = 60$ ksi) and the tension-controlled strain limit of 0.005, the strength reduction factor, ϕ, will be less than 0.9. Consequently, the design moment strength of the section ϕM_n will be smaller than ϕM_n with $\phi = 0.9$ (refer to Fig. 3.6). In the transition region, ε_t should not be less than 0.004 for flexural members (ACI Code, Section 10.3). (See Example 3.8.)

Example 3.7

Determine the design moment strength of a rectangular concrete section reinforced with four no. 9 bars in one row (Fig. 3.21).

Given: $b = 12$ in., $d = 16.5$ in., $h = 19$ in., $f'_c = 4$ ksi, and $f_y = 60$ ksi.

Solution

1. By the ACI Code provisions, for $f'_c = 4$ ksi, $f_y = 60$ ksi, and tension-controlled conditions ($\rho_b = 0.0285$ and $\rho_{max} = 0.01806$), check $\rho = A_s/bd = 4/(12 \times 6) = 0.02083 > \rho_{max}$. This indicates that the section is in the transition region and $\phi < 0.9$.

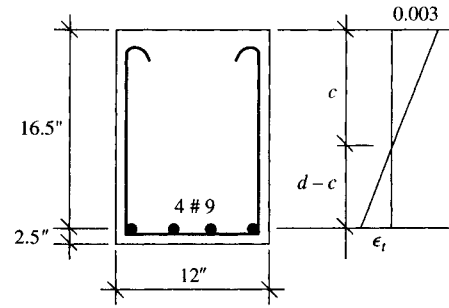


Figure 3.21 Example 3.7 ($d = d_t$).

2. Calculate a, c, and ε_t:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4 \times 60}{0.85 \times 4 \times 12} = 5.882 \text{ in.}$$

$$c = \frac{a}{0.85} = 6.92 \text{ in.} \quad d_t = d = 16.5 \text{ in.} \quad \frac{c}{d_t} = 0.42 > 0.375$$

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003 = \left(\frac{16.5 - 6.92}{6.92}\right) 0.003 = 0.004153 > 0.004$$

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) = 0.829$$

3. Calculate:

$$\phi M_n = \phi A_s f_y (d - a/2)$$

$$= 0.829(4)(60)\frac{\left(16.5 - \frac{5.882}{2}\right)}{12} = 224.9 \text{ K-ft}$$

Discussion

A slightly conservative approach can be used assuming tension-controlled section, $\rho = \rho_{\max} = 0.01806$ and $\phi = 0.9$. $A_{s\max} = 0.01806(12 \times 16.5) = 3.576 \text{ in.}^2$, $a = 5.259$ in., and $\phi M_n = 223.2$ K-ft (almost equal to the above ϕM_n).

Example 3.8: Two Rows of Bars

Determine the design moment strength of a rectangular concrete section reinforced with six no. 9 bars in two rows (Fig. 3.22).

Given: $b = 12$ in., $d = 23.5$ in., $h = 27$ in., $d_t = 24.5$ in., $f'_c = 4$ ksi, and $f_y = 60$ ksi.

Solution

1. For tension-controlled condition, $\varepsilon_t = 0.005$, $\rho_{\max} = 0.01806$ (Table 3.2) and $\rho_b = 0.0285$. Check

$$\rho = \frac{A_s}{bd} = \frac{6}{12 \times 23.5} = 0.02128 > \rho_{\max}$$

Section is in the transition region.

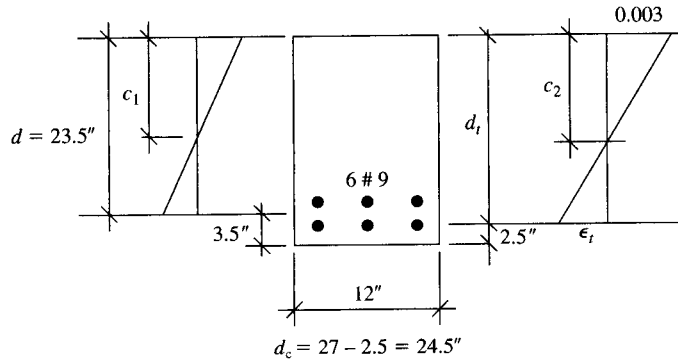


Figure 3.22 Example 3.8.

2. Calculate a, c, and ε_t:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{6 \times 60}{0.85 \times 4 \times 12} = 8.824 \text{ in.}$$

$$c = \frac{a}{0.85} = 10.38 \text{ in.} \quad d_t = h - 2.5 = 27 - 2.5 = 24.5$$

$$\frac{c}{d_t} = 0.424 > 0.375$$

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003 = \left(\frac{24.5 - 10.38}{10.38}\right) 0.003 = 0.00408 > 0.004$$

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) = 0.823$$

3. Calculate

$$\phi M_n = \phi A_s f_y (d - a/2)$$

$$= 0.823(6)(60)\frac{\left(23.5 - \frac{8.824}{2}\right)}{12} = 471 \text{ K·ft}$$

Discussion

For a tension-controlled section limitation, $\rho_{\max} = 0.01806$ and $R_u = 820\,\text{psi}$,

$$\phi M_n = R_u b d^2 = 0.82(12)\frac{(23.5)^2}{12} = 452.8 \text{ K·ft}$$

This is a conservative value: It is advisable to choose adequate reinforcement to produce tension-controlled condition with $\phi = 0.9$.

3.14 RECTANGULAR SECTIONS WITH COMPRESSION REINFORCEMENT

In concrete sections proportioned to resist the bending moments resulting from external loading on a structural member, the internal moment is equal to or greater than the external moment, but a concrete section of a given width and effective depth has a minimum capacity when $\rho_{\max}$ is used. If the external factored moment is greater than the design moment strength, more compressive and tensile reinforcement must be added.

Compression reinforcement is used when a section is limited to specific dimensions due to architectural reasons, such as a need for limited headroom in multistory buildings. Another advantage of compression reinforcement is that long-time deflection is reduced, as is explained in Chapter 6. A third use of bars in the compression zone is to hold stirrups, which are used to resist shear forces.

Two cases of doubly reinforced concrete sections will be considered, depending on whether compression steel yields or does not yield.

3.14.1 When Compression Steel Yields

Internal moment can be divided into two moments, as shown in Fig. 3.23. M_{u_1} is the moment produced by the concrete compressive force and an equivalent tension force in steel, A_{s_1}, acting as

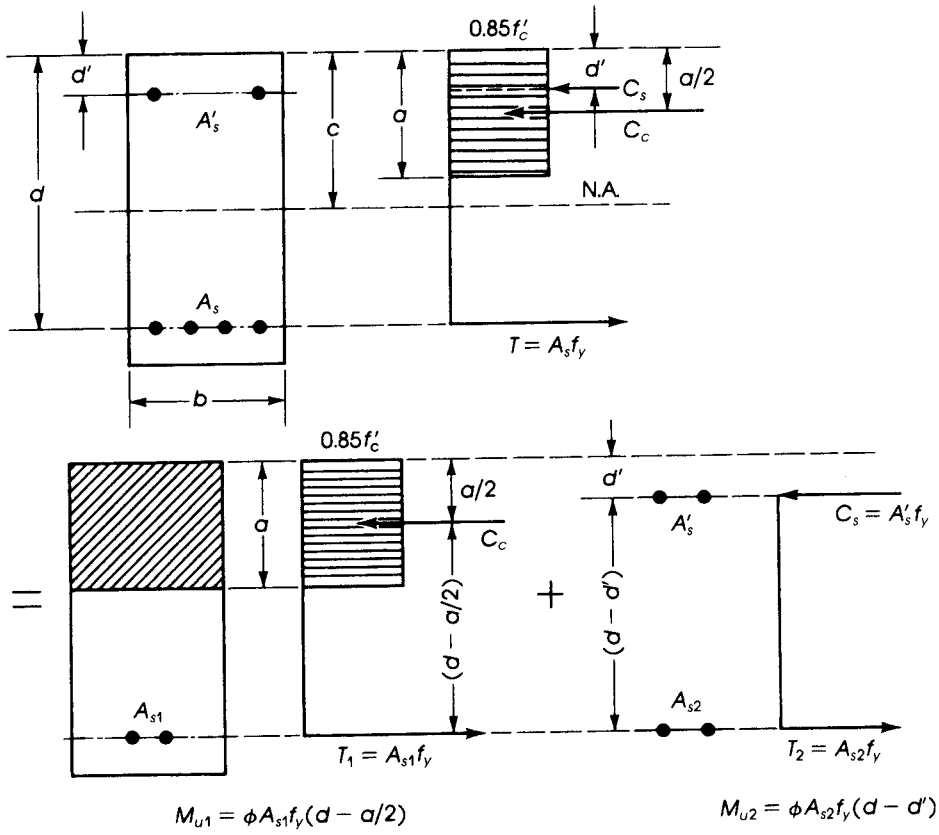


Figure 3.23 Rectangular section with compression reinforcement.

a basic section. M_{u_2} is the additional moment produced by the compressive force in compression steel A'_s and the tension force in the additional tensile steel, A_{s_2}, acting as a steel section.

The moment M_{u_1} is that of a singly reinforced concrete basic section,

$$T_1 = C_c \tag{3.36}$$

$$A_{s_1} f_y = C_c = 0.85 f'_c ab \tag{3.37}$$

$$a = \frac{A_{s_1} f_y}{0.85 f'_c b} \tag{3.38}$$

$$M_{u_1} = \phi A_{s_1} f_y \left(d - \frac{a}{2}\right) \tag{3.39}$$

The restriction for M_{u_1} is that $\rho_1 < A_{s_1}/bd$ shall be equal to or less than $\rho_{\max}$ for singly reinforced tension-controlled sections, as given in Eq. 3.31*a*.

Considering the moment M_{u_2} and assuming that the compression steel designated as A'_s yields,

$$M_{u_2} = \phi A_{s_2} f_y (d - d') \tag{3.40a}$$

$$M_{u_2} = \phi A'_s f_y (d - d') \tag{3.40b}$$

In this case $A_{s_2} = A'_s$, producing equal and opposite forces, as shown in Fig. 3.23. The total resisting moment, M_u, is then the sum of the two moments M_{u_1} and M_{u_2}:

$$\phi M_n = M_{u_1} + M_{u_2} = \phi\left[A_{s_1} f_y \left(d - \frac{a}{2}\right) + A'_s f_y (d - d')\right] \tag{3.41}$$

The total steel reinforcement used in tension is the sum of the two steel amounts A_{s_1} and A_{s_2}. Therefore,

$$A_s = A_{s_1} + A_{s_2} = A_{s_1} + A'_s \tag{3.42}$$

and

$$A_{s_1} = A_s - A'_s$$

Then, substituting $(A_s - A'_s)$ for A_{s_1} in Eqs. 3.38, 3.39, and 3.41,

$$a = \frac{(A_s - A'_s) f_y}{0.85 f'_c b} \tag{3.43}$$

$$\phi M_n = \phi\left[(A_s - A'_s) f_y \left(d - \frac{a}{2}\right) + A'_s f_y (d - d')\right] \tag{3.44}$$

and

$$(\rho - \rho') \le \rho_{\max} = \rho_b \left(\frac{0.003 + f_y/E_s}{0.008}\right) \tag{3.45}$$

For $f_y = 60$ ksi, $(\rho - \rho') \le 0.63375\ \rho_b$, $\phi = 0.9$, and $\varepsilon_t = 0.005$. Equation 3.45 must be fulfilled in doubly reinforced concrete sections, which indicates that the difference between total tension steel and the compression steel should not exceed the maximum steel for singly reinforced concrete tension-controlled sections. Failure due to yielding of the total tensile steel will then be expected, and sudden failure of concrete is avoided.

If $\rho_1 = (\rho - \rho') > \rho_{\max}$, the section will be in the transition region with a limit of $(\rho - \rho') \le \rho_{\max t}$ (Eq. 3.34*a*). In this case, $\phi < 0.9$ for M_{u_1} and $\phi = 0.9$ for M_{u_2}. Equation 3.44 becomes

$$\phi M_n = \phi\left[(A_s - A'_s) f_y \left(d - \frac{a}{2}\right)\right] + 0.9 A'_s f_y (d - d') \tag{3.44a}$$

Note that $(A_s - A'_s) \le \rho_{\max t}(bd)$.

In the compression zone, the force in the compression steel is $C_s = A'_s(f_y - 0.85 f'_c)$, taking into account the area of concrete displaced by A'_s. In this case,

$$T = A_s f_y = C_c + C_s = 0.85 f'_c ab + A'_s(f_y - 0.85 f'_c)$$

$$A_s f_y - A'_s f_y + 0.85 f'_c A'_s = 0.85 f'_c ab = C_c = A_{s_1} f_y \text{ (for the basic section)}$$

Dividing by bdf_y,

$$\rho - \rho'\left(1 - 0.85\frac{f'_c}{f_y}\right) = \rho_1, \qquad \text{where } \rho_1 = \frac{A_{s_1}}{bd} \le \rho_{\max}$$

Therefore,

$$\rho - \rho'\left(1 - 0.85\frac{f'_c}{f_y}\right) \le \rho_{\max} = \rho_b \left(\frac{0.003 + f_y/E_s}{0.008}\right) \tag{3.46}$$

Although Eq. 3.46 is more accurate than Eq. 3.45, it is quite practical to use both equations to check the condition for maximum steel ratio in rectangular sections when compression steel yields.

For example, if $f'_c = 3$ ksi and $f_y = 60$ ksi, Eq. 3.46 becomes $(\rho - 0.9575\rho') \leq 0.016$; if $f'_c = 4$ ksi and $f_y = 60$ ksi, then $(\rho - 0.9575\rho') \leq 0.02138$.

The maximum total tensile steel ratio, ρ, that can be used in a rectangular section when compression steel yields is as follows:

$$\text{Max } \rho = (\rho_{\max} + \rho') \tag{3.47}$$

where $\rho_{\max}$ is maximum tensile steel ratio for the basic singly reinforced tension-controlled concrete section. This means that maximum total tensile steel area that can be used in a rectangular section when compression steel yield is as follows:

$$\text{Max } A_s = bd(\rho_{\max} + \rho') \tag{3.47a}$$

In the preceding equations, it is assumed that compression steel yields. To investigate this condition, refer to the strain diagram in Fig. 3.24. If compression steel yields, then

$$\varepsilon'_s \geq \varepsilon_y = \frac{f_y}{E_s}$$

From the two triangles above the neutral axis, substitute $E_s = 29{,}000$ ksi and let f_y be in ksi. Then

$$\frac{c}{d'} = \frac{0.003}{0.003 + \frac{f_y}{E_s}} = \frac{87}{87 - f_y}$$

$$c = \left(\frac{87}{87 - f_y}\right) d' \tag{3.48}$$

From Eq. 3.37,

$$A_{s_1} f_y = 0.85 f'_c ab \tag{3.47}$$

but

$$A_{s_1} = A_s - A'_s \quad \text{and} \quad \rho_1 = (\rho - \rho')$$

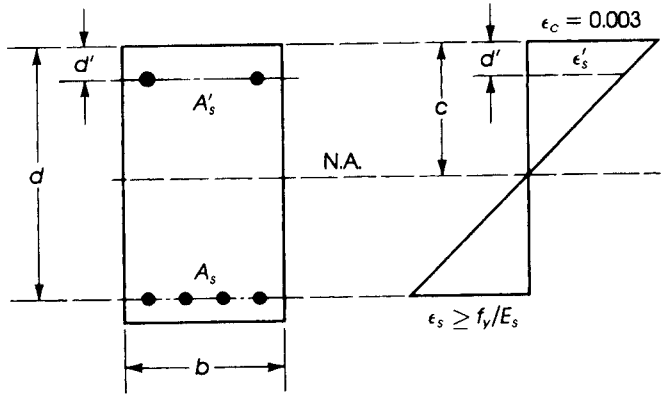


Figure 3.24 Strain diagram in doubly reinforced section.

Therefore, Eq. 3.37 becomes $(A_s - A_s')f_y = 0.85f_c'ab$:

$$(\rho - \rho')bdf_y = 0.85f_c'ab$$

$$(\rho - \rho') = 0.85\left(\frac{f_c'}{f_y}\right)\left(\frac{a}{d}\right)$$

Also,

$$a = \beta_1 c = \beta_1\left(\frac{87}{87 - f_y}\right)d'$$

Therefore,

$$(\rho - \rho') = 0.85\beta_1\left(\frac{f_c'}{f_y}\right)\left(\frac{d'}{d}\right)\left(\frac{87}{87 - f_y}\right) = K \tag{3.49}$$

The quantity $(\rho - \rho')$ is the steel ratio, or $(A_s - A_s')/bd = A_{s_1}/bd = \rho_1$ for the singly reinforced basic section.

If $(\rho - \rho')$ is greater than the value of the right-hand side in Eq. 3.49, then compression steel will also yield. In Fig. 3.25 we can see that if A_{s_1} is increased, T_1 and, consequently, C_1

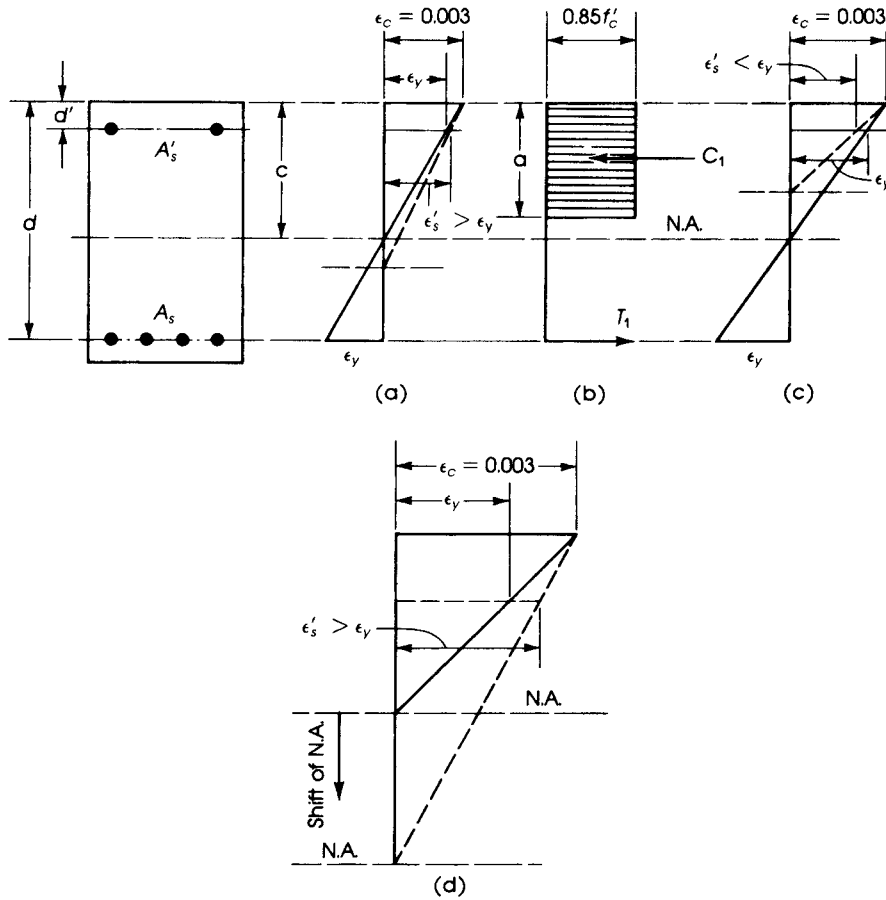


Figure 3.25 Yielding and nonyielding cases of compression reinforcement. Diagram (*d*), a close-up of (*a*), shows how the neutral axis responds to an increase in A_{s1}.

Table 3.4 Values of K for Different f'_c and f_y

f'_c (ksi)	f_y (ksi)	K	K (for $d' = 2.5$ in.)
3	40	$0.1003d'/d$	$0.251/d$
3	60	$0.1164d'/d$	$0.291/d$
4	60	$0.1552d'/d$	$0.388/d$
5	60	$0.1826d'/d$	$0.456/d$

will be greater and the neutral axis will shift downward, increasing the strain in the compression steel and ensuring its yield condition. If the tension steel used (A_{s_1}) is less than the right-hand side of Eq. 3.49, then T_1 and C_1 will consequently be smaller, and the strain in compression steel, ε'_s, will be less than or equal to ε_y, because the neutral axis will shift upward, as shown in Fig. 3.25*c*, and compression steel will not yield.

Therefore, Eq. 3.49 can be written

$$(\rho - \rho') \geq 0.85\beta_1 \frac{f'_c}{f_y} \times \frac{d'}{d} \times \frac{87}{87 - f_y} = K \tag{3.49a}$$

where f_y is in ksi, and this is the condition for compression steel to yield.

For example, the values of K for different values of f'_c and f_y are as shown in Table 3.4.

Example 3.9

A rectangular beam has a width of 12 in. and an effective depth of $d = 22.5$ in. to the centroid of tension steel bars. Tension reinforcement consists of six no. 9 bars in two rows; compression reinforcement consists of two no. 7 bars placed as shown in Fig. 3.26. Calculate the design moment strength of the beam if $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. Check if compression steel yields:

$$A_s = 6.0 \text{ in.}^2 \quad \rho = \frac{A_s}{bd} = \frac{6.0}{12 \times 22.5} = 0.02222$$

$$A'_s = 1.2 \text{ in.}^2 \quad \rho' = \frac{A'_s}{bd} = \frac{1.2}{12 \times 22.5} = 0.00444$$

$$A_s - A'_s = 4.8 \text{ in.}^2 \quad \rho - \rho' = 0.01778$$

For compression steel to yield,

$$(\rho - \rho') \geq 0.85\beta_1 \frac{f'_c}{f_y} \times \frac{d'}{d} \times \frac{87}{87 - f_y} = K$$

β_1 is 0.85 because $f'_c = 4000$ psi; therefore,

$$K = (0.85)^2 \left(\frac{4}{60}\right)\left(\frac{2.5}{22.5}\right)\left(\frac{87}{87 - 60}\right) = 0.0175$$

$$(\rho - \rho') = 0.01778 > 0.0175$$

Therefore, compression steel yields.

2. Check that $(\rho - \rho') \leq \rho_{max}$ (Eq. 3.45): For $f'_c = 4$ ksi and $f_y = 60$ ksi, $\rho_b = 0.0285$ and $\rho_{max} = 0.01806$ (Table 3.2). $(\rho - \rho') \leq 0.01778 < \rho_{max}$, and $\phi = 0.9$ (a tension-controlled condition).

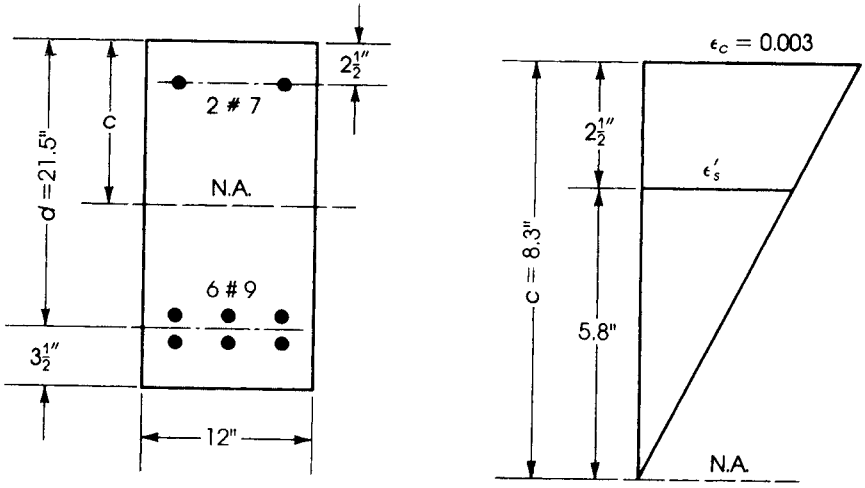


Figure 3.26 Example 3.9.

3. ϕM_n can be calculated by Eq. 3.44:

$$\phi M_n = \phi\left[(A_s - A'_s)f_y\left(d - \frac{a}{2}\right) + A'_s f_y(d - d')\right]$$

$$a = \frac{(A_s - A'_s)f_y}{0.85 f'_c b} = \frac{4.8 \times 60}{0.85 \times 4 \times 12} = 7.06 \text{ in.}$$

$$\phi M_n = (0.9)\left[4.8 \times 60\left(22.5 - \frac{7.06}{2}\right) + 1.2 \times 60(22.5 - 2.5)\right]$$

$$= 6213 \text{ K·in.} = 517.8 \text{ K·ft}$$

4. An alternative approach for checking if compression steel yields can be made as follows:

$$c = \frac{a}{0.85} = \frac{7.06}{0.85} = 8.3 \text{ in.}$$

$$\varepsilon'_s = \frac{5.8}{8.3} \times 0.003 = 0.0021 \qquad \varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.00207$$

Because ε'_s, exceeds ε_y, compression steel yields.

5. Check ε_t:$c = 8.3$ in., $d_t = 26 - 2.5 = 23.5$ in.

$$\varepsilon_t = \left(\frac{23.5 - 8.3}{8.3}\right)0.003 = 0.0055 > 0.005$$

$$\text{or } \frac{c}{d} = 0.353 < 0.375 \qquad \text{(o.k.)}$$

6. The maximum total tension steel for this section, max A_s, is equal to

$$\text{Max } A_s = bd(\rho_{\max} + \rho') = 12 \times 22.5(0.01806 + 0.00444)$$

$$= 6.08 \text{ in.}^2 > A_s = 6.0 \text{ in.}^2 \qquad \text{(used in the section)}$$

3.14.2 When Compression Steel Does Not Yield

As was explained earlier, if

$$(\rho - \rho') < \left(0.85\beta_1 \times \frac{f'_c}{f_y} \times \frac{d'}{d} \times \frac{87}{87 - f_y}\right) = K \tag{3.50}$$

then compression steel does not yield. This indicates that if $(\rho - \rho') < K$, the tension steel will yield before concrete can reach its maximum strain of 0.003, and the strain in compression steel, ε'_s, will not reach ε_y at failure (Fig. 3.25). Yielding of compression steel will also depend on its position relative to the extreme compressive fibers d'. A higher ratio of d'/c will decrease the strain in the compressive steel, ε'_s, as it places compression steel A'_s nearer to the neutral axis.

If compression steel does not yield, a general solution can be performed by analysis based on statics. Also, a solution can be made as follows: Referring to Figs. 3.23 and 3.24,

$$\varepsilon'_s = 0.003\left(\frac{c - d'}{c}\right) \quad f'_s = E_s\varepsilon'_s = 29{,}000(0.003)\left(\frac{c - d'}{c}\right) = 87\left(\frac{c - d'}{c}\right)$$

Let $C_c = 0.85f'_c\beta_1 cb$:

$$C_s = A'_s(f'_s - 0.85f'_c) = A'_s\left[87\frac{(c - d')}{c} 0.85f'_c\right]$$

Because $T = A_s f_y = C_c + C_s$, then

$$A_s f_y = (0.85f'_c\beta_1 cb) + A'_s\left[87\left(\frac{c - d'}{c}\right) - 0.85f'_c\right]$$

Rearranging terms yields

$$(0.85f'_c\beta_1 b)c^2 + [(87A'_s) - (0.85f'_c A'_s) - A_s f_y]c - 87A'_s d' = 0$$

This is similar to $A_1c^2 + A_2c + A_3 = 0$, where

$$A_1 = 0.85f'_c\beta_1 b$$
$$A_2 = A'_s(87 - 0.85f'_c) - A_s f_y$$
$$A_3 = -87A'_s d'$$

Solve for c:

$$c = \frac{1}{2A_1}\left[-A_2 \pm \sqrt{A_2^2 - 4A_1A_3}\right] \tag{3.51}$$

Once c is determined, then calculate f'_s, a, C_c and C_s.
$f'_s = 87[(c - d')/c]$; $a = \beta_1 c$; $C_c = 0.85f'_c ab$; and $C_s = A'_s(f'_s - 0.85f'_c)$.

$$\phi M_n = \phi\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right] \tag{3.52}$$

When compression steel does not yield, $f'_s < f_y$, and the maximum total tensile steel reinforcement needed for a rectangular section is

$$\text{Max } A_s = \rho_{\max} bd + A'_s\frac{f'_s}{f_y} = bd\left(\rho_{\max} + \frac{\rho' f'_s}{f_y}\right) \tag{3.53}$$

Using steel ratios and dividing by bd:

$$\text{Max } \rho = \frac{\max A_s}{bd} \leq \rho_{\max} + \rho' \frac{f'_s}{f_y} \tag{3.54}$$

or

$$\left(\rho - \rho' \frac{f'_s}{f_y}\right) \leq \rho_{\max} \tag{3.55}$$

where $\rho_{\max}$ is the maximum steel ratio for the tension-controlled singly reinforced rectangular section (Eq. 3.31).

In this case,

$$a = \frac{A_s f_y - A'_s f'_s}{0.85 f'_c b} \tag{3.56}$$

$$\phi M_n = \phi \left[(A_s f_y - A'_s f'_s) \left(d - \frac{a}{2}\right) + A'_s f'_s (d - d') \right] \tag{3.57}$$

In summary, the procedure for analyzing sections with compression steel is as follows:

1. Calculate ρ, ρ', and $(\rho - \rho')$. Also calculate $\rho_{\max}$ and $\rho_{\min}$.
2. Calculate

$$K = 0.85 \beta_1 \left(\frac{f'_c}{f_y}\right) \left(\frac{87}{87 - f_y}\right) \left(\frac{d'}{d}\right)$$

   Use ksi units.
3. If $(\rho - \rho') \geq K$, then compression steel yields, and $f'_s = f_y$; if $(\rho - \rho') < K$, then compression steel does not yield, and $f'_s < f_y$.
4. If compression steel yields, then
   a. Check that $\rho_{\max} \geq (\rho - \rho') \geq \rho_{\min}$ (to use $\phi = 0.9$) or check $\varepsilon_t \geq 0.005$.
   b. Calculate

$$a = \frac{(A_s - A'_s) f_y}{0.85 f'_c b}.$$

   c. Calculate

$$\phi M_n = \phi \left[(A_s - A'_s) f_y \left(d - \frac{a}{2}\right) + A'_s f_y (d - d') \right].$$

   d. The maximum A_s that can be used in the section is Max $A_s = bd(\rho_{\max} + \rho') \geq A_s$ (used).
5. If compression steel does *not* yield, then
   a. Calculate the distance to the neutral axis c by using analysis (Example 3.10) or by using the quadratic equation (3.51).
   b. Calculate

$$f'_s = 87 \left(\frac{c - d'}{c}\right) \text{(ksi)}.$$

c. Check that $(\rho - \rho' f'_s/f_y) \leq \rho_{max}$ or max A_s that can be used in the section is greater than or equal to the A_s used.

$$\text{Max } A_s = bd\left(\rho_{max} + \frac{\rho' f'_s}{f_y}\right) \geq A_s \quad \text{(used)}$$

d. Calculate

$$a = \frac{(A_s f_y - A'_s f'_s)}{0.85 f'_c b} \text{ or } a = \beta_1 c.$$

e. Calculate

$$\phi M_n = \phi\left[(A_s f_y - A'_s f'_s)\left(d - \frac{a}{2}\right) + A'_s f'_s (d - d')\right].$$

For flow charts, refer to Chapter 21.

Example 3.10

Determine the design moment strength of the section shown in Fig. 3.27 using $f'_c = 5$ ksi, $f_y = 60$ ksi, $A'_s = 2.37$ in.^{2} (three no. 8 bars), and $A_s = 7.62$ in.^{2} (six no. 10 bars).

Solution

1. Calculate ρ and ρ':

$$\rho = \frac{A_s}{bd} = \frac{7.62}{14 \times 22.5} = 0.0242 \quad \rho' = \frac{A'_s}{bd} = \frac{2.37}{14 \times 22.5} = 0.00753$$

$$(\rho - \rho') = 0.01667$$

2. Apply Eq. 3.50, assuming $\beta_1 = 0.8$ for $f'_c = 5000$ psi.

$$K = 0.85\beta_1 \times \frac{f'_c}{f_y} \times \frac{d'}{d} \times \frac{87}{87 - f_y} = 0.85 \times 0.8\left(\frac{5}{60}\right)\left(\frac{2.5}{22.5}\right)\left(\frac{87}{87 - 60}\right) = 0.0203$$

(or from Table 3.3, $K = 0.456/d = 0.0203$).

$$(\rho - \rho') = 0.01667 < 0.0203$$

Therefore, compression steel does not yield, and $f'_s < 60$ ksi.

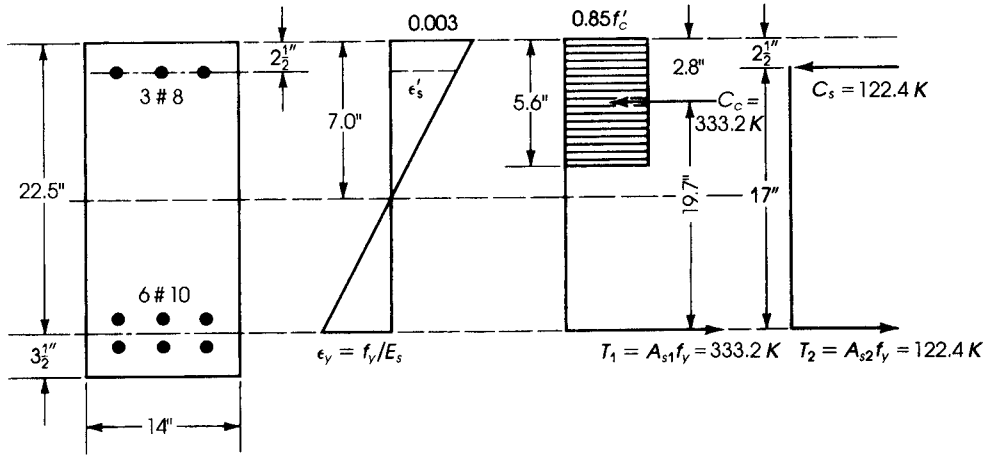


Figure 3.27 Example 3.10 analysis solution.

For $f'_c = 5$ ksi and $f_y = 60$ ksi, $\rho_b = 0.0335$ and $\rho_{max} = 0.02123$ [Table 3.2 $(\rho - \rho') < \rho_{max}$, for the basic section]. $\phi = 0.9$, so this is a tension-controlled condition.

3. Calculate ϕM_n by analysis. Internal forces:

$$C_c = 0.85 f'_c ab \qquad a = \beta_1 c = 0.8c$$

$$C_c = 0.85 \times 5(0.8c) \times 14 = 47.7c$$

C_s = the force in compression steel

= $A'_s f'_s$ – the force in displaced concrete

= $A'_s(f'_s - 0.85 f'_c)$

From strain triangles,

$$\varepsilon'_s = 0.003\frac{c - d'}{c}$$

$f'_s = E_s \varepsilon'_s$ (since steel is in the elastic range)

$$= 29{,}000\left[\frac{0.003(c - d')}{c}\right] = \frac{87(c - d')}{c} \text{ (ksi)}$$

Therefore,

$$C_s = 2.37[87\frac{(c - d')}{c} - (0.85 \times 5)](\text{kips}) = \left[\frac{206.2(c - 2.5)}{c}\right] - 10.07$$

$$T = T_1 + T_2 = (A_{s1} + A_{s2}) f_y = S_s f_y = 7.62(60) = 457.2 \text{ kips}$$

4. Equate internal forces to determine the position of the neutral axis (the distance c):

$$T = C = C_c + C_s$$

$$457.2 = 47.6c + \frac{206.2(c - 2.5)}{c} - 10.07$$

$$c^2 - 5.48c - 10.83 = 0$$

$$c = 7.0 \text{ in.} \qquad a = 0.8c = 5.6 \text{ in.}$$

Equation 3.51 can also be used to calculate c and a.

5. Calculate f'_s, C_c, and C_s:

$$f'_s = \frac{87(c - 2.5)}{c} = \frac{87(7.0 - 2.5)}{7.0} = 55.9 \text{ ksi}$$

which confirms that compression steel does not yield.

$$C_c = 47.6c = 47.6(7.0) = 333.2 \text{ kips}$$

$$C_s = (A'_s f'_s - 10.07) = 2.37(55.90) - 10.07 = 122.40 \text{ kips}$$

6. To calculate ϕM_n, take moments about the tension steel A_s:

$$\phi M_n = \phi\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right] = 0.9[333.2(22.5 - 2.8) + 122.40(22.5 - 2.5)]$$

$$= 8110.8 \text{ K·in.} = 675.9 \text{ K·ft}$$

7. Check that $(\rho - \rho' f_s'/f_y) \leq \rho_{max}$ (Eq. 3.55):

$$0.0242 - 0.00754\left(\frac{55.9}{60}\right) = 0.0171 < \rho_{max} = 0.02123$$

The maximum total tension steel that can be used in this section is calculated by Eq. 3.50

$$\max A_s = bd\left(\rho_{max} + \frac{\rho' f_s'}{f_y}\right)$$

$$= 14(22.5)\left(0.02123 + \frac{0.00753 \times 55.9}{60}\right) = 8.9 \text{ in.}^2 > 7.62 \text{ in.}^2 \qquad \text{(o.k.)}$$

8. ε_t can be checked as follows: $c = 7.0$ in., $d_t = 23.5$ in.

$$\frac{c}{d_t} = 0.3 < 0.375 \quad \text{or}$$

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right)0.003 = \left(\frac{23.5 - 7}{7}\right)0.003 = 0.0071 > 0.005$$

Tension-controlled section.

3.15 ANALYSIS OF T- AND I-SECTIONS

3.15.1 Description

It is normal to cast concrete slabs and beams together, producing a monolithic structure. Slabs have smaller thicknesses than beams. Under bending stresses, those parts of the slab on either side of the beam will be subjected to compressive stresses, depending on the position of these parts relative to the top fibers and relative to their distances from the beam. The part of the slab acting with the beam is called the flange, and it is indicated in Fig. 3.28*a* by area bt. The rest of the section confining the area $(h - t)b_w$ is called the *stem*, or *web*.

In an I-section there are two flanges, a compression flange, which is actually effective, and a tension flange, which is ineffective, because it lies below the neutral axis and is thus neglected completely. Therefore, the analysis and design of an I-beam is similar to that of a T-beam.

3.15.2 Effective Width

In a T-section, if the flange is very wide, the compressive stresses are at a maximum value at points adjacent to the beam and decrease approximately in a parabolic form to almost 0 at a distance x from the face of the beam. Stresses also vary vertically from a maximum at the top fibers of the flange to a minimum at the lower fibers of the flange. This variation depends on the position of the neutral axis and the change from elastic to inelastic deformation of the flange along its vertical axis.

An equivalent stress area can be assumed to represent the stress distribution on the width b of the flange, producing an equivalent flange width, b_e, of uniform stress (Fig. 3.28*c*).

Analysis of equivalent flange widths for actual T-beams indicate that b_e is a function of span length of the beam [7]. Other variables that affect the effective width b_e are (Fig. 3.29).

- Spacing of beams
- Width of stem (web) of beam b_w
- Relative thickness of slab with respect to the total beam depth

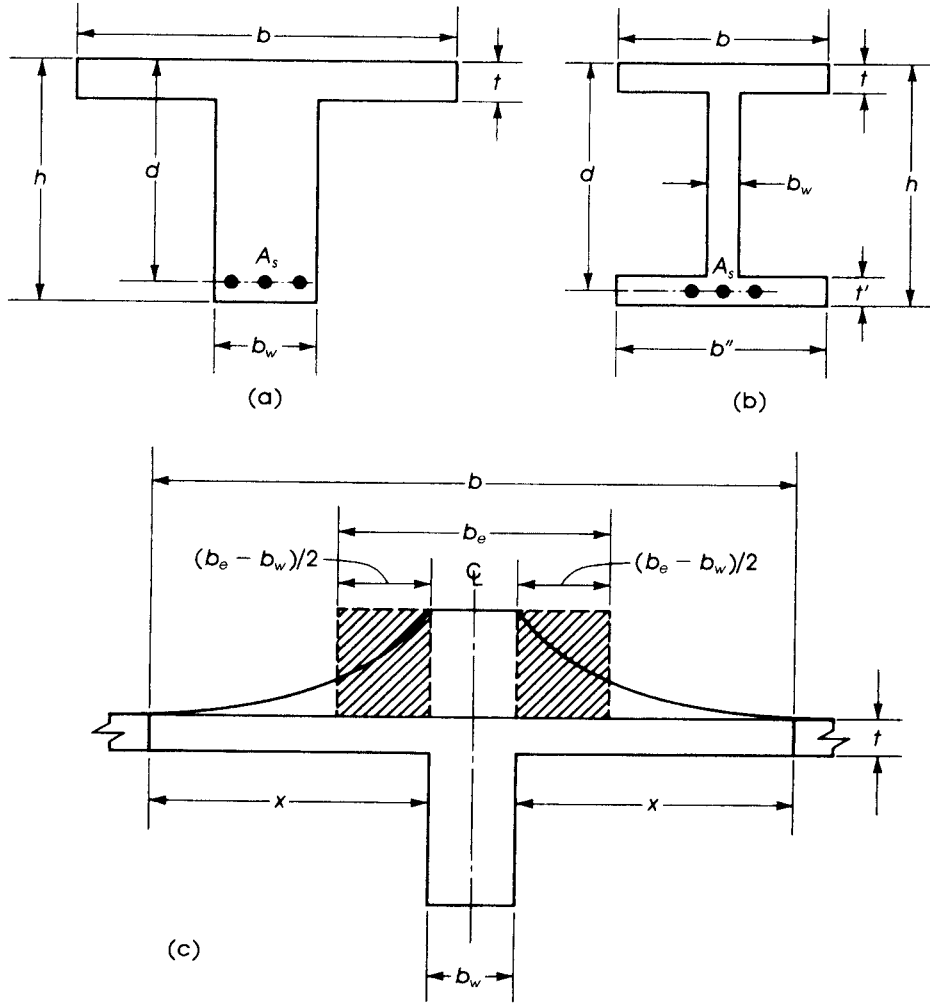


Figure 3.28 (*a*) T-section and (*b*) I-section, with (*c*) illustration of effective flange width b_e.

- End conditions of the beam (simply supported or continuous)
- The way in which the load is applied (distributed load or point load)
- The ratio of the length of the beam between points of zero moment to the width of the web and the distance between webs

The ACI Code, Section 8.10.2, prescribes the following limitations on the effective flange width b_e, considering that the span of the beam is equal to L:

1. $b_e = L/4$
2. $b_e = 16t + b_w$
3. $b_e = b$, where b is the distance between centerlines of adjacent slabs

The *smallest* of the aforementioned three values must be used.

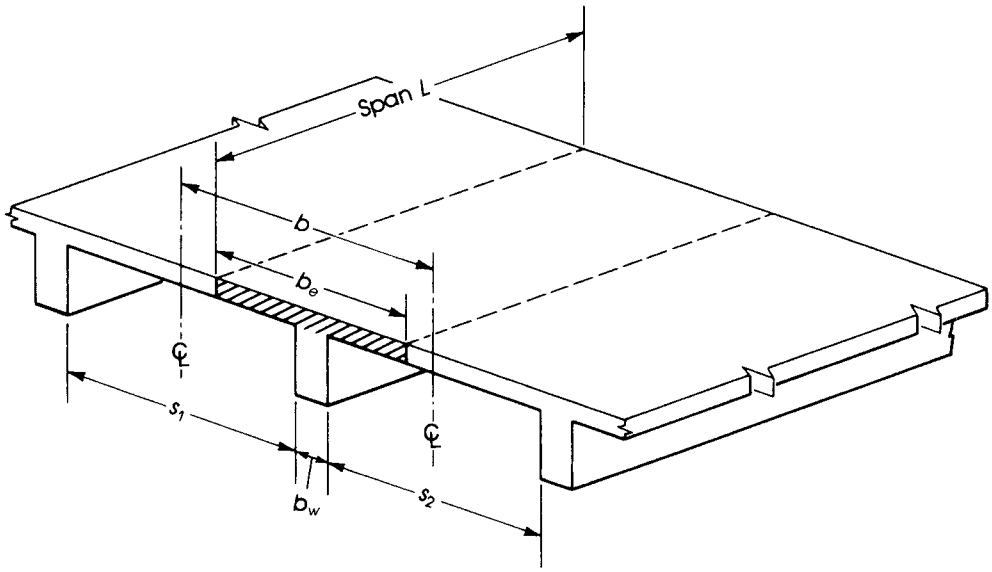


Figure 3.29 Effective flange width of T-beams.

These values are conservative for some cases of loading and are adequate for other cases. A similar effective width of flange can be adopted for I-beam sections. Investigations indicate that the effective compression flange increases as load is increased toward the ultimate value [6]. Under working loads, stress in the flange is within the elastic range.

A T-shaped or I-shaped section may behave as a rectangular section or a T-section. The two cases are investigated as follows.

3.15.3 T-Sections Behaving as Rectangular Sections

In this case, the depth of the equivalent stress block a lies within the flange, with extreme position at the level of the bottom fibers of the compression flange ($a \leq t$). When the neutral axis lies within the flange (Fig. 3.30*a*), the depth of the equivalent compressive distribution stress lies within the flange, producing a compressed area equal to $b_e a$. The concrete below the neutral axis is assumed ineffective, and the section is considered singly reinforced, as explained earlier, with b_e replaced by b. Therefore,

$$a = \frac{A_s f_y}{0.85 f'_c b_e} \tag{3.58}$$

and

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) \tag{3.59}$$

If the depth a is increased such that $a = t$, then the factored moment capacity is that of a singly reinforced concrete section:

$$\phi M_n = \phi A_s f_y \left(d - \frac{t}{2}\right) \tag{3.60}$$

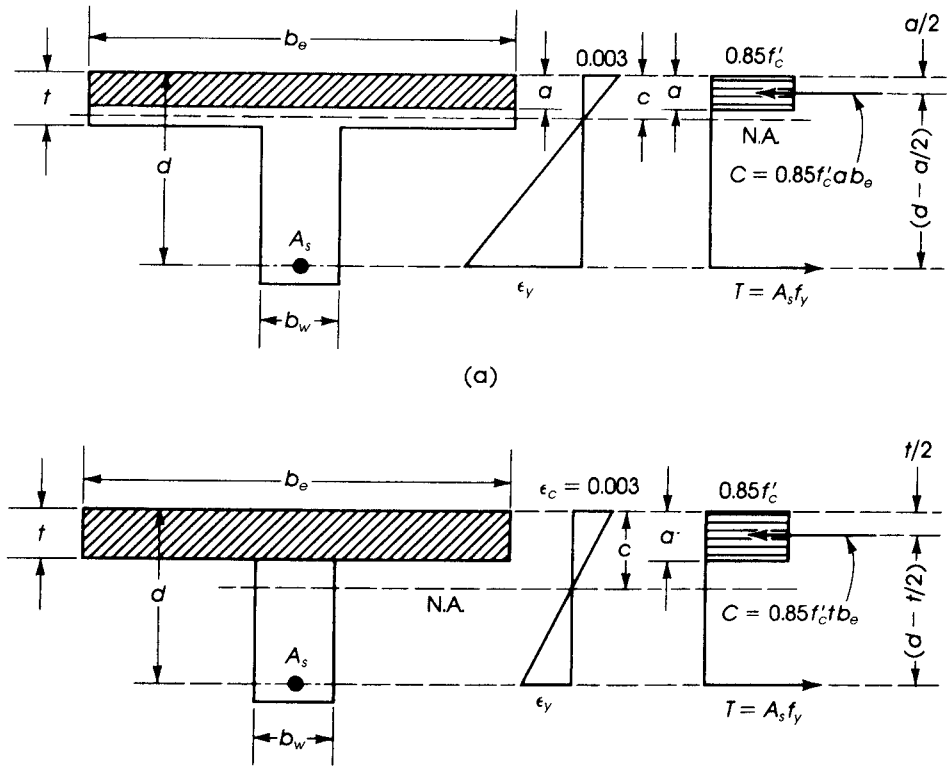


Figure 3.30 Rectangular section behavior (*a*) when the neutral axis lies within the flange and (*b*) when the stress distribution depth equals the slab thickness.

In this case

$$t = \frac{A_s f_y}{0.85 f'_c b_e} \text{ or } A_s = \frac{0.85 f'_c b_e t}{f_y} \tag{3.61}$$

In this analysis, the limit of the steel area in the section should apply: $A_s \leq A_{s\max}$, and $\varepsilon_t \geq 0.005$.

3.15.4 Analysis of a T-Section

In this case the depth of the equivalent compressive distribution stress lies below the flange; consequently, the neutral axis also lies in the web. This is due to an amount of tension steel A_s more than that calculated by Eq. 3.61. Part of the concrete in the web will now be effective in resisting the external moment. In Fig. 3.31, the compressive force C is equal to the compression area of the flange and web multiplied by the uniform stress of 0.85 f'_c:

$$C = 0.85 f'_c [b_e t + b_w (a - t)]$$

The position of C is at the centroid of the T-shaped compressive area at a distance z from top fibers.

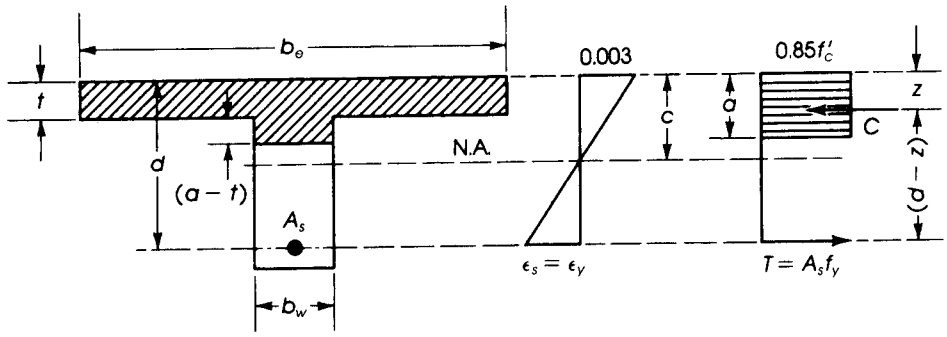


Figure 3.31 T-section behavior.

The analysis of a T-section is similar to that of a doubly reinforced concrete section, considering an area of concrete $(b_e - b_w)t$ as equivalent to the compression steel area A'_s. The analysis is divided into two parts, as shown in Fig. 3.32:

1. A singly reinforced rectangular basic section, $b_w d$, and steel reinforcement A_{s1}. The compressive force, C_1, is equal to $(0.85\ f'_c a b_w)$, the tensile force, T_1, is equal to $A_{s1} f_y$, and the moment arm is equal to $(d - a/2)$.
2. A section that consists of the concrete overhanging flange sides $2 \times [(b_e - b_w)t]/2$ developing the additional compressive force (when multiplied by $0.85\ f'_c$) and a moment arm equal to $(d - t/2)$. If A_{sf} is the area of tension steel that will develop a force equal to the

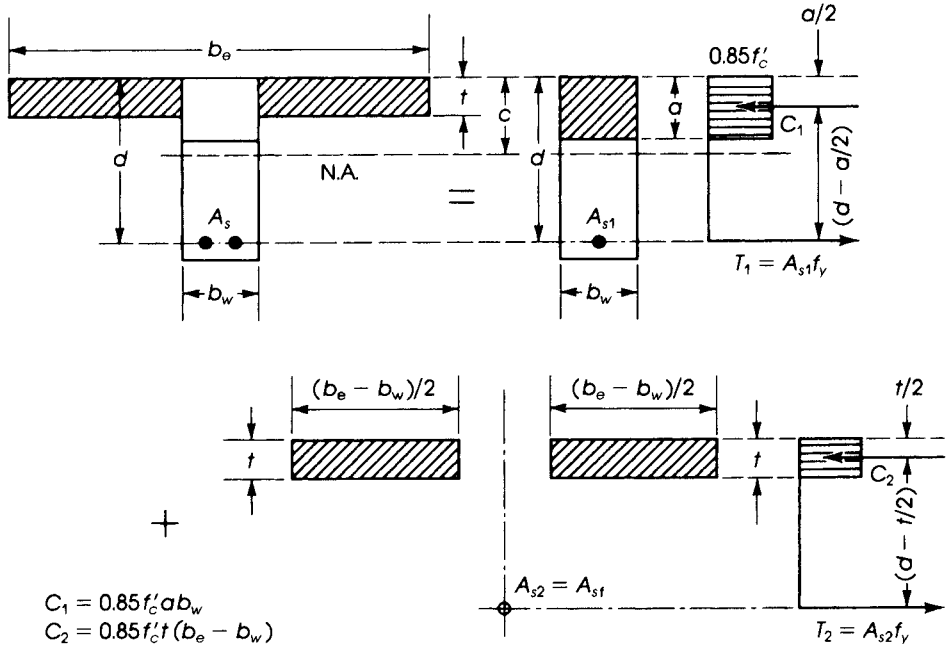


Figure 3.32 T-section analysis.

compressive strength of the overhanging flanges, then

$$A_{sf} f_y = 0.85 f'_c (b_e - b_w) t$$

$$A_{sf} = \frac{0.85 f'_c t (b_e - b_w)}{f_y} \tag{3.62}$$

The total steel used in the T-section A_s is equal to $A_{s1} + A_{sf}$, or

$$A_{s1} = A_s - A_{sf} \tag{3.63}$$

The T-section is in equilibrium, so $C_1 = T_1$, $C_2 = T_2$, and $C_1 = C_1 + C_2 = T_1 + T_2 = T$. Considering equation $C_1 = T_1$ for the basic section, then $A_{s1} f_y = 0.85\ f'_c a b_w$ or $(A_s - A_{sf}) f_y = 0.85\ f'_c a b_w$; therefore,

$$a = \frac{(A_s - A_{sf}) f_y}{0.85 f'_c b_w} \tag{3.64}$$

Note that b_w is used to calculate a. The factored moment capacity of the section is the sum of the two moments M_{u1} and M_{u2}.

$$\phi M_n = M_{u1} + M_{u2}$$

$$M_{u1} = \phi A_{s1} f_y \left(d - \frac{a}{2}\right) = \phi (A_s - A_{sf}) f_y \left(d - \frac{a}{2}\right)$$

where

$$A_{s1} = (A_s - A_{sf}) \quad \text{and} \quad a = \frac{(A_s - A_{sf}) f_y}{0.85\ f'_c b_w}$$

$$M_{u2} = \phi A_{sf} f_y \left(d - \frac{t}{2}\right) \tag{3.65}$$

$$\phi M_n = \phi \left[(A_s - A_{sf}) f_y \left(d - \frac{a}{2}\right) + A_{sf} f_y \left(d - \frac{t}{2}\right) \right]$$

Considering the web section $b_w d$, the net tensile strain (NTS), ε_t, can be calculated from a, c, and d_t as follows:
If $c = a/\beta_1$ (from Eq. 3.64) and $d_t = h - 2.5$ in., then $\varepsilon_t = 0.003\ (c - d_t)/c$. For tension-controlled section in the web, $\varepsilon_t \geq 0.005$.

The design moment strength of a T-section or I-section can be calculated from the preceding equation(3.65). It is necessary to check the following:

1. The total tension steel ratio relative to the web effective area is equal to or greater than ρ_{min}.

$$\rho_w = A_s / b_w d \geq \rho_{min}$$

$$\rho_{min} = (3\sqrt{f'_c})/f_y \geq 200/f_y \tag{3.66}$$

2. Also, check that the NTS is equal to or greater than 0.005 for tension-controlled sections.
3. The maximum tension steel (Max A_s), in a T-section must be equal to or greater than the steel ratio used, A_s, for tension-controlled sections, with $\phi = 0.9$.

$$\text{Max } A_s = A_{sf} \text{ (flange)} + \rho_{max} (b_w d) \text{ (web)} \tag{3.67}$$

$$\text{Max } A_s = (1/f_y)[0.85 f'_c t (b - b_w)] + \rho_{max} (b_w d) \tag{3.68}$$

In steel ratios, relative to the web only, divide Eq. 3.67 by $b_w d$:

$$\rho_w = A_s/b_w d \leq (\rho_{\max} + A_{sf}/b_w d) \tag{3.69}$$

Or

$$(\rho_w - \rho_f) \leq \rho_{\max} \text{ (web)} \tag{3.70}$$

where $\rho_{\max}$ is the maximum steel ratio for the basic singly reinforced web section (Table 3.2), and $\rho_f = A_{sf}/b_w d$.

A general equation for calculating (Max A_s) in a T-section when $a > t$ can be developed as follows:

$$C = 0.85 f'_c[(b_e - b_w)t + ab_w]$$

For $\varepsilon_c = 0.003$ and $\varepsilon_t = 0.005$, then $c/d = 0.003/(0.003 + 0.005) = 0.375$ (for the web). Hence, $a = \beta_1 c = 0.375\beta_1 d$.

The maximum steel area is equal to C/f_y and therefore

$$\text{Max } A_s = (0.85 f'_c/f_y)[(b_e - b_w)t + 0.375\beta_1 b_w d] \tag{3.71}$$

where Max A_s is the maximum tension steel area that can be used in a T-section when $a > t$. For example for $f'_c = 3$ ksi and $f_y = 60$ ksi, the preceding equation is reduced to:

$$\text{Max } A_s = 0.0425[(b_e - b_w)t + 0.319 b_w d] \tag{3.72}$$

For $f'_c = 4$ ksi and $f_y = 60$ ksi,

$$\text{Max } A_s = 0.0567[(b_e - b_w)t + 0.319 b_w d] \tag{3.73}$$

In summary, the procedure to analyze a T-section or inverted L-section is as follows:

1. Determine the effective width of the flange b_e (refer to Section 3.15.3). Calculate $\rho_{\max}$ and $\rho_{\min}$ (or take from tables).
2. Check if $a \leq t$ as follows: $a' = A_s f_y/(0.85 f'_c b_e)$
3. If $a' \leq t$, it is a rectangular section analysis.
   a. Calculate $\phi M_n = \phi A_s f_y(d - a/2)$, $a = a'$
   Note that $c = a/\beta_1$ and $\varepsilon_t = 0.003\ (d_t - c)/c \geq 0.005$ for tension-controlled section and $\phi = 0.9$.
   b. Check that $\rho_w = A_s/b_w d \geq \rho_{\min}$.
   c. Max A_s can be calculated from Eq. 3.68 and should be $\geq A_s$ used. When $a < t$, normally this condition is met.
4. If $a' > t$, it is a T-section analysis:
   a. Calculate $A_{sf} = 0.85 f'_c t(b_e - b_w)/f_y$
   b. Check that $(\rho_w - \rho_f) \leq \rho_{\max}$ (relative to the web area), where

   $$\rho_w = A_s/b_w d \quad \text{and} \quad \rho_f = A_{sf}/b_w d$$

   Or check that Max $A_s \geq A_s$ used in the section, for $\phi = 0.9$, (Eq. 3.71).
   c. Check that $\rho_w = A_s/b_w d \geq \rho_{\min}$. This condition is normally met when $a > t$.
   d. Calculate $a = (A_s - A_{sf})/0.85 f'_c b_w$ (for the web section).
   e. Calculate ϕM_n from Eq. 3.65.

Example 3.11

A series of reinforced concrete beams spaced at 7 ft, 10 in. on centers have a simply supported span of 15 ft. The beams support a reinforced concrete floor slab 4 in. thick. The dimensions and reinforcement of the beams are shown in Fig. 3.33. Using $f'_c = 3$ ksi and $f_y = 60$ ksi, determine the design moment strength of a typical interior beam.

Solution

1. Determine the effective flange width b_e. The effective flange width is the smallest of

$$16t + b_w = (16 \times 4) + 10 = 74 \text{ in.}$$

$$\text{Span}/4 = 15 \times 12/4 = 45 \text{ in.}$$

$$\text{Center to center of beams} = (7 \times 12) + 10 = 94 \text{ in.}$$

   Therefore, $b_e = 45$ in. controls.

2. Check the depth of the stress block. If the section behaves as a rectangular one, then the stress block lies within the flange (Fig. 3.30). In this case, the width of beam used is equal to 45 in.

$$a' = A_s f_y/(0.85 f'_c b) = 2.37 \times 60/(0.85 \times 3 \times 45) = 1.24 \text{ in.} < t$$

   Therefore, it is a rectangular section with $a = a' = 1.24$ in.

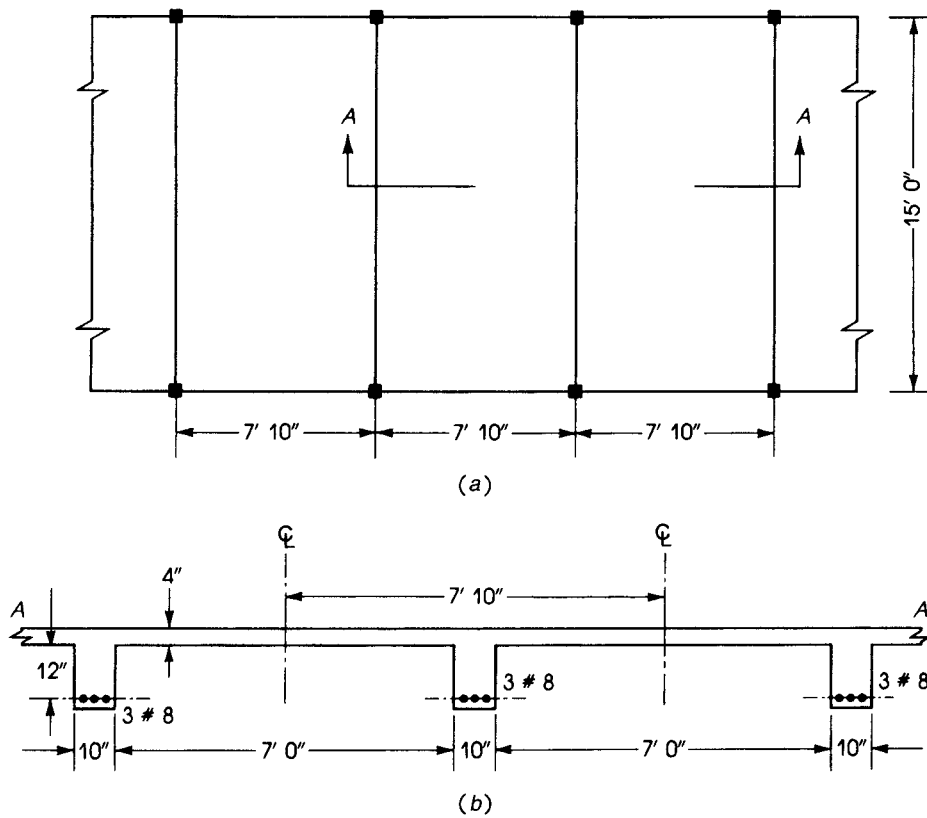


Figure 3.33 Example 3.11: (*a*) plan of slab-beam roof and (*b*) section *A-A*.

3. Check that

$$\rho_w = \frac{A_s}{b_w d} \geq \rho_{\min} = 0.00333$$

$$\rho_w = \frac{2.37}{(10 \times 16)} = 0.0148 > 0.00333$$

4. Check ε_t : $a = 1.24$ in., $c = 1.24/0.85 = 1.46$ in., $d_t = d = 16$ in.

$$\varepsilon_t = 0.003(d_t - c)/c = 0.003(16 - 1.46)/1.46 = 0.0299 > 0.005, \phi = 0.9$$

5. Calculate $\phi M_n = \phi A_s f_y(d - a/2) = 0.9(2.37)(60)(16 - 1.24/2)$

$$= 1968 \text{ K·in.} = 164 \text{ K·ft.}$$

6. You may check that A_s used is less than or equal to A_s (Eq. 3.72), which is not needed when $a < t$:

$$\text{Max } A_s = 0.0425[(45 - 10) + 0.31 \times 10 \times 16] = 8.11 \text{ in.}^2 > 2.37 \text{ in.}^2$$

Example 3.12

Calculate the design moment strength of the T-section shown in Fig. 3.34 using $f'_c = 3.5$ ksi and $f_y = 60$ ksi.

Solution

1. Given $b = b_e = 36$ in., $b_w = 10$ in., $d = 17$ in., and $A_s = 6.0$ in.^{2}, check if $a \leq t$:

$$a' = A_s f_y/(0.85 f'_c b) = 6 \times 60/(0.85 \times 3.5 \times 36) = 3.36 \text{ in.}$$

Since $a' > t$, it is a T-section analysis.

2. $A_{sf} = 0.85 f'_c t(b - b_w)/f_y = 0.85 \times 3.5 \times 3(36 - 10)/60 = 3.87$ in.^{2}.$(A_s - A_{sf}) = A_{s1}$ (web) $= 6 - 3.87 = 2.13$ in.^{2}

3. Check ε_t:a (web) $= A_{s1} f_y/(0.85 f'_c b_w) = 2.13 \times 60/(0.85 \times 3.5 \times 10) = 4.3$ in. c $= 4.3/0.85 = 5.06$ in., $d_t = 20.5 - 2.5 = 18$ in., and $c/d_t = 0.281 < 0.375$ Or, $\varepsilon_t = 0.003(d_t - c)/c = 0.0077 > 0.005$, then $\phi = 0.9$

4. Check that $A_s > A_{s\min}$, $\rho_{\min} = 0.00333$

$$A_{s\min} = 0.00333 \times 10 \times 17 = 0.57 \text{ in.}^2$$

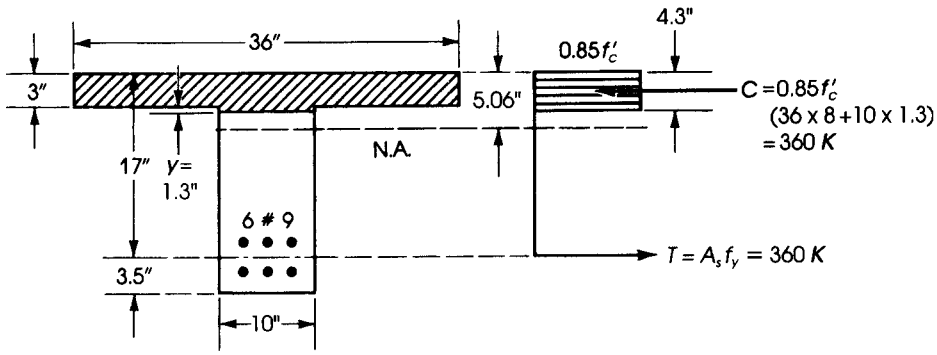


Figure 3.34 Example 3.12.

5. Calculate ϕM_n using Eq. 3.65:

$$\phi M_n = \phi\left[(A_s - A_{sf})f_y\left(d - \frac{a}{2}\right) + A_{sf}f_y\left(d - \frac{t}{2}\right)\right]$$

$$= 0.9\left[2.13 \times 60\left(17 - \frac{4.3}{2}\right) + 3.87 \times 60\left(17 - \frac{3}{2}\right)\right]$$

$$= 4947 \text{ K·in.} = 412.3 \text{ K·ft}$$

Another approach to check whether $a \leq t$ is to calculate the tension force, $T = A_s f_y$, and compare it to the compressive force in the total flange (Fig. 3.34).

$$T = A_s f_y = 60 \times 60 = 360 \text{ K}$$

$$C = 0.85 f'_c t b_e = 0.85 \times 3.5 \times 3 \times 36 = 321.3 \text{ K}$$

Since T exceeds C, then $a \leq t$, and the section acts as a T-section.

An additional area of concrete should be used to provide the difference of (360 − 321.3) = 38.7 K. This area has a width of 10 in. and a depth of y. Therefore,

$$b_w y(0.85 f'_c) = 38.7 \text{ K or } 10(y)(0.85 \times 3.5) = 38.7 \text{ K}$$

$y = 1.3$ in., and $a = y + t = 1.3 + 3 = 4.3$ in., as calculated earlier.

3.16 DIMENSIONS OF ISOLATED T-SHAPED SECTIONS

In some cases, isolated beams with the shape of a T-section are used in which additional compression area is provided to increase the compression force capacity of sections. These sections are commonly used as prefabricated units.

The ACI Code, Section 8.10.4, specifies the size of isolated T-shaped sections as follows:

1. Flange thickness, t, shall be equal to or greater than one-half of the width of the web, b_w.
2. Total flange width b shall be equal to or less than four times the width of the web, b_w (Fig. 3.35).

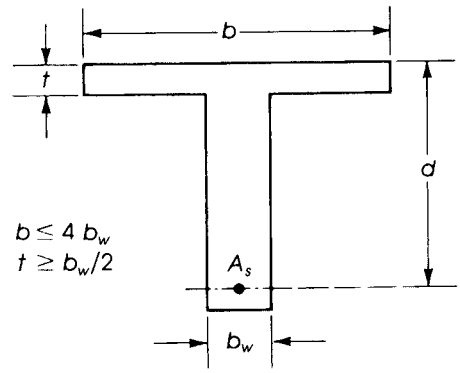


Figure 3.35 Isolated T-shaped sections.

3.17 INVERTED L-SHAPED SECTIONS

In slab-beam girder floors, the end beam is called a *spandrel beam*. This type of floor has part of the slab on one side of the beam and is cast monolithically with the beam. The section is unsymmetrical under vertical loading (Fig. 3.36*a*). The loads on slab S_1 cause torsional moment uniformly distributed on the spandrel beam B_1. Design for torsion is explained later. The over-hanging flange width $(b - b_w)$ of a beam with the flange on one side only is limited by the ACI Code, Section 8.10.2, to the smallest of the following:

1. One-twelfth of the span of the beam
2. Less than or equal to six times the thickness of the slab
3. Less than or equal to one-half the clear distance to the next beam.

If this is applied to the spandrel beam in Fig. 3.36*b*, then

1. $(b - 12) \leq (20 \times 12)/12 = 20$ in. (controls)
2. $(b - 12) \leq 6 \times 6 = 36$ in.
3. $(b - 12) \leq 3.5 \times 12 = 42$ in.

Therefore, the effective flange width is $b = 20 + 12 = 32$ in., and the effective dimensions of the spandrel beam are as shown in Fig. 3.36*d*.

3.18 SECTIONS OF OTHER SHAPES

Sometimes a section different from the previously defined sections is needed for special requirements of structural members. For instance, sections such as those shown in Fig. 3.37 may be used in the precast concrete industry. The analysis of such sections is similar to that of a rectangular section, taking into consideration the area of the removed or added concrete. The next example explains the analysis of such sections.

Example 3.13

The section shown in Fig. 3.38 represents a beam in a structure containing prefabricated elements. The total width and total depth are limited to 14 and 21 in., respectively. Tension reinforcement used is four no. 9 bars. Using $f'_c = 4$ ksi and $f_y = 60$ ksi., determine the design moment strength of the section.

Solution

1. Determine the position of the neutral axis based on $T = 4 \times 60 = 240$ K.

$$240 = 0.85 f'_c [2(4 \times 5) + 14(a - 4)]$$

where a = depth of the equivalent compressive block needed to produce a total compressive force of 240 K.

If $240 = (0.85 \times 4)(40 + 14a - 56)$, then $a = 6.18$ in. and $c = a/0.85 = 7.28$ in.

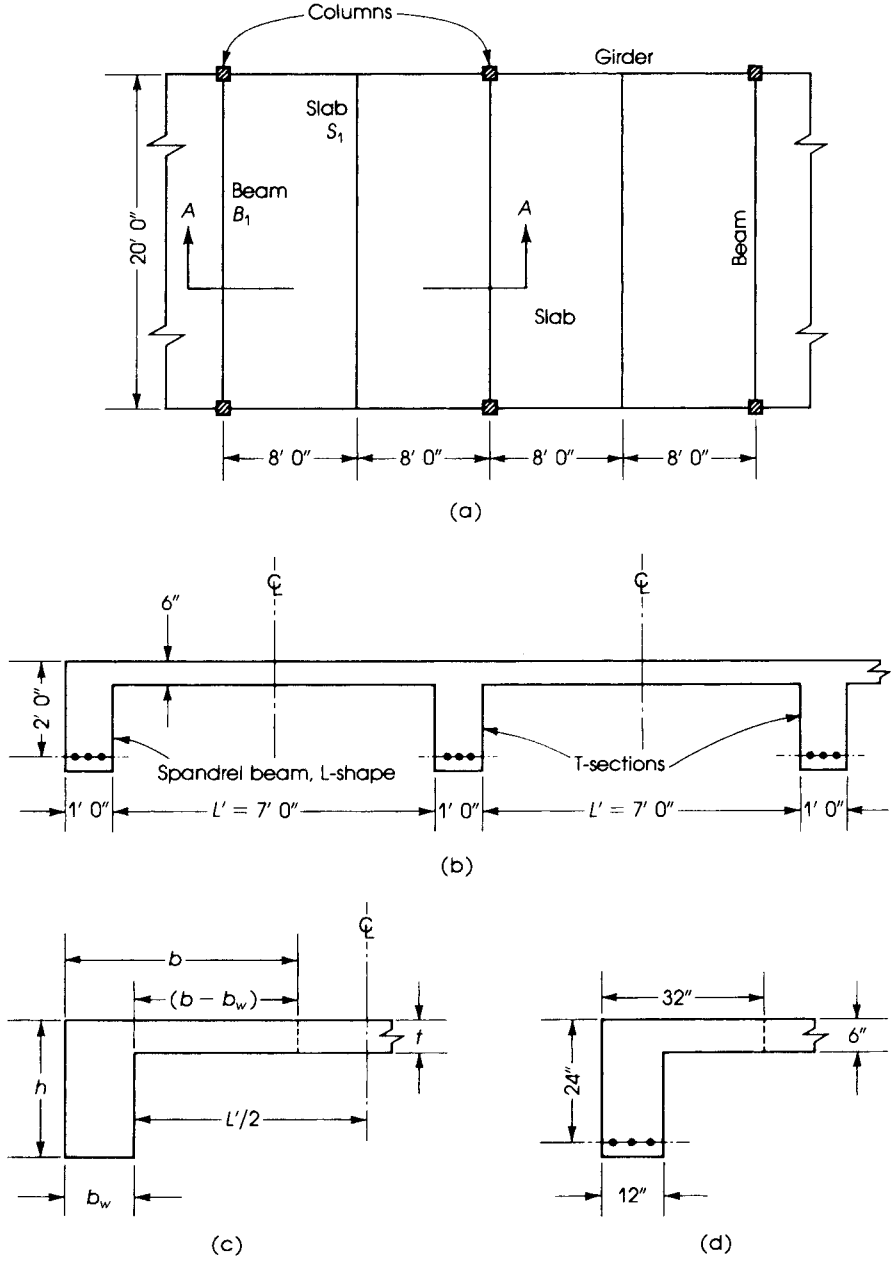


Figure 3.36 Slab-beam-girder floor, showing (*a*) plan, (*b*) section including spandrel beam, (*c*) dimensions of the spandrel beam, and (*d*) its effective flange width.

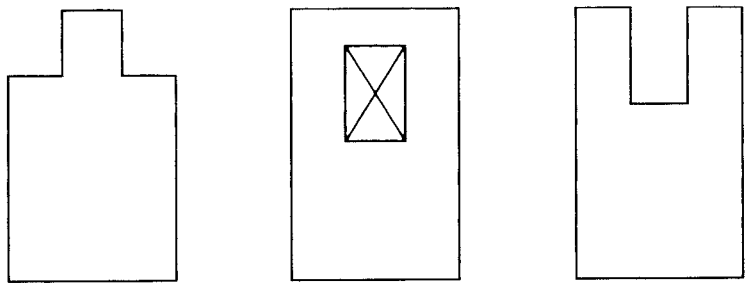


Figure 3.37 Sections of other shapes.

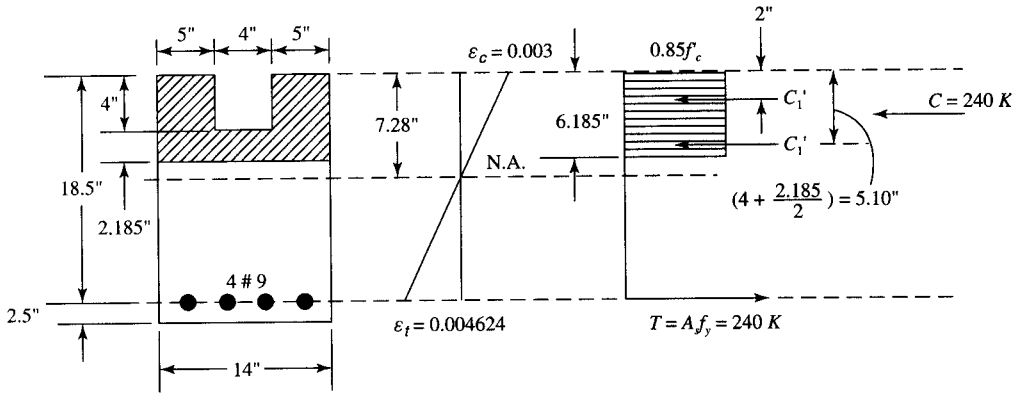


Figure 3.38 Example 3.13: (*a*) balanced and (*b*) under-reinforced sections.

2. Calculate M_n by taking moments of the two parts of the compressive forces (each by its arm), about the tension steel.

$$C_1' = \text{compressive force on the two small areas, } 4 \times 5 \text{ in.}$$
$$= 0.85 \times 4\,(2 \times 4 \times 5) = 136 \text{ K.}$$
$$C_1'' = \text{compressive force on area, } 14 \times 2.185$$
$$= 0.85 \times 4 \times 14 \times 2.185 = 104 \text{ K.}$$
$$M_n = C_1'(d - 2) + C_1''(d - 5.10)$$
$$= 136 \times 16.5 + 104 \times 13.4 = 3637.6 \text{ K·in.} = 303.1 \text{ K·ft}$$

3. Calculate $\varepsilon_t = 0.003(d_t - c)/c$, where $d_t = 18.5$ in.

$$\varepsilon_t = 0.003(18.5 - 7.28)/7.28 = 0.004624 < 0.005 \text{ but} > 0.004$$

Section is in the transition region and $\phi < 0.9$.

$$\phi = 0.48 + 83\varepsilon_t = 0.864$$
$$\phi M_n = 0.864(303.1) = 261.9 \text{ K·ft}$$

3.19 ANALYSIS OF SECTIONS USING TABLES

Reinforced concrete sections can be analyzed and designed using tables shown in Appendix A (for U.S. customary units) and Appendix B (for SI units). The tables give the value of R_u as related to the steel ratio, ρ, in addition to the maximum and minimum values for ρ and R_u. When the section dimensions are known, R_u is calculated; then ρ and A_s are determined from tables. The values in the tables are calculated based on tension-controlled sections with $\phi = 0.9$. If ϕ is less than 0.9 (transition region), the values of R_u should be multiplied by the ratio $\phi/0.9$.

$$\phi M_n = R_u b d^2 \qquad R_u = M_u/bd^2 = \phi\rho f_y[1 - \rho f_y/1.7f'_c]$$

$$A_s = \rho bd \quad \text{and} \quad \rho = A_s/bd$$

For any given value of ρ, R_u can be determined from tables. Then ϕM_n can be calculated. The values of ρ and R_u range between a minimum value of R_u (min) when ρ minimum is used, to a maximum value as limited by the ACI Code, when ρ is equal to ρ (max), for tension controlled sections with $\phi = 0.9$.

The use of tables will reduce the manual calculation time. The next example explains the use of tables.

Example 3.14

Calculate the design moment strength of the section shown in Example 3.2, Fig. 3.14 using tables. Use $b = 12$ in., $d = 21$ in., $f'_c = 3$ ksi, $f_y = 60$ ksi and three no. 9 bars.

Solution

1. Using three no. 9 bars, $A_s = 3.0\,\text{in.}^2$, $\rho = A_s/bd = 3.0/(12 \times 21) = 0.0119$. From Table 3.2, $\rho_{max} = 0.01356 > \rho$ used. Therefore, $\phi = 0.9$, and it is a tension-controlled section.
   From Table A1, for $\rho = 0.0119$, $f'_c = 3$ ksi and $f_y = 60$ ksi, get $R_u = 553$ psi (by interpolation).
2. Calculate $\phi M_n = R_u b d^2 = 0.553\ (12)(21)^2 = 2926$ K·in. $= 243.8$ K·ft

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3.20 ADDITIONAL EXAMPLES

The following examples are introduced to enhance the understanding of the analysis and design applications.

Example 3.15

Calculate the design moment strength of the precast concrete section shown in Fig. 3.39 using $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. The section behaves as a rectangular section with $b = 14$ in., and $d = 21.5$ in. Note that the width b is that of the section on the compression side.
2. Check that $\rho = A_s/bd = 5/(14 \times 21.5) = 0.01661$, which is less than the maximum steel ratio of 0.018 for tension-controlled sections. Therefore, $\phi = 0.9$. Also $\rho > \rho_{min} = 0.00333$. Therefore, ρ is within the limits of a tension-controlled section.
3. Calculate a : $a = A_s f_y/(0.85 f'_c b) = 5 \times 60/(0.85 \times 4 \times 14) = 6.3$ in.

$$\phi M_n = \phi A_s f_y(d - a/2) = 0.9 \times 5 \times 60(21.5 - 6.3/2) = 4954.5\ \text{K·in} = 412.9\ \text{K·ft}.$$

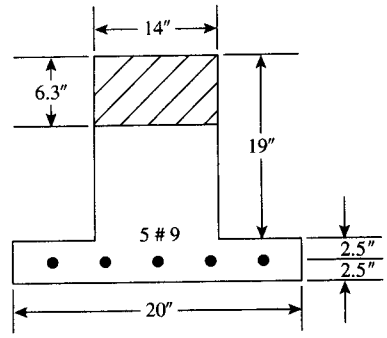


Figure 3.39 Example 3.15.

Example 3.16

A reinforced concrete beam was tested to failure and had a rectangular section, $b = 14$ in., and $d = 18.5$ in. At ultimate moment (failure), the strain in the tension steel was recorded and was equal to 0.004106. The strain in the concrete at failure may be assumed to be 0.003. If $f'_c = 3$ ksi and $f_y = 60$ ksi, it is required to:

1. Check if the tension steel has yielded.
2. Calculate the steel area provided in the section to develop the above strains. Then calculate the applied moment.
3. Calculate the design moment strength based on the ACI Code provisions. (Refer to Fig. 3.40.)

Solution

1. Check the strain in the tension steel relative to the yield strain. The yield strain $\varepsilon_y = f_y/E_s = 60/29{,}000 = 0.00207$. The measured strain in the tension steel is equal to 0.004106, which is much greater than 0.00207, indicating that the steel bars have yielded and in the elastoplastic range. The concrete strain was 0.003 indicating that the concrete has failed and started to crush. Therefore, the tension steel has yielded.
2. Calculate the depth of the neutral axis c from the strain diagram. (Fig. 3.40). From the triangles of the strain diagram,

$$c/d = 0.003/(0.003 + 0.004106) \quad \text{and} \quad c = 18.5\left(\frac{3}{7.106}\right) = 7.81 \text{ in.}$$

$$a = \beta_1 c = 0.85 \times 7.81 = 6.64 \text{ in.}$$

The compression force in the concrete, $C_c = 0.85\, f'_c ab = 0.85 \times 3 \times 6.64 \times 14 = 237$ K. The tension steel $A_s = C_c/f_y = 237/60 = 3.95 \text{ in.}^2$ (section has five no. 8 bars).

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 3.95 \times 60(18.5 - 6.64/2) = 3597.6 \text{ K·in} = 299.8 \text{ K·ft}$$

3. Check $\varepsilon_t = 0.003(d_t - c)/c$.

$$c = 7.81 \text{ in.}, \; d_t = h - 2.5 \text{ in.} = 22 - 2.5 = 19.5 \text{ in.}$$

$\varepsilon_t = 0.003(19.5 - 7.81)/7.81 = 0.0049$, which is less than 0.005 for tension-controlled sections, but greater than 0.004. Section is in the transition region, and $\phi < 0.9$.

$$\phi = 0.48 + 83\varepsilon_t = 0.853$$

The allowable design moment $= \phi M_n = 0.863 \times 299.8 = 255.6$ K·ft.

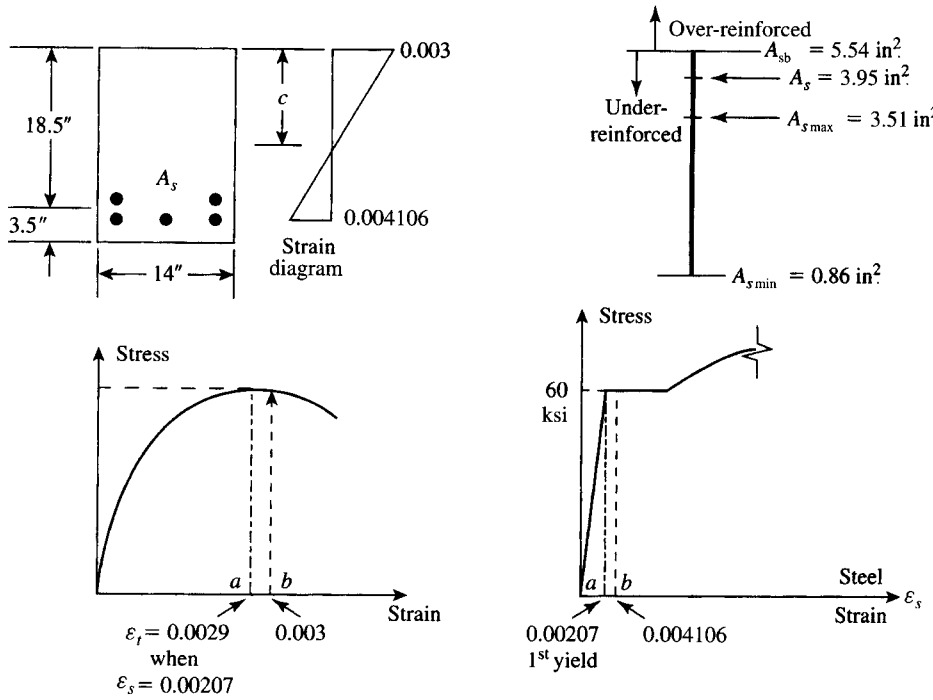


Figure 3.40 Example 3.16.

Discussion

From Table 3.2, $\rho_b = 0.0214$ and $\rho_{max} = 0.01356$. For comparison, A_s (max) $= 0.01356(14 \times 18.5) = 3.51$ in.^{2} for $\phi = 0.9$, and A_s (balanced) $= 5.54$ in.^{2}. The ratio of $A_s/A_{s\,max} = 3.95/3.51 = 1.125$ and $A_s/A_{sb} = 0.713$. If $A_s = A_{max} = 3.51$ in.^{2} is used with $\phi = 0.9$, then

$$a = 3.51 \times 60/(0.85 \times 3 \times 14) = 5.9 \text{ in.}$$

and

$$\phi M_n = 0.9 \times 3.51 \times 60(18.5 - 5.9/2) = 2947.2 \text{ K·in.} = 245.6 \text{ K·ft.}$$

which is equal to 96% of the moment calculated above. Figure 3.40 shows the behavior of the tested beam.

3.21 EXAMPLES USING SI UNITS

The following equations are some of those mentioned in this chapter but converted to SI units. The other equations, which are not listed here, can be used for both U.S. Customary and SI units. Note that f'_c and f_y are in MPa (N/mm^2).

$$\rho_b = 0.85\beta_1(f'_c/f_y)[600/(600 + f_y)] \tag{3.18}$$

For tension-controlled condition,

$$\rho_{max} = (0.003 + f_y/E_s)\rho_b/0.008 \tag{3.31}$$

$$(\rho - \rho') \geq 0.85\beta_1(f'_c/f_y)(d'/d)[600/(600 - f_y)] = K \tag{3.49}$$

Example 3.17

Determine the design moment strength and the position of the neutral axis of a rectangular section that has $b = 300$ mm, $d = 500$ mm, and is reinforced with five 20-mm-diameter bars. Given $f'_c = 20$ MPa and $f_y = 400$ MPa.

Solution

1. Area of five 20-mm bars is 1570 mm^2.

$$\rho = A_s/bd = 1570/(300 \times 500) = 0.01047 \quad \rho_{min} = 1.4/f_y = 0.0035$$

For $f'_c = 20$ MPa and $f_y = 400$ MPa, $\rho_b = 0.0217$ and $\rho_{max} = 0.01356$. Note that $E_s = 200{,}000$ MPa and $f_y/E_s = 0.002$. Because $\rho < \rho_{max}$, it is a tension-controlled section with $\phi = 0.9$. Also $\rho > \rho_{min}$.

2. Calculate the design moment strength:

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right)$$

$$a = A_s f_y/(0.85 f'_c b) = 1570 \times 400/(0.85 \times 20 \times 300) = 123 \text{ mm}$$

$$\phi M_n = 0.9 \times 1570 \times 400\left(500 - \frac{123}{2}\right) \times 10^{-6} = 247.8 \text{ KN·m}$$

Note that the moment was multiplied by 10^{-6} to get the answer in KN·m. The distance to the neutral axis from the compression fibers $(c) = a/\beta_1$, where $\beta_1 = 0.85$ for $f'_c < 30$ MPa. Therefore, $c = 123/0.85 = 145$ mm.

Example 3.18

A 2.4-m-span cantilever beam has a rectangular section with $b = 300$ mm, $d = 490$ mm, and is reinforced with three bars, 25 mm in diameter. The beam carries a uniform dead load (including its own weight) of 25.5 KN/m and a uniform live load of 32 KN/m. Check the adequacy of the section if $f'_c = 30$ MPa and $f_y = 400$ MPa.

Solution

1. $U = 1.2D + 1.6L = 1.2 \times 25.5 + 1.6 \times 32 = 81.8$ KN/m. External factored moment $= M_u = UL^2/2 = 81.8(2.4^2)/2 = 235.6$ KN·m.

2. Calculate the design moment strength:

$$A_s = 1470 \text{ mm}^2 \quad \rho = A_s/bd = 1470/(300 \times 490) = 0.01$$

$$\rho_b = 0.85\beta_1(f'_c/f_y)[600/(600 + f_y)] = 0.0325$$

$$\rho_{max} = (0.005/0.008)\rho_b = \left(\frac{5}{8}\right)(0.0325) = 0.0203, \quad \rho_{min} = \frac{1.4}{400} = 0.0035$$

Since $\rho < \rho_{max}$ but $> \rho_{min}$, it is a tension-controlled section and $\phi = 0.9$. $a = A_s f_y/(0.85 f'_c b) = 1470 \times 400/(0.85 \times 30 \times 300) = 77$ mm, $c = 90$ mm. $\phi M_n = \phi A_s f_y(d - a/2) = 0.9 \times 1470 \times 400(490 - 77/2) \times 10^{-6} = 238.9$ KN·m. $\varepsilon_t = 0.003(d_t - c)/c = 0.003\ (490 - 90)/90 = 0.01333 > 0.005$, $\phi = 0.9$ as assumed.

3. The internal design moment strength is greater than the external factored moment. Therefore, the section is adequate.

Example 3.19

Calculate the design moment strength of a rectangular section with the following details: $b = 250$ mm, $d = 440$ mm, $d' = 60$ mm, tension steel is six bars 25 mm in diameter (in two rows), compression steel is three bars 20 mm in diameter, $f'_c = 20$ MPa and $f_y = 350$ MPa.

Solution

1. Check if compression steel yields:

$$A_s = 490 \times 6 = 2940 \text{ mm}^2, \quad A'_s = 314 \times 3 = 942 \text{ mm}^2 \quad A_s - A'_s = 1998 \text{ mm}^2$$

$$\rho = 2940/(250 \times 440) = 0.0267 \quad \rho' = 942/(250 \times 440) = 0.00856$$

$$\rho - \rho' = 0.01814.$$

For compression steel to yield:

$$(\rho - \rho') \geq 0.85 \times 0.85 \times (20/350)(60/440)(600/600 - 350) = 0.01351$$

$(\rho - \rho') = 0.01814 > 0.01351$. Therefore, compression steel yields.

2. Calculate M_n:

$$a = (A_s - As')/0.85 f'_c b = 1998/(0.85 \times 20 \times 250) = 164 \text{ mm}$$

$$M_n = [1998 \times 350\left(440 - \frac{164}{2}\right) + 942 \times 350(440 - 60)] \times 10^{-6} = 417.3 \text{ KN·m}$$

3. Check ϕ based on $\varepsilon_t \geq 0.005$.

$$\varepsilon_t = 0.003(d_t - c)/c \quad a = 164 \text{ mm} \quad c = 164/0.85 = 193 \text{ mm}$$

$$d_t = h - 65 \text{ mm} = d + 25 \text{ mm for two rows of tension bars.}$$

$$d_t = 440 + 25 = 465 \text{ mm}$$

$\varepsilon_t = 0.003(465 - 193)/193 = 0.04228$, which is less than 0.005, but greater than the 0.004 limit. $\phi = 0.48 + 83 \times \varepsilon_t, = 0.831$, and $\phi M_n = 0.831\ (417.3) = 346.8$ KN·m.

SUMMARY

Flow charts for the analysis of sections are given on www.wiley.com/college/hassoun.

Section 3.1–3.8

1. The type of failure in a reinforced concrete flexural member is based on the amount of tension steel used, A_s.
2. Load factors for dead and live loads are $U = 1.2D + 1.6L$. Other values are given in the text.
3. The reduction strength factor for beams (ϕ) = 0.9 for tension controlled sections with $\varepsilon_t \geq 0.005$.
4. An equivalent rectangular stress block can be assumed to calculate the design moment strength of the beam section, ϕM_n.
5. Design provisions are based on four conditions, Section 3.5

Section 3.9–3.13: Analysis of a Singly Reinforced Rectangular Section

Given: f'_c, f_y, b, d, and A_s. Required: the design moment strength, ϕM_n.

To determine the design moment strength of a singly reinforced concrete rectangular section,

1. Calculate the compressive force, $C = 0.85f'_cab$ and the tensile force, $T = A_sf_y$. Calculate $a = A_sf_y/(0.85f'_cb)$. Calculate $\phi M_n = \phi C(d - a/2) = \phi T(d - a/2) = \phi A_sf_y(d - a/2)$. Check $\varepsilon_t = 0.003(d_t - c)/c \geq 0.005$ for $\phi = 0.9$ (tension-controlled section). (See Section 3.6.)
2. Calculate the balanced, maximum, and minimum steel ratios:

$$\rho_b = 0.85\beta_1(f'_c/f_y)[87/(87 + f_y)) \quad \rho_{max} = (0.003 + f_y/E_s)\rho_b/0.008$$

$$\rho_{min} = 0.2/f_y \text{ for } f'_c \leq 4.5 \text{ ksi}$$

(where f'_c and f_y are in ksi. (See Section 3.9.2.) The steel ratio in the section is $\rho = A_s/bd$. Check that $\rho_{min} \leq \rho \leq \rho_{max}$.

3. Another form of the design moment strength is

$$M_n = \rho f_y(bd^2)(1 - \rho f_y/1.7f'_c) = R_nbd^2$$

$$R_n = \rho f_y[1 - (\rho f_y/1.7f'_c)] \quad \text{and} \quad R_u = \phi R_n$$

4. For $f_y = 60$ ksi and $f'_c = 3$ ksi (Table 3.2), $\rho_{max} = 0.01356$, $\rho_{min} = 0.00333$, $R_n = 686$ psi, and $R_u = 615$ psi.

   For $f_y = 60$ ksi and $f_c' = 4$ ksi, $\rho_{max} = 0.01806$, $\rho_{min} = 0.00333$, $R_n = 911$ psi, and $R_u = 820$ psi.

Section 3.14: Analysis of Rectangular Section with Compression Steel

Given: b, d, d', A_s, A'_s, f'_c, and f_y. Required: the deign moment strength, ϕM_n.

1. Calculate $\rho = A_s/bd$, $\rho' = A_s/bd$, and $(\rho - \rho')$.
2. Calculate ρ_b, ρ_{max}, and ρ_{min} as given above (or see Section 3.10)
3. Calculate $K = 0.85\beta_1(f'_c/f_y)(d'/d)[87/(87 - f_y)]$. ($f'_c$ and f_y are in ksi.)
4. When compression steel yields,
   a. Check that $\rho \geq \rho_{min}$.
   b. Check that $(\rho - \rho') \geq K$ for compression steel to yield. If not, then compression steel does not yield.
   c. If compression steel yields, then $f'_s = f_y$.
   d. Check that $\rho \leq (\rho_{max} + \rho')$ or $(\rho - \rho') \leq \rho_{max}$.
   e. Calculate $a = (A_s - A'_s)f_y/(0.85f'_cb)$.
   f. Calculate $\phi M_n = \phi(A_s - A'_s)f_y(d - a/2) + \phi A'_sf_y(d - d')$.
   g. If $(\rho - \rho') > \rho_{max}$ but $< \rho_{max\,t}$ (for the transition region), then $\phi < 0.9$ for M_{u1} and $\phi = 0.9$ for M_{u2} (Eq. 3.44 *a*).
5. When compression steel does not yield,
   a. Compression steel does not yield when $(\rho - \rho') < K$. The value of f'_s is not known.
   b. Calculate c = the distance to the neutral axis from the compression fibers as follows:

$$A_1c^2 + A_2c + A_3 = 0,$$

where

$$A_1 = 0.85 f'_c \beta_1 b$$
$$A_2 = A'_s(87 - 0.85 f_c) - A_s f_y$$
$$A_3 = -87 A'_s d'.$$

Solve for c.

An alternative solution to calculate c is as follows:

$$C + C' = T$$
$$C = 0.85 f'_c(\beta_1 cb - A'_s) \quad C' = A'_s[87(c - d')/c] - 0.85 f'_c A'_s$$

and

$$T = A_s f_y$$

Solve for c.

c. Calculate $f'_s = 87(c - d')/c \leq f_y$(in ksi).

d. Check that $\rho \leq [\rho_{\max} + \rho'(f'_s/f_y)]$ or $A_s \leq [\rho_{\max}(bd) + A'_s(f'_s/f_y)]$.

e. Calculate a:

$$a = (A_s f_y - A'_s f'_s)/(0.85 f'_c b) \quad \text{or} \quad a = \beta_1 c$$

f. Calculate ϕM_n:

$$\phi M_n = \phi[(A_s f_y - A'_s f'_s)(d - a/2) + A'_s f'_s(d - d')]$$

Note that $(A_s f_y - A'_s f_y) = A_{s1} = A_s - A_{s2} = A_s - (A'_s f'_s/f_y)$ and $A_{s2} f_y = A'_s f'_s$. Also, $a = A_{s1} f_y/(0.85 f'_c b)$

Sections 3.15–3.17: Analysis of T-Sections

Given: f'_c, f_y, A_s, and section dimensions. Required: design moment strength, ϕM_n. Two possible cases may develop. (Determine the effective flange width, b_e, first.)

Case 1

1. If $a \leq t$ (the slab thickness), then it is a T-section shape but acts as a singly reinforced rectangular section using $b = b_e$ (the flange effective width) to calculate ϕM_n.

$$a' = A_s f_y/(0.85 f'_c b_e) \leq t$$

Or, check that A_c (the area of concrete in compression) $= A_s f_y/(0.85 f'_c) \leq bt$. If $A_c \geq bt$, then it is a T-section analysis.

2. If $a' \leq t$ or $A_c \leq bt$, then $a' = a$ and $\phi M_n = \phi A_s f_y(d - a/2)$.
3. Check that ρ_w (steel ratio in web) $= A_s/b_w d \geq \rho_{\min}$.
4. Check that $A_s \leq A_{s\max}$ from Eq. 3.71. (Normally, this is o.k. for this case.)

$$A_{s\max} = 0.6375(f'_c/f_y)[t(b - b_w) + (0.375) b_w \beta_1 d]$$

5. Check that $\varepsilon_t \geq 0.005$ for $\phi = 0.9$. (Normally this is o.k. for this case.)
6. The effective flange width $b = b_e$ is the smallest of
   a. Span/4

b. Center to center of adjacent slabs

c. $(b_w + 16t)$, where t = slab thickness

Case 2

1. When $a > t$ or $A_c > bt$, it is a T-section analysis.
2. For the flange, $C_f = 0.85f'_c t(b - b_w) = A_{sf}f_y$, calculate $A_{sf} = C_f/f_y$.
3. For the web,

$$A_{sw} = \text{tension steel in the web} = A_s - A_{sf}$$

$$a = (A_s - A_{sf})f_y/(0.85f'_c b_w)$$

$$C_w(\text{web}) = 0.85f'_c ab_w = A_{sw}f_y$$

4.
$$\begin{aligned}\phi M_n &= \phi[M_w(\text{web}) + M_f(\text{flange})] = \phi[C_w(d - a/2) + C_f(d - t/2)] \\ &= \phi[0.85f'_c ab_w(d - a/2) + 0.85f'_c t(b - b_w)(d - t/2)] \\ &= \phi[(A_s - A_{sf})f_y(d - a/2) + A_{sf}f_y(d - t/2)]\end{aligned}$$

5. Check that $\varepsilon_t \geq 0.005$ for tension-controlled section and $\phi = 0.9$. (See Example 3.12).
6. Check that $A_{s\,\min} \leq A_s \leq A_{s\,\max}$. (See case 1.)

Sections 3.18–3.21

1. Analysis of nonuniform sections is explained in Example 3.13.
2. Tables in Appendix A may be used for the analysis of rectangular sections.
3. Examples in SI units are introduced.

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PROBLEMS

3.1 Singly reinforced rectangular sections. Determine the design moment strength of the sections given in the following table, knowing that $f'_c = 4$ ksi and $f_y = 60$ ksi. (Answers are given in the right column.)

No.	b (in.)	d (in.)	A_s (in.^{2})	ϕM_n (K·ft)
a	14	22.5	5.08 (4 no. 10)	441.2
b	18	28.5	7.62 (6 no. 10)	849.1
c	12	23.5	4.00 (4 no. 9)	370.1
d	12	18.5	3.16 (4 no. 8)	230.0
e	16	24.5	6.35 (5 no. 10)	600
f	14	26.5	5.00 (5 no. 9)	525.3
g	10	17.5	3.00 (3 no. 9)	200.5
h	20	31.5	4.00 (4 no. 9)	535.2

For problems in SI units, 1 in. = 25.4 mm, 1 in.^{2} = 645 mm^2, 1 ksi = 6.9 MPa (N/mm^2), and 1 M_u (K·ft) = 1.356 kN·m.

3.2 Rectangular section with compression steel. Determine the design moment strength of the sections given in the following table, knowing that f'_c4 ksi, $f_y = 60$ ksi, and $d' = 2.5$ in. (Answers are given in the right column. In the first four problems, $f'_s = f_y$)

No.	b (in.)	d (in.)	A_s (in.^{2})	A'_s(in.^{2})	ϕM_n (K·ft)
a	15	22.5	8.0 (8 no. 9)	2.0 (2 no. 9)	692.2
b	17	24.5	10.08 (8 no. 10)	2.54 (2 no. 10)	950
c	13	22	7.00 (7 no. 9)	1.8 (3 no. 7)	590.2
d	10	21.5	5.08 (4 no. 10)	1.2 (2 no. 7)	464.7
e	14	20.5	7.62 (6 no. 10)	2.54 (2 no. 10)	597.9
f	16	20.5	9.0 (9 no. 9)	4.0 (4 no. 9)	716.3
g	20	18.0	12.0 (12 no. 9)	6.0 (6 no. 9)	820.3
h	18	20.5	10.16 (8 no. 10)	5.08 (4 no. 10)	813.7

For problems in SI units: 1 in. = 25.4 mm, 1 in.^{2} = 645 mm^2, 1 ksi = 6.9 MPa (N/mm^2), and 1 M_u(K·ft) = 1.356 kN·m.

3.3 T-sections. Determine the design moment strength of the T-sections given in the following table, knowing that $f'_c = 3$ ksi and $f_y = 60$ ksi. (Answers are given in the right column. In the first three problems, $a < t$.)

No.	b (in.)	b_w (in.)	t (in.)	d (in.)	A_s (in.^{2})	ϕM_n (K·ft)
a	54	14	3	17.5	5.08 (4 no. 10)	374.8
b	48	14	4	16.5	4.0 (4 no. 9)	279.4
c	72	16	4	18.5	10.16 (8 no. 10)	769.9
*d	32	16	3	15.5	6.0 (6 no. 9)	N.G.
e	44	12	4	20.5	8.0 (8 no. 9)	660.1
f	50	14	3	16.5	7.0 (7 no. 9)	466.8
g	40	16	3	16.5	6.35 (5 no. 10)	415.0
h	42	12	3	17.5	6.0 (6 no. 9)	425.8

For problems in SI units: 1 in. = 25.4 mm, 1 in.^{2} = 645 mm^2, 1 ksi = 6.9 MPa (N/mm^2), and 1 M_u (K·ft) = 1.356 kN·m.
*Answer = 325.5 K·ft if ρ_{max} is used.

3.4 Calculate ρ_b, ρ_{max}, R_u(max), R_u, a/d, and max (a/d) for a rectangular section that has a width of b = 12 in. (300 mm) and an effective depth of d = 20 in. (500 mm) for the following cases:

a. $f'_c = 3$ ksi, $f_y = 40$ ksi, A_s = four no. 8 bars
b. $f'_c = 4$ ksi, $f_y = 60$ ksi, A_s = four no. 7 bars
c. $f'_c = 4$ ksi, $f_y = 75$ ksi, A_s = four no. 9 bars
d. $f'_c = 5$ ksi, $f_y = 60$ ksi, A_s = four no. 9 bars
e. $f'_c = 30$ MPa, $f_y = 400$ MPa, $A_s = 3 \times 30$ mm
f. $f'_c = 20$ MPa, $f_y = 300$ MPa, $A_s = 3 \times 25$ mm
g. $f'_c = 30$ MPa, $f_y = 500$ MPa, $A_s = 4 \times 25$ mm
h. $f'_c = 25$ MPa, $f_y = 300$ MPa, $A_s = 4 \times 20$ mm

3.5 Using the ACI Code requirements, calculate the design moment strength of a rectangular section that has a width of b = 250 mm (10 in.) and an effective depth of d = 550 mm (22 in.) when $f'_c = 20$ MPa (3 ksi), $f_y = 420$ MPa (60 ksi), and the steel used is as follows:

a. 4 × 20 mm b. 3 × 25 mm c. 4 × 30 mm
d. 2 no. 9 bars e. 6 no. 9 bars

3.6 A reinforced concrete simple beam has a rectangular section with a width of b = 8 in. (200 mm) and effective depth of d = 18 in. (450 mm). At design moment (failure), the strain in the steel was recorded and was equal to 0.0015. (The strain in concrete at failure may be assumed to be 0.003.) Use $f'_c = 3$ ksi (20 MPa) and $f_y = 50$ ksi (350 MPa) for all parts.

a. Check if the section is balanced, under-reinforced, or over-reinforced.
b. Determine the steel area that will make the section balanced.
c. Calculate the steel area provided in the section to produce the aforementioned strains, and then calculate its moment. Compare this value with the design moment strength allowed by the ACI Code using ρ_{max}.
d. Calculate the design moment strength of the section if the steel percentage used is ρ = 1.4%.

3.7 A 10-ft.- (3-m-)span cantilever beam has an effective cross-section (bd) of 12 in. by 24 in. (300 by 600 mm) and is reinforced with five no. 8 (5 × 25 mm) bars. If the uniform load due to its own weight and the dead load are equal to 685 lb/ft (10 kN/m), determine the allowable uniform live load on the beam using the ACI load factors. Given: $f'_c = 3$ ksi (20 MPa) and $f_y = 60$ ksi (400 MPa).

3.8 The cross-section of a 17-ft- (5-m-) span simply supported beam is 10 by 28 in. (250 by 700 mm), and it is reinforced symmetrically with eight no. 6 bars (8 × 20 mm) in two rows. Determine the allowable concentrated live load at midspan considering the total acting dead load (including self-weight) is equal to 2.55 K/ft (37 kN/m). Given: $f'_c = 3$ ksi (20 MPa) and $f_y = 40$ ksi (300 MPa).

3.9 Determine the design moment strength of the sections shown in Fig. 3.41. Neglect the lack of symmetry in (b). Given: $f'_c = 4$ ksi (30 MPa) and $f_y = 60$ ksi (400 MPa).

3.10 A rectangular concrete section has a width of $b = 12$ in. (300 mm), an effective depth of $d = 18$ in. (450 mm), and d′ = 2.5 in. (60 mm). If compression steel consisting of two no. 7 bars (2 × 20 mm) is used, calculate the allowable moment strength that can be applied on the section if the tensile steel, A_s, is as follows:

a. Four no. 7 (4 × 20 mm) bars **b.** Eight no. 7 (8 × 20 mm) bars
Given: $f'_c = 3$ ksi (20 MPa) and $f_y = 40$ ksi (300 MPa).

3.11 A 16-ft- (4.8-m-) span simply supported beam has a width of $b = 12$ in. (300 mm), $d = 22$ in. (500 mm), $d' = 2.5$ in. (60 mm), and A'_s = three no. 6 bars (3 × 20 mm). The beam carries a uniform dead load of 2 K/ft (30 kN/m), including its own weight. Calculate the allowable uniform live load that can be safely applied on the beam. Given: $f'_c = 4$ ksi (20 MPa) and $f_y = 60$ ksi (400 MPa). (Hint: Use ρ_{max} for the basic section to calculate M_u.)

3.12 Check the adequacy of a 10-ft- (3-m-)span cantilever beam, assuming a concrete strength of $f'_c = 4$ ksi (30 MPa) and a steel yield strength of $f_y = 60$ ksi (400 MPa) are used. The dimensions of the beam section are $b = 10$ in. (250 mm), $d = 20$ in. (500 mm), $d' = 2.5$ in. (60 mm), A_s = six no. 7 bars (6 × 20 mm), A'_s = two no. 5 bars (2 × 15 mm). The dead load on the beam, excluding its own weight, is equal to 2 K/ft (30 kN/m), and the live load equals 1.25 K/ft (20 kN/m). (Compare the internal M_u with the external factored moment.)

3.13 A series of reinforced concrete beams spaced at 9 ft (2.7 m) on centers are acting on a simply supported span of 18 ft (5.4 m). The beam supports a reinforced concrete floor slab 4 in. (100 mm) thick. If the width of the web is $b_w = 10$ in. (250 mm), $d = 18$ in. (450 mm), and the beam is reinforced with three

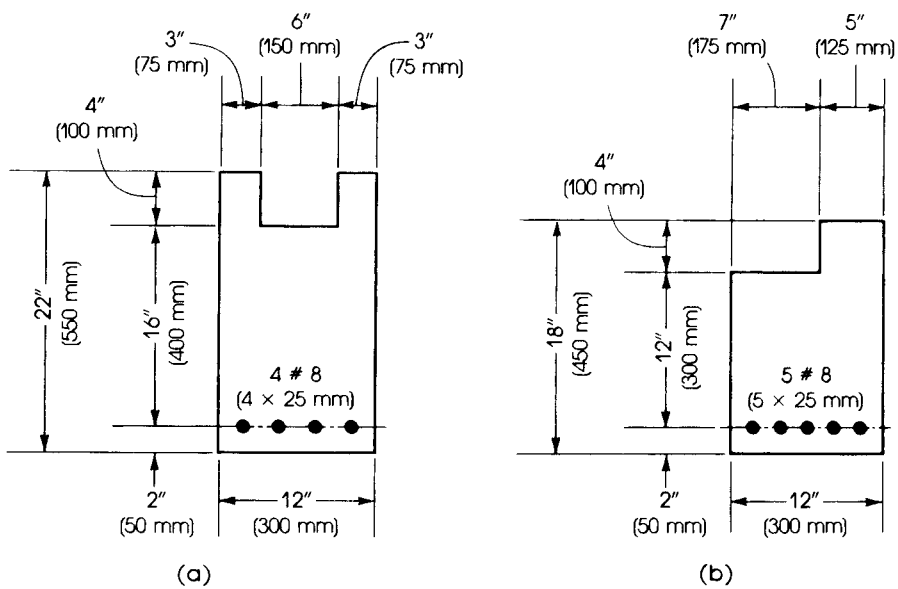


Figure 3.41 Problem 3.9.

no. 9 bars (3 × 30 mm), determine the moment strength of a typical interior beam. Given: $f'_c = 4$ ksi (30 MPa) and $f_y = 60$ ksi (400 MPa).

3.14 Calculate the design moment strength of a T-section that has the following dimensions:

- Flange width = 30 in. (750 mm)
- Flange thickness = 3 in. (75 mm)
- Web width = 10 in. (250 mm)
- Effective depth (d) = 18 in. (450 mm)
- Tension reinforcement: six no. 8 bars (6 × 25 mm)
- $f'_c = 3$ ksi (20 MPa)
- $f_y = 60$ ksi (400 MPa)

3.15 Repeat Problem 3.14 if $d = 24$ in. (600 mm).

3.16 Repeat Problem 3.14 if the flange is an inverted L shape with the same flange width projecting from one side only. (Neglect lack of symmetry.)

CHAPTER 4

FLEXURAL DESIGN OF REINFORCED CONCRETE BEAMS



Reinforced conrete office building, Amman, Jordan.

4.1 INTRODUCTION

In the previous chapter, the analysis of different reinforced concrete sections was explained: Details of the section were given, and we had to determine the design moment of the section. In this chapter, the process is reversed: The external moment is given, and we must find safe, economic, and practical dimensions of the concrete section and the area of reinforcing steel that provide adequate internal moment strength.

4.2 RECTANGULAR SECTIONS WITH REINFORCEMENT ONLY

From the analysis of rectangular singly reinforced sections (Section 3.9), the following equations were derived for tension-controlled sections, where f'_c and f_y are in ksi:

$$\rho_b = 0.85\ \beta_1\ \frac{f'_c}{f_y}\left(\frac{87}{87 + f_y}\right) \tag{3.18}$$

$$\rho_{\max} = \rho_b\left(\frac{0.003 + \dfrac{f_y}{E_s}}{0.008}\right) \tag{3.31}$$

For $f_y = 60$ ksi,

$$\rho_{\max} = 0.63375\rho_b\ (\text{or } 0.634\rho_b) \tag{3.32}$$

Also,

$$\text{for } f_y = 40 \text{ ksi}, \rho_{max} = 0.5474\ \rho_b$$

$$\text{for } f_y = 50 \text{ ksi}, \rho_{max} = 0.5905\ \rho_b$$

$$\text{for } f_y = 75 \text{ ksi}, \rho_{max} = 0.6983\ \rho_b$$

The value of β_1 in ρ_b is 0.85 when $f'_c \leq 4000$ psi (30 N/mm^2) and decreases by 0.05 for every increase of 1000 psi (7 N/mm^2) in concrete strength. The steel percentage of balanced section, ρ_b, and the maximum allowable steel percentage, ρ_{max}, can be calculated for different values of f'_c and f_y, as shown in Table 4.1 or Table A.4 in Appendix A.

It should be clarified that the designer has a wide range of choice between a large concrete section and relatively small percentage of steel, ρ, producing high ductility and a small section with a high percentage of steel with low ductility. A high value of the net tensile strain, ε_t, indicates a high ductility and a relatively low percentage of steel. The limit of the net tensile strain for tension-controlled sections is 0.005, with $\phi = 0.9$. The strain limit of 0.004 can be used with a reduction in ϕ. If the ductility index is represented by the ratio of the net tensile strain, ε_t, to the yield strain, $\varepsilon_y = f_y/E_s$, the relationship between ε_t, ρ/ρ_b, ϕ, and $\varepsilon_t/\varepsilon_y$ is shown in Table 4.2 for $f_y = 60$ ksi. Also, the ACI Code, Section 8.4, indicates that ε_t should be ≥ 0.0075 for the redistribution of moments in continuous flexural members producing a ductility index of 3.75. It can be seen that adopting $\varepsilon_t \geq 0.005$ is preferable to the use of a higher steel ratio, ρ/ρ_b, with $\varepsilon_t = 0.004$, because the increase in M_n is offset by a lower ϕ. The value of $\varepsilon_t = 0.004$ represents the use of minimum steel percentage of 0.00333 for $f'_c = 4$ ksi and $f_y = 60$ ksi. This case should be avoided. The value of ϕ between $\varepsilon_t = 0.005$ and 0.004 can be calculated from Eq. 3.8: $\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right)$.

Table 4.1 Suggested Design Steel Ratios, ρ_s

f'_c (ksi)	f_y (ksi)	% ρ_b	% ρ_{max}	% ρ_s	Ratio ρ_s/ρ_b	Ratio ρ_s/ρ_{max}	R_{us} (psi)	$R_{u\,max}$ (psi)
3	40	3.71	2.031	1.4	0.38	0.69	450	614
	60	2.14	1.356	1.2	0.56	0.89	556	615
4	60	2.85	1.806	1.4	0.49	0.78	662	820
	75	2.07	1.445	1.2	0.58	0.83	702	820
5	60	3.35	2.123	1.6	0.48	0.75	766	975
	75	2.43	1.700	1.4	0.58	0.82	830	975

Table 4.2 Relation Between ε_t, ρ/ρ_b, ϕ, and $\varepsilon_t/\varepsilon_y$ ($f_y = 60$ ksi)

ε_t	0.004	0.005	0.006	0.007	0.0075	0.008	0.009	0.010	0.040
$\frac{\rho}{\rho_b}$	0.714	0.625	0.555	0.500	0.476	0.454	0.417	0.385	0.117
$\frac{\varepsilon_t}{\varepsilon_y}$	2.0	2.5	3.0	3.5	3.75	4.0	4.5	5.0	20
ϕ	0.82	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

The design moment equations were derived in the previous chapter in the following forms:

$$\phi M_n = M_u = R_u b d^2 \tag{3.21}$$

where

$$R_u = \phi \rho f_y \left(1 - \frac{\rho f_y}{1.7 f_c'}\right) = \phi R_n \tag{3.22}$$

where $\phi = 0.9$, for tension-controlled sections and less than 0.9 for sections in the transition region.

$$\phi M_n = M_u = \phi A_s f_y \left(d - \frac{A_s f_y}{1.7 f_c' b}\right) \tag{3.19a}$$

Also,

$$\phi M_n = M_u = \phi \rho f_y b d^2 \left(1 - \frac{\rho f_y}{1.7 f_c'}\right) \tag{3.20}$$

We can see that for a given factored moment and known f_c' and f_y, there are three unknowns in these equations: the width, b, the effective depth of the section, d, and the steel ratio, ρ. A unique solution is not possible unless values of two of these three unknowns are assumed. Usually ρ is assumed (using $\rho_{\max}$, for instance), and b can also be assumed.

Based on the preceding discussion, the following cases may develop for a given M_u, f_c', and f_y:

1. If ρ is assumed, then R_u can be calculated from Eq. 3.22, giving $bd^2 = M_u/R_u$. The ratio of d/b usually varies between 1 and 3, with a practical ratio of 2. Consequently, b and d can be determined, and $A_s = \rho b d$. The ratio ρ for a singly reinforced rectangular section must be equal to or less than $\rho_{\max}$, as given in Eq. 3.31. It is a common practice to assume a value of ρ that ranges between $\frac{1}{2}\rho_{\max}$ and $\frac{1}{2}\rho_b$. Table 4.1 gives suggested values of the steel ratio ρ to be used in singly reinforced sections when ρ is not assigned. For example, if $f_y = 60$ ksi, the value $\rho_s = 1.4\%$ is suggested for $f_c' = 4$ ksi 1.6% for $f_c' = 5$ ksi and 1.2% for $f_c' = 3$ ksi. The designer may use ρ up to $\rho_{\max}$, which produces the minimum size of the singly reinforced concrete section. Using ρ_{min} will produce the maximum concrete section. If b is assumed in addition to ρ, then d can be determined as follows:

$$d = \sqrt{\frac{M_u}{R_u b}} \tag{4.1}$$

If $d/b = 2$, then $d = \sqrt[3]{2M_u/R_u}$ and $b = d/2$, rounded to the nearest higher inch.

2. If b and d are given, then the required reinforcement ratio ρ can be determined by rearranging Eq. 3.20 to obtain

$$\rho = \frac{0.85 f_c'}{f_y}\left[1 - \sqrt{1 - \frac{4M_u}{1.7\phi f_c' b d^2}}\right] \tag{4.2}$$

$$= \frac{0.85f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}}\right] \tag{4.2a}$$

or

$$\rho = \frac{f'_c}{f_y}[0.85 - \sqrt{(0.85)^2 - Q}]$$

where

$$Q = \left(\frac{1.7}{\phi f'_c}\right)\frac{M_u}{bd^2} = \left(\frac{1.7}{\phi f'_c}\right)R_u \tag{4.3}$$

$$A_s = \rho bd = \left(\frac{f'_c}{f_y}\right)bd[0.85 - \sqrt{(0.85)^2 - Q}] \tag{4.4}$$

where all units are in kips (or pounds) and inches, and Q is dimensionless. For example, if $M_u = 2440$ K·in., $b = 12$ in., $d = 18$ in., $f'_c = 3$ ksi, and $f_y = 60$ ksi, then $\rho = 0.01389$ (from Eq. 4.2) and $A_s = \rho bd = 0.01389(12)(18) = 3.0\,\text{in.}^2$, or directly from Eq. 4.4, $Q = 0.395$ and $A_s = 3.0\ \text{in}^2$. When b and d are given, it is better to check if compression steel is or is not required because of a small d. This can be achieved as follows:

a. Calculate $\rho_{\max}$ and $R_u(\max) = \phi\rho_{\max}f_y[1 - (\rho_{\max}f_y/1.7f'_c)]$.

b. Calculate $\phi M_n(max) = R_u bd^2 =$ the design moment strength of a singly reinforced concrete section.

c. If $M_u < \phi M_{n\max}$, then no compression reinforcement is needed. Calculate ρ and A_s from the preceding equations.

d. $M_u > \phi M_{n\max}$, then compression steel is needed. In this case, the design procedure is explained in Section 4.4.

3. If ρ and b are given, calculate R_u:

$$R_u = \phi\rho f_y\left(1 - \frac{\rho f_y}{1.7f'_c}\right)$$

Then calculate d from Eq. 4.1:

$$d = \sqrt{\frac{M_u}{R_u b}} \quad \text{and} \quad A_s = \rho bd$$

4.3 SPACING OF REINFORCEMENT AND CONCRETE COVER

4.3.1 Specifications

Figure 4.1 shows two reinforced concrete sections. The bars are placed such that the clear spacings shall be at least equal to nominal bar diameter D but not less than 1 in. (25 mm). Vertical clear spacings between bars in more than one layer shall not be less than 1 in. (25 mm), according to the ACI Code, Section 7.6.

The width of the section depends on the number, n, and diameter of bars used. Stirrups are placed at intervals; their diameters and spacings depend on shear requirements, to be explained

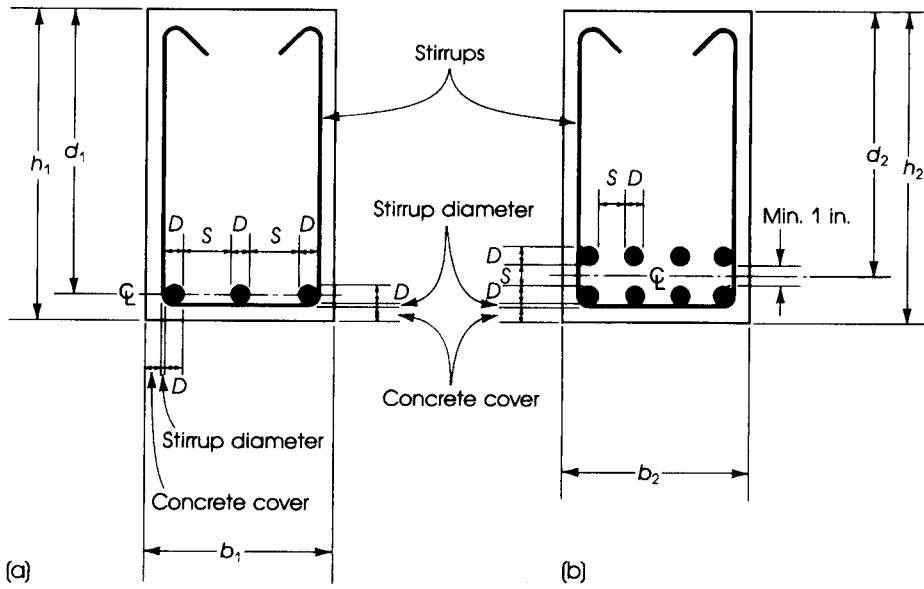


Figure 4.1 Spacing of steel bars (a) in one row or (b) two rows.

later. At this stage, stirrups of $\frac{3}{8}$ in. (10 mm) diameter can be assumed to calculate the width of the section. There is no need to adjust the width, b, if different diameters of stirrups are used. The specified concrete cover for cast-in-place and precast concrete is given in the ACI Code, Section 7.7. Concrete cover for beams and girders is equal to $\frac{3}{2}$ in. (38 mm), and that for slabs is equal to $\frac{3}{4}$ in. (20 mm), when concrete is not exposed to weather or in contact with ground.

4.3.2 Minimum Width of Concrete Sections

The general equation for the minimum width of a concrete section can be written in the following form:

$$b_{\min} = nD + (n-1)s + 2(\text{stirrup's diameter}) + 2(\text{concrete cover}) \tag{4.5a}$$

where

n = number of bars
D = diameter of the largest bar used
s = spacing between bars (equal to D or 1 in., whichever is larger)

If the stirrup's diameter is taken equal to $\frac{3}{8}$ in. (10 mm) and concrete cover equals $\frac{3}{2}$ in. (38 mm), then

$$b_{\min} = nD + (n-1)s + 3.75 \text{ in. (95 mm)} \tag{4.5b}$$

This equation, if applied to the concrete sections in Fig. 4.1, becomes

$$b_1 = 3D + 2S + 3.75 \text{ in. (95 mm)}$$

$$b_2 = 4D + 3S + 3.75 \text{ in. (95 mm)}$$

To clarify the use of Eq. 4.5, let the bars used in sections of Fig. 4.1 be no. 10 (32-mm) bars. Then

$$b_1 = 5 \times 1.27 + 3.75 = 10.10 \text{ in. } (s = D) \quad \text{say, 11 in.}$$
$$b_1 = 5 \times 32 + 95 = 225 \text{ mm} \quad \text{say, 250 mm}$$
$$b_2 = 7 \times 1.27 + 3.75 = 12.64 \text{ in.} \quad \text{say, 13 in.}$$
$$b_1 = 7 \times 232 + 95 = 319 \text{ mm} \quad \text{say, 320 mm}$$

If the bars used are no. 6 (20 mm), the minimum widths become

$$b_1 = 3 \times 0.75 + 2 \times 1 + 3.75 = 8.0 \text{ in.} \quad s = 1 \text{ in.}$$
$$b_1 = 3 \times 20 + 2 \times 25 + 95 = 205 \text{ mm} \quad \text{say, 210 mm}$$
$$b_2 = 4 \times 0.75 + 3 \times 1 + 3.75 = 9.75 \text{ in.} \quad \text{say, 10 in.}$$
$$b_2 = 4 \times 20 + 3 \times 25 + 95 = 250 \text{ mm}$$

The width of the concrete section shall be increased to the nearest inch. Table A.7 in Appendix A gives the minimum beam width for different numbers of bars in the section.

4.3.3 Minimum Overall Depth of Concrete Sections

The effective depth, d, is the distance between the extreme compressive fibers of the concrete section and the centroid of the tension reinforcement. The minimum total depth is equal to d plus the distance from the centroid of the tension reinforcement to the extreme tension concrete fibers, which depends on the number of layers of the steel bars. In application to the sections shown in Fig. 4.1,

$$h_1 = d_1 + \frac{D}{2} + \frac{3}{8} \text{ in.} + \text{concrete cover} \tag{4.6a}$$
$$= d_1 + \frac{D}{2} + 1.857 \text{ in. } (50 \text{ mm})$$

for one row of steel bars and

$$h_2 = d_2 = 0.5 + D + \frac{3}{8} \text{ in.} + \text{concrete cover} \tag{4.6b}$$
$$= d_2 + D + 2.375 \text{ in. } (60 \text{ mm})$$

for two layers of steel bars. The overall depth, h, shall be increased to the nearest half inch (10 mm) or, better, to the nearest inch (20 mm in SI). For example, if $D = 1$ in. (25 mm), $d_1 = 18.9$ in. (475 mm), and $d_2 = 20.1$ in. (502 mm),

$$\text{Minimum } h_1 = 18.9 + 0.5 + 1.875 = 21.275 \text{ in.}$$

say, 21.5 in. or 22 in.,

$$h_1 = 475 + 13 + 50 = 538 \text{ mm}$$

say, 540 mm or 550 mm, and

$$\text{Minimum } h_2 = 20.1 + 1.0 + 2.375 = 23.475 \text{ in.}$$

say, 23.5 in. or 24 in.,

$$h_2 = 502 + 25 + 60 = 587 \text{ mm}$$

say, 590 mm or 600 mm.

If no. 9 or smaller bars are used, a practical estimate of the total depth, h, can be made as follows:

$$h = d + 2.5 \text{ in. (65 mm), for one layer of steel bars}$$

$$h = d + 3.5 \text{ in. (90 mm), for two layers of steel bars}$$

For more than two layers of steel bars, a similar approach may be used.

It should be mentioned that the minimum spacing between bars depends on the maximum size of the coarse aggregate used in concrete. The nominal maximum size of the coarse aggregate shall not be larger than one-fifth of the narrowest dimension between sides of forms, nor one-third of the depth of slabs, nor three-fourths of the minimum clear spacing between individual reinforcing bars or bundles of bars (ACI Code, Section 3.3).

Example 4.1

Design a simply reinforced rectangular section to resist a factored moment of 361 K-ft using the maximum steel percentage ρ_{max} for tension-controlled sections. Given: $f'_c = 3$ ksi and $f_y = 60$ ksi.

Solution

For $f'_c = 3$ ksi, $f_y = 60$ ksi, and $\beta_1 = 0.85$, ρ_{max} for a tension-controlled section is calculated as follows ($\phi = 0.9$):

$$\rho_b = (0.85)\beta_1 \left(\frac{f'_c}{f_y}\right)\left[\frac{87}{(87 + f_y)}\right],$$

$$\rho_b = (0.85)^2 \left(\frac{3}{60}\right)\left(\frac{87}{147}\right) = 0.0214$$

$$\rho_{max} = \rho_b \left(\frac{0.003 + \frac{f_y}{E_s}}{0.008}\right) = 0.63375\rho_b = 0.01356 \qquad \text{(Table 4.1)}$$

$$R_{u\,max} = \phi\rho_{max} f_y \left(1 - \frac{\rho_{max} f_y}{1.7 f'_c}\right)$$

$$= 0.9 \times 0.01356 \times 60 \times \left(1 - \frac{0.01356 \times 60}{1.7 \times 3}\right) = 0.615 \text{ ksi}$$

(Or, use the tables in Appendix A or Table 4.1.)

Since $M_u = R_u bd^2$,

$$bd^2 = \frac{M_u}{R_u} = \left(\frac{361 \times 12}{0.615}\right) = \frac{4332}{0.615} = 7043 \text{ in.}^3$$

Thus, for the following assumed b, calculate d and $A_s = \rho bd$:

$b = 10$ in.	$d = 26.5$ in.	$A_s = 4.24$ in.^{2}	
$b = 12$ in.	$d = 24.2$ in.	$A_s = 4.65$ in.^{2}	6 no. 8 bars ($A_s = 4.71$ in.^{2})
$b = 14$ in.	$d = 22.4$ in.	$A_s = 5.01$ in.^{2}	5 no. 9 bars ($A_s = 5.0$ in.^{2})
$b = 16$ in.	$d = 21.0$ in.	$A_s = 5.37$ in.^{2}	

The choice of the effective depth d depends on three factors:

1. The width b required. A small width will result in a deep beam that decreases the headroom available. Furthermore, a deep narrow beam may lower the design moment strength of the structural member due to possible lateral deformation.
2. The amount and distribution of reinforcing steel. A narrow beam may need more than one row of steel bars, thus increasing the total depth of the section.
3. The wall thickness. If cement block walls are used, the width b is chosen to be equal to the wall thickness. Exterior walls in buildings in most cases are thicker than interior walls. The architectural plan of the structure will show the different thicknesses.

A reasonable choice of d/b varies between 1 and 3, with practical value about 2. It can be seen from the previous calculations that the deeper the section, the more economical it is, as far as the quantity of concrete used, expressed by the area bd of a 1-ft length of the beam. Alternatively, calculate $bd^2 = M_u/R_u$ and then choose adequate b and d.

The area of the steel reinforcement, A_s, is equal to ρbd. The area of steel needed for the different choices of b and d for this example was shown earlier. Because the steel percentage required is constant ($\rho_{max} = 0.01356$), A_s is proportional to bd. For a choice of a 12 × 24.2-in. section, the required A_s is 4.65 in.^{2} Choose six no. 8 bars in two rows (actual $A_s = 4.71$ in.^{2}). The minimum b required for three no. 8 bars in one row is 8.9 in. < 12 in., and total $h = 24.2 + 3.5 = 27.7$ in., say, 28 in. (actual $d = 24.6$ in.). Another choice is a section with a 14 × 22.4-in. section with a total depth (h) of 25 in. and five no. 9 bars in one row. The choice of bars depends on

1. Adequate placement of bars in the section, normally in one or two rows, fulfilling the restrictions of the ACI Code for minimum spacing between bars
2. The area of steel bars chosen closest to the required calculated steel area

The final section is shown in Fig. 4.2.

Example 4.2

Solve Example 4.1 using a steel percentage ρ of about 1% and $b = 14$ in.

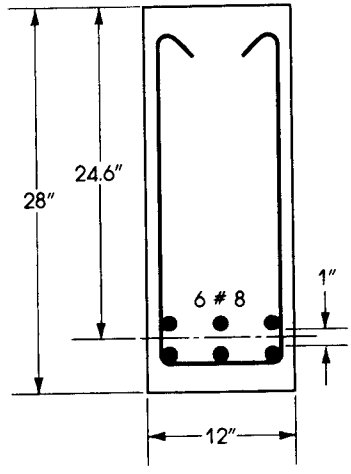


Figure 4.2 Example 4.1.

Solution

1. For $f'_c = 3$ ksi and $f_y = 60$ ksi, $\rho_{max} = 0.01356$ for a tension-controlled section:

$$R_u = \phi\rho f_y\left(1 - \frac{\rho f_y}{1.7f'_c}\right)$$

$$= 0.9 \times 0.01 \times 60\left(1 - \frac{0.01 \times 60}{1.7 \times 3}\right) = 0.476 \text{ ksi}$$

(From the tables in Appendix A, for $\rho = 0.01$, $R_u = 476$ psi.)

2. $bd^2 = M_u/R_u = 4332/0.476 = 9100\,\text{in.}^3$. Choosing $b = 14$ in. and $d = 25.5$ in.,

$$A_s = \rho bd = 0.01 \times 14 \times 25.5 = 3.57 \text{ in.}^2$$

Choose four no. 9 bars in one layer; $A_s = 4.00\,\text{in.}^2$

$$b_{min} = nD + (n-1)s + 3.75$$

$$= 7 \times 1.128 + 3.75 = 11.7 \text{ in.} < 14 \text{ in.}$$

$$h_{min} = d + \frac{D}{2} + 1.875$$

$$= 25.5 + \frac{1.138}{2} + 1.875 = 27.94 \text{ in.} \qquad \text{say, 28 in.} \qquad (d = 25.5 \text{ in.})$$

3. Because the actual A_s used is greater than the calculated A_s, a smaller depth can be adopted. Therefore, take $h = 26$ in. Then $d = 26 - 1.138/2 - 1.875 = 23.5$ in.

For small variation in depth, $A_s = 3.57(25.5/23.5) = 3.87\,\text{in.}^2$, which is less than the $4.00\,\text{in.}^2$ used (Fig. 4.3). A check of the design moment strength of the section can be made:

$$\text{actual } \rho = \frac{4}{14 \times 23.5} = 0.0121$$

Since $\rho < \rho_{max} = 0.01356$ for a tension-controlled section ($\phi = 0.9$),

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4.0 \times 60}{0.85 \times 3 \times 14} = 6.72 \text{ in.}$$

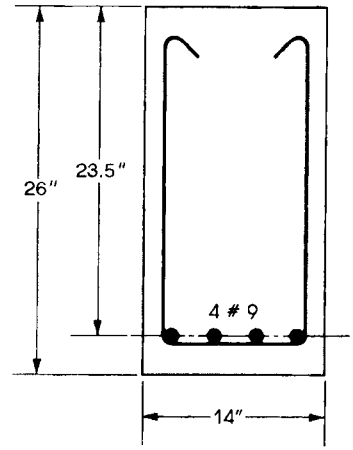


Figure 4.3 Example 4.2.

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$= 0.9 \times 4 \times 60 \left(23.5 - \frac{6.72}{2}\right) = 4350 \text{ K·in.} > 4332 \text{ K·in.}$$

which is acceptable.

4. Check the net tensile strain, ε_t. For $f_y = 60$ ksi,

$$\varepsilon_t = \left(\frac{0.005}{\frac{\rho}{\rho_b}}\right) - 0.003 \tag{3.25}$$

$$\rho_b = 0.0214 \qquad \text{(Table 4.1)}$$

$$\frac{\rho}{\rho_b} = \frac{0.0121}{0.0214} = 0.5654$$

$$\varepsilon_t = \frac{0.005}{0.5654} - 0.003 = 0.00584 > 0.005 \qquad \text{(tension-controlled section)}$$

Or, alternatively, $c = a/0.85 = 7.9$ in., $d_t = 26 - 2.5 = 23.5$ in., $c/d_t = 0.336 < 0.375$, which is o.k.

Example 4.3

Find the necessary reinforcement for a given section that has a width of 10 in. and a total depth of 20 in. (Fig. 4.4) if it is subjected to an external factored moment of 163 K·ft. Given: $f'_c = 4$ ksi and $f_y = 60$ ksi.

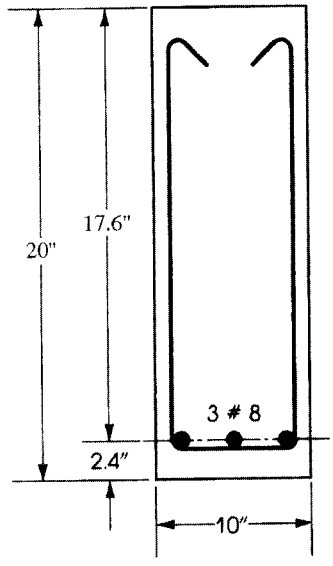


Figure 4.4 Example 4.3.

Solution

1. Assuming one layer of no. 8 steel bars (to be checked later), $d = 20 - 0.5 - 1.875 = 17.625$ in. (or $d = 20 - 2.5$ in. $= 17.5$ in.).
2. Check if the section is adequate without compression reinforcement. Compare the moment strength of the section (using ρ_{max} for tension-controlled condition). For $f'_c = 4$ ksi and $f_y = 60$ ksi, $\rho_{max} = 0.01806$.

$$R_{u\,max} = \phi\rho_{max} f_y \left(1 - \frac{\rho_{max} f_y}{1.7 f'_c}\right) = 820 \text{ psi} \qquad \text{(from Table 4.1)}$$

The moment strength of a singly reinforced basic section is

$$\phi M_{n\,max} = R_{u\,max} bd^2 = 0.82(10)(17.5)^2$$
$$= 2511 \text{ K·in.} > 163 \times 12 = 1956 \text{ K·in.}$$

Therefore, $\rho < \rho_{max}$ and the section is singly reinforced, and tension controls ($\phi = 0.9$).

3. Calculate ρ from Eq 4.2 or 4.3:

$$Q = \left(\frac{1.7}{\phi f'_c}\right) \times \frac{M_u}{bd^2} = \left(\frac{1.7}{0.9 \times 4}\right) \times \left(\frac{1956}{10 \times 17.5^2}\right) = 0.302$$

$$\rho = \frac{f'_c}{f_y}(0.85 - \sqrt{(0.85)^2 - Q}) = 0.0134 < \rho_{max} \qquad \text{(tension-controlled condition)}$$

$A_s = \rho bd = 0.0134(10)(17.5) = 2.345$ in.^{2} Use three no. 8 bars ($A_s = 2.35$ in.^{2}) in one row, $b_{min} < 10$ in. The final section is shown in Fig. 4.4.

Example 4.4

Find the necessary reinforcement for a given section, $b = 15$ in., if it is subjected to a factored moment of 313 K·ft. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. For $f'_c = 4$ ksi and $f_y = 60$ ksi, and from Table 4.1: $\rho_b = 0.0285$, $\rho_{max} = 0.01806$ (tension-controlled section), $R_{u\,max} = 820$ psi.
2. Using $\rho_{max} = 0.01806$ and $R_u = 820$ psi,

$$bd^2 = \frac{M_u}{R_u} = \frac{313(12)}{0.820} = 4581 \text{ in.}^3$$

For $b = 15$ in. and $d = 17.50$,

$$A_s = \rho bd = 0.01806(15)(17.5) = 4.74 \text{ in.}^2$$

Choose four no. 10 bars, $A_s = 5.08$ in.$^2 > 4.74$ in.^{2}. Bars can be placed in one row, $b_{min} = 12.7$ in. in Table A.7. Total depth (h) $= 17.5 + 2.5 = 20$ in.

Discussion

1. Since a steel area of 5.08 in.^{2} used is greater than 4.74 in.^{2} required (the limit for a tension-controlled section with $\phi = 0.9$), the section is in the transition zone. Actually, the section is under-reinforced and the nominal moment $= M_n = A_s f_y(d - a/2) = 368.6$ K·ft. ($A_s = 5.08$ in.^{2} and $a = 5.976$ in.). If $\phi = 0.9$ is used then $\phi M_n = 331.7$ K·ft.

2. The ACI Code indicates that for sections in the transition zone, $\phi < 0.9$, and $\varepsilon_t \geq 0.004$.

Checking $\varepsilon_t = \left(\frac{0.005}{\frac{\rho}{\rho_b}}\right) - 0.003$,

$$\rho = \frac{5.08}{15 \times 17.5} = 0.01935 \qquad \frac{\rho}{\rho_b} = 0.679$$

$$\varepsilon_t = \left(\frac{0.00507}{0.679}\right) - 0.003 = 0.004467 > 0.004$$

Or, alternatively, calculate $a = 5.08 \times 60/(0.85 \times 4 \times 15) = 5.976$, $c = a/.85 = 7.03$, $d_t = d = 17.5$ in. Then $\varepsilon_t = 0.003(d_t - c)/c = 0.004467$. Calculate

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) = 0.856$$

$$\phi M_n = 0.856(368.6) = 315.4 \text{ K·ft}$$

3. It can be noticed that despite an additional amount of steel, $5.08 - 4.67 = 0.41$ in.^{2} (or about 9%), the design moment strength remained the same. This is because the strength reduction factor, ϕ, was decreased. Therefore, the design of sections within the tension-controlled zone with $\phi = 0.9$ gives a more economical design based on the ACI Code limitations.

4.4 RECTANGULAR SECTIONS WITH COMPRESSION REINFORCEMENT

A singly reinforced section has its moment strength when ρ_{max} of steel is used. If the applied factored moment is greater than the internal moment strength, as in the case of a limited cross-section, a doubly reinforced section may be used, adding steel bars in both the compression and the tension zones. Compression steel will provide compressive force in addition to the compressive force in the concrete area.

4.4.1 Assuming One Row of Tension Bars

The procedure for designing a rectangular section with compression steel when M_u, f'_c, b, d, and d' are given can be summarized as follows:

1. Calculate the balanced and the maximum steel ratio, ρ_{max}, using Eqs. 3.18 and 3.31.

$$\rho_b = 0.85\beta_1 \frac{f'_c}{f_y}\left(\frac{87}{87 + f_y}\right)$$

Calculate $A_{s\,max} = A_{s1} = \rho_{max} bd$ (maximum steel area as singly reinforced).

2. Calculate $R_{u\,max}$ using ρ_{max} ($\phi = 0.9$):

$$R_{u\,max} = \phi\rho_{max} f_y\left(1 - \frac{\rho_{max} f_y}{1.7 f'_c}\right)$$

($R_{u\,max}$ can be obtained from the tables in Appendix A or Table 4.1.)

3. Calculate the moment strength of the section, M_{u1}, as singly reinforced, using ρ_{max} and $R_{u\,max}$.

$$M_{u1} = R_{u\,max} bd^2$$

If $M_{u1} < M_u$ (the applied moment), then compression steel is needed. Go to the next step. If $M_{u1} > M_u$, then compression steel is not needed. Use Eq. 4.2 to calculate ρ and $A_s = \rho bd$, as explained earlier.

4. Calculate $M_{u2} = M_u - M_{u1}$ = the moment to be resisted by compression steel.
5. Calculate A_{s2} from $M_{u2} = \phi A_{s2} f_y (d - d')$.
   Then calculate the total tension reinforcement, A_s:
$$A_s = A_{s1} + A_{s2}$$
6. Calculate the stress in the compression steel as follows:
   a. Calculate $f'_s = 87[(c - d')/c]$ ksi $\leq f_y$. (f'_s cannot exceed f_y.)
   b. Or, ε'_s can be calculated from the strain diagram, and $f'_s = (\varepsilon'_s \cdot E_s)$. If $\varepsilon'_s \geq \varepsilon_y$, then compression steel yields and $f'_s = f_y$.
   c. Calculate A'_s from $M_{u2} = \phi A'_s f'_s (d - d')$. If $f'_s = f_y$, then $A'_s = A_{s2}$. If $f'_s < f_y$, then $A'_s > A_{s2}$, and $A'_s = A_{s2}(f_y/f'_s)$.
7. Choose bars for A_s and A'_s to fit within the section width, b. In most cases, A_s bars will be placed in two rows, whereas A'_s bars are placed in one row.
8. Calculate $h = d + 2.5$ in. for one row of tension bars and $h = d + 3.5$ in. for two rows of tension steel. Round h to the next higher inch. Now check that $\lfloor \rho - \rho\prime(f'_s/f_y) \rfloor < \rho_{\max}$ using the new d, or check that $A_{s\max} = bd\lfloor \rho_{\max} + \rho'(f'_s/f_y) \rfloor \geq A_s$ (used).
$$\rho = \frac{A_s}{(bd)} \quad \text{and} \quad \rho\prime = \frac{A'_s}{(bd)}$$
   This check may not be needed if $\rho_{\max}$ is used in the basic section.
9. If desired, the design moment strength of the final section, ϕM_n, can be calculated and compared with the applied moment, M_u: $\phi M_n \geq M_u$. Note that a steel ratio ρ smaller than $\rho_{\max}$ can be assumed in step 1, say $\rho = 0.6\rho_b$ or $\rho = 0.9\rho_{\max}$, so that the final tension bars can be chosen to meet the given $\rho_{\max}$ limitation.
10. The strain at the bars level can be checked as follows:
$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003 \geq 0.005$$

4.4.2 Assuming Two Rows of Tension Bars

In the case of two rows of bars, it can be assumed that $d = h - 3.5$ in. and $d_t = h - 2.5$ in. $= d + 1.0$ in.

Two approaches may be used to design the section.

1. One approach is to assume a strain at the level of the centroid of the tension steel equal to 0.005 or $\varepsilon_s = 0.005$ (at d level). In this case, the strain in the lower row of bars is greater than 0.005. $\varepsilon_t = (d_t - c/c)0.003 > 0.005$, which still meets the ACI Code limitation. For this case, follow the above steps 1 to 9. Example 4.6, solution 1 explains this approach.
2. A second approach is to assume a strain $\varepsilon_t = 0.005$ at the level of the lower row of bars, d_t. In this case, the stain at the level of the centroid of bars is less than 0.005: $\varepsilon_s = [(d_t - c)/c]0.003 < 0.005$, which is still acceptable. Example 4.6, solution 2 explains this approach. The solution can be summarized as follows:

a. Calculate $d_t = h - 2.5$ in., and then form the strain diagram and calculate c, the depth of the neutral axis

$$c = \left(\frac{0.003}{0.003 + \varepsilon_t}\right) d_t$$

For $\varepsilon_t = 0.005$,

$$c = \left(\frac{3}{8}\right) d_t \text{ and } a = \beta_1 c$$

b. Calculate the compression force in the concrete.

$$C_1 = 0.85 f'_c ab = T_1 = A_{s1} f_y$$

Determine A_{s1}. Calculate $M_{u1} = \phi A_{s1} f_y (d - a/2)$. $\rho_1 = A_{s1}/bd$, $\phi = 0.9$.

c. Calculate $M_{u2} = M_u - M_{u1}$; assume $d' = 2.5$ in.

d. Calculate A_{s2}: $M_{u2} = \phi A_{s2} f_y (d - d')$, $f'_c = f_y$, $\phi = 0.9$. Total $A_s = A_{s1} + A_{s2}$.

e. Check if compression steel yields similar to step 6 above in section 4.4.1.

Example 4.5

A beam section is limited to a width of $b = 10$ in. and a total depth of $h = 22$ in. and has to resist a factored moment of 226.5 K·ft. Calculate the required reinforcement. Given: $f'_c = 3$ ksi and $f_y = 50$ ksi.

Solution

1. Determine the design moment strength that is allowed for the section as singly reinforced based on tension-control conditions. This is done by starting with ρ_{max}. For $f'_c = 3$ ksi and $f_y = 50$ ksi, and from Eqs. 3.18, 3.22, and 3.31,

$$\rho_b = 0.0275 \qquad \rho_{max} = 0.01624 \qquad R_u = 614 \text{ psi}$$

$$M_u = R_u bd^2 \qquad b = 10 \text{ in.} \qquad d = 22 - 3.5 = 18.5 \text{ in.}$$

$$M_u = 226.5 \times 12 = 2718 \text{ K·in.}$$

(This calculation assumes two rows of steel, to be checked later.) $M_{u1} = 0.614 \times 10 \times (18.5)^2 = 2101$ K·in. = max ϕM_n, as singly reinforced. Design $M_u = 2718$ K·in. > 2101 K·in. Therefore, compression steel is needed to carry the difference.

2. Compute A_{s1}, M_{u1}, and M_{u2}:

$$A_{s1} = \rho_{max} bd = 0.01624 \times 10 \times 18.5 = 3.0 \text{ in.}^2$$

$$M_{u1} = 2101 \text{ K·in.}$$

$$M_{u2} = M_u - M_{u1} = 2718 - 2102 = 617 \text{ K·in.}$$

3. Calculate A_{s2} and A'_s, the additional tension and compression steel due to M_{u2}. Assume $d' = 2.5$ in.; $M_{u2} = \phi\ A_{s2} f_y\ (d - d')$.

$$A_{s2} = \frac{M_{u2}}{\phi f_y (d - d')} = \frac{617}{0.9 \times 50(18.5 - 2.5)} = 0.86 \text{ in.}^2$$

Total tension steel is equal to A_s.

$$A_s = A_{s1} + A_{s2} = 3.0 + 0.86 = 3.86 \text{ in.}^2$$

The compression steel has $A'_s = 0.86$ in.^{2} (in A'_s yields).

4. Check if compression steel yields:

$$\varepsilon_y = \frac{f_y}{29{,}000} = \frac{50}{29{,}000} = 0.00172$$

Let $a = (A_{s1} f_y)/(0.85 f'_c b) = (3.0 \times 50)/(0.85 \times 3 \times 10) = 5.88$ in.

$$c(\text{distance to neutral axis}) = \frac{a}{\beta_1} = \frac{5.88}{0.85} = 6.92 \text{ in.}$$

ε'_s = strain in compression steel (from strain triangles)

$$= 0.003 \times \left(\frac{5.88 - 2.5}{5.88}\right) = 0.00173 > \varepsilon_y = 0.001724$$

5. Check ε_t:

$$\rho_1 = \frac{3}{10 \times 18.5} = 0.016216$$

$$\frac{\rho_1}{\rho_b} = 0.5897 \qquad f_y = 50$$

From Eq. 3.24, $\varepsilon_{t_s} = 0.005$ is assumed at the centroid of the tension steel for $\rho_{\max}$ and R_u used. Calculate ε_t (at the lower row of bars):

$$d_t = 22 - 2.5 = 19.5 \text{ in.}$$

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003$$

$$= \left(\frac{19.5 - 6.92}{6.92}\right) 0.003$$

$$= 0.00545 > 0.005$$

as expected.

6. Choose steel bars as follows: $A_s = 3.86\,\text{in.}^2$ Choose five no. 8 bars ($A_s = 3.95\,\text{in.}^2$) in two rows, as assumed. $A'_s = 0.86$ in.^{2} Choose two no. 6 bars ($A'_s = 0.88$ in.^{2}).
7. Check actual d: Actual $d = 22 - (1.5 + 0.375 + 1.5) = 18.625$ in. It is equal approximately to the assumed depth. The final section is shown in Fig. 4.5.

Example 4.6

A beam section is limited to $b = 12$ in. and to a total depth of $h = 20$ in. and is subjected to a factored moment $M_u = 298.4$ K·ft. Determine the necessary reinforcement using $f'_c = 4$ ksi and $f_y = 60$ ksi. (Refer to Fig. 4.6.)

Solution 1: Two Solutions Are Presented

1. Determine the maximum moment capacity of the section as singly reinforced based on tension-controlled conditions. For $f'_c = 4$ ksi and $f_y = 60$ ksi, $\rho_{\max} = 0.01806$ and $R_u = 820$ psi (Table 4.1). Assuming two rows of bars, $d = 20 - 3.5 = 16.5$ in.

$$\text{Max } M_{u1} = R_{u\max} b d^2 = 0.82(12)(16.5)^2 = 2679 \text{ K·in.} = 223.25 \text{ K·ft.}$$

The design moment is $M_u = 298.4 \times 12 = 3581$ K·in. $> M_{u1}$; therefore, compression steel is needed.

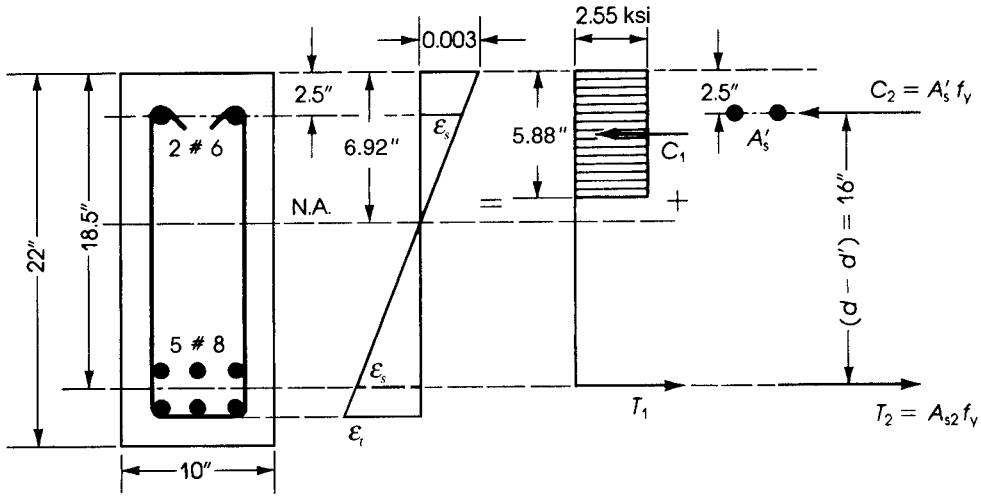


Figure 4.5 Example 4.5: doubly reinforced concrete section.

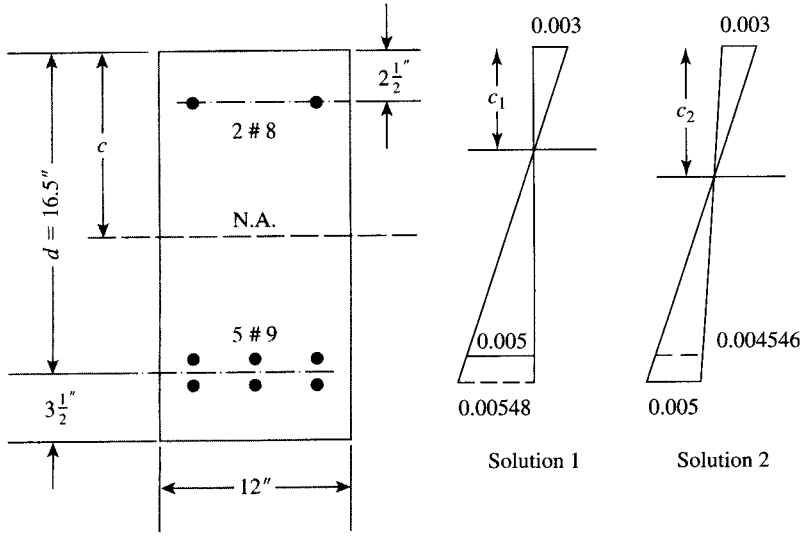


Figure 4.6 Example 4.6.

2. Calculate A_{s1}, M_{u2}, A_{s2}, and A_s.

$$A_{s1} = \rho_{\max} bd = 0.01806(12)(16.5) = 3.576 \text{ in.}^2$$

$$M_{u2} = M_u - M_{u1} = 3581 - 2679 = 902 \text{ K·in.}$$

$$M_{u2} = \phi A_{s2} f_y (d - d'), \text{ assume } d' = 2.5 \text{ in.}$$

$$902 = 0.9A_{s2}(60)(16.5 - 2.5), A_{s2} = 1.19 \text{ in.}^2$$

$$\text{Total } A_s = A_{s1} + A_{s2} = 3.576 + 1.19 = 4.77 \text{ in.}^2 \quad \text{(five no. 9 bars)}$$

3. Check if compression steel yields by Eq. 3.46. Compression steel yields if

$$\rho - \rho' \geq K = 0.85\beta_1 \frac{f'_c}{f_y}\left(\frac{d'}{d}\right)\left(\frac{87}{87 - f_y}\right)$$

$$K = (0.85)^2\left(\frac{4}{60}\right)\left(\frac{2.5}{16.5}\right)\left(\frac{87}{27}\right) = 0.0235$$

$$\rho - \rho' = \frac{A_{s1}}{bd} = \frac{3.576}{(12)(16.5)} = 0.01806 \leq K$$

Therefore, compression steel does not yield: $f'_s < f_y$

4. Calculate f'_s : $f'_s = 87[(c - d')/c] \leq f_y$. Determine c from A_{s1}: $A_{s1} = 3.576\,\text{in.}^2$,

$$a = \frac{A_{s1} f_y}{0.85 f'_c b} = \frac{3.576 \times 60}{0.85 \times 4 \times 12} = 5.26 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{5.26}{0.85} = 6.19 \text{ in.}$$

$$f'_s = 87 \times \left(\frac{6.19 - 2.5}{6.19}\right) = 51.8 \text{ ksi} < 60 \text{ ksi}$$

5. Calculate A'_s from $M_{u2} = \phi A'_s f'_s (d - d')$:

$$902 = 0.9A'_s(51.8)(16.5 - 2.5)$$

Thus, $A'_s = 1.38\,\text{in.}^2$, or calculate A'_s from $A'_s = A_{s2}(f_y/f'_s) = 1.38\,\text{in.}^2$ (two no. 8 bars). Note that the condition $[\rho - \rho'(f'_s/f_y)] = (\rho - \rho') \leq \rho_{\max}$ is already met.

$$\left(\rho - \rho'\frac{f'_s}{f_y}\right) = \frac{1}{bd}(A_s - A_{s2}) = \frac{3.576}{12 \times 16.5} = 0.01806$$

as assumed in the solution.

6. These calculations using $\rho_{\max}$ and R_u are based on a strain of 0.005 at the centroid of the tension steel.

$$\varepsilon_t(\text{at bottom row}) = \left(\frac{d_t - c}{c}\right)0.003$$

$$d_t = 20 - 2.5 = 17.5 \text{ in.} \qquad \varepsilon_t = \left(\frac{17.5 - 6.19}{6.19}\right)0.003 = 0.00548 > 0.005$$

as expected.

Solution 2

Assuming two rows of tension bars and a strain at the lower row, $\varepsilon_t = 0.005$, the solution will be as follows:

1. Calculate $d_t = 20 - 2.5 = 17.5\,\text{in.}$ From the strain diagram:

$$\frac{c}{d_t} = \frac{0.003}{0.003 + \varepsilon_t} = \frac{0.003}{0.008} = 0.375$$

$$c = 0.375(17.5) = 6.5625 \text{ in.} \qquad a = 0.85c = 5.578 \text{ in.}$$

2. The compression force in the concrete $= C_1 = 0.85\, f'_c ab$

$$C_1 = 0.85(4)(5.578)(12) = 227.6 \text{ K} = T_1 \text{ (as singly reinforced)}$$

$$A_{s1} = \frac{C_1}{f_y} = \frac{T_1}{f_y} = \frac{227.6}{60} = 3.793 \text{ in.}^2$$

$$d = 20 - 3.5 = 16.5 \text{ in.}$$

$$M_{u1} = \phi A_{s1} f_y \left(d - \frac{a}{2}\right) = 0.9(3.793)(60)\left(16.5 - \frac{5.578}{2}\right) = 2808 \text{ K·in.}$$

$$= 234 \text{ K} \cdot \text{ft}$$

$$R_{u1} = \frac{M_{u1}}{bd^2} = \frac{2808.3}{12(16.5)^2} = 0.86 \text{ ksi} = 860 \text{ psi}$$

$$\rho_1 = \frac{A_{s1}}{bd} = 0.01916$$

3. Since $M_u = 3581$ K·in. $> M_{u1}$, compression steel is needed.

$$M_{u2} = 3581 - 2808 = 773 \text{ K·in.}$$

$$M_{u2} = 0.9 A_{s2} f_y (d - d')$$

$$773 = 0.9 A_{s2}(60)(16.5 - 2.5) \qquad A_{s2} = 1.022 \text{ in.}^2$$

$$\text{Total } A_s = A_{s1} + A_{s2} = 3.793 + 1.022 = 4.815 \text{ in.}^2$$

Use five no. 9 bars.

4. Check if compression steel yields as in step 3 in the first solution.

$$K = 0.0235(\rho - \rho') = \rho_1 = 0.01916 < K$$

Compression steel does not yield.

$$f'_s = 87\left(\frac{c - d'}{c}\right) = \left(\frac{6.56 - 2.5}{6.56}\right)87 = 53.84 \text{ ksi}$$

Calculate A_{s2}:

$$M_{u2} = \phi A'_s f'_s (d - d')$$

$$773 = 0.9 A'_s (53.84)(16.5 - 2.5) \qquad A'_s = 1.14 \text{ in.}^2$$

Use two no. 7 bars ($A'_s = 1.2$ in.^{2}).

5. Check the design moment strength.

$$A_s = 5.0 \text{ in.}^2 \qquad A'_s = 1.2 \text{ in.}^2 \qquad A_{s1} = (A_s - A'_s) = 3.8 \text{ in.}^2$$

$$\phi M_n = \phi\left[A_{s1} f_y \left(d - \frac{a}{2}\right) + A'_s f'_s (d - d')\right]$$

$$= 0.9[3.8(60)(16.5 - 5.578/2) + 1.2(53.84)(16.5 - 2.5)]$$

$$= 3627.6 \text{ K·in.} = 302.3 \text{ K·ft.}$$

which is adequate. Note that the strain ε_s at the centroid level of the tension steel is less than 0.005.

$$\varepsilon_s = \left(\frac{d - c}{c}\right)0.003 = \left(\frac{16.5 - 6.56}{6.56}\right)0.003 = 0.004546$$

Both solutions are adequate.

Discussion

1. In the first solution, the net tensile strain, $\varepsilon_t = 0.005$, was assumed at the centroid of the tension steel. In this case ρ_{max} and $R_{u\,max}$ can be determined from Table 4.1 or tables in Appendix A. The strain in the lower row of bars will be always greater than 0.005, which meets the ACI Code requirement.
2. In the second solution, the strain limit, $\varepsilon_t = 0.005$, is assumed at the lower row. In this case, the strain at the centroid of the two rows of bars will be less than 0.005 and its value depends on the depth of the section. Moreover, ρ and R_u for this case are not known and their values depend on the effective depth d.
3. Comparing the two solutions, the neutral axis depth, c_1, in solution 1 is slightly smaller than c_2 for the second solution because of the strain limitations, producing a smaller A_{s1} and then higher A_{s2}. Total A_s will normally be very close. It is clear that solution 1 is easier to use because of the use of tables.
4. Note that solution 1 can have the same results as solution 2 by calculating A_{s1} as follows: $A_{s1} = \rho_{max}\, bd_t = 0.01806\ (12 \times 17.5) = 3.793\ \text{in}^2$, which is the same A_{s1} calculated in solution 2, producing $\varepsilon_t = 0.005$ at the lower row of bars.

4.5 DESIGN OF T-SECTIONS

In slab-beam-girder construction, the slab dimensions as well as the spacing and position of beams are established first. The next step is to design the supporting beams, namely, the dimensions of the web and the steel reinforcement. Referring to the analysis of T-section in the previous chapter, we can see that a large area of the compression flange, forming a part of the slab, is effective in resisting a great part or all of the compression force due to bending. If the section is designed on this basis, the depth of the web will be small; consequently, the moment arm is small, resulting in a large amount of tension steel, which is not favorable. Shear requirements should be met, and this usually requires quite a deep section.

In many cases web dimensions can be known based on the flexural design of the section at the support in a continuous beam. The section at the support is subjected to a negative moment, the slab being under tension and considered not effective, and the beam width is that of the web.

In the design of a T-section for a given factored moment, M_u, the flange thickness, t, and width, b, would have been already established from the design of the slab and the ACI Code limitations for the effective flange width, b, as given in Section 3.15. The web thickness, b_w, can be assumed to vary between 8 in. and 20 in., with a practical width of 12 to 16 in. Two more unknowns still need to be determined, d and A_s. Knowing that M_u, f'_c, and f_y are always given, two cases may develop as follows:

1. When d is given and we must calculate A_s,
   a. Check if the section acts as a rectangular or T-section by assuming $a = t$ and calculating the moment strength of the whole flange:

$$\phi M_{nf}(\text{flange}) = \phi(0.85\ f'_c)bt\left(d - \frac{t}{2}\right) \tag{4.7}$$

   If $M_u > \phi M_{nf}$, then $a > t$. If $M_u < \phi M_{nf}$, then $a < t$, and the section behaves as a rectangular section.

   b. If $a < t$, then calculate ρ using Eq. 4.2, and $A_s = \rho bd$. Check that $\rho_w \geq \rho_{min}$.

c. If $a > t$, determine A_{sf} for the overhanging portions of the flange, as explained in Section 3.15.4.

$$A_{sf} = 0.85 f_c'(b - b_w)t/f_y \tag{4.8}$$

$$M_{u2} = \phi A_{sf} f_y \left(d - \frac{t}{2}\right) \tag{4.9}$$

The moment resisted by the web is

$$M_{u1} = M_u - M_{u2}$$

Calculate ρ_1 using M_{u1}, b_w, and d in Eq. 4.2 and determine $A_{s1} = \rho_1 b_w d$.

$$\text{Total } A_s = A_{s1} + A_{sf}$$

Then check that $A_s \leq A_{s\,max}$, as explained in Section 3.15. Also check that $\rho_w = A_s/(b_w d) \geq \rho_{min}$.

d. If $a = t$, then $A_s = \phi(0.85 f_c')bt/f_y$.

2. When d and A_s are not known, the design may proceed as follows:

a. Assume $a = t$ and calculate the amount of total steel, A_{sft}, needed to resist the compression force in the whole flange, bt.

$$A_{sft} = \frac{(0.85\ f_c')bt}{f_y} \tag{4.10}$$

b. Calculate d based on A_{sft} and $a = t$ from the following equation:

$$M_u = \phi A_{sft} f_y \left(d - \frac{t}{2}\right) \tag{4.11}$$

If the depth, d, is acceptable, then $A_s = A_{sft}$ and $h = d + 2.5$ in. for one row of bars or $h = d + 3.5$ in. for two rows of bars.

c. If a new d_1 is adopted greater than the calculated d, then the section behaves as a rectangular section, and ρ can be calculated using Eq. 4.2; $A_s = \rho bd < A_{sft}$.

d. If a new d_2 is adopted that is smaller than the calculated d, then the section will act as a T-section, and the final A_s will be greater then A_{sft}. In this case, proceed as in step 1(c) to calculate A_s.

Example 4.7

The T-beam section shown in Fig. 4.7 has a web width, b_w, of 10 in., a flange width, b, of 40 in., a flange thickness of 4 in., and an effective depth, d, of 14.5 in. Determine the necessary reinforcement if the applied factored moment is 3350 K·in. Given: $f_c' = 3$ ksi and $f_y = 60$ ksi.

Solution

1. Check the position of the neutral axis; the section may be rectangular. Assume the depth of compression block a is 4 in.; that is, $a = t = 4$ in. Then

$$\phi M_n = \phi(0.85 f_c')bt \left(d - \frac{t}{2}\right) = 4590 \text{ K·in.} > M_u = 3350 \text{ K·in.}$$

The design moment that the concrete flange can resist is greater than the factored applied moment. Therefore, the section behaves as a rectangular section.

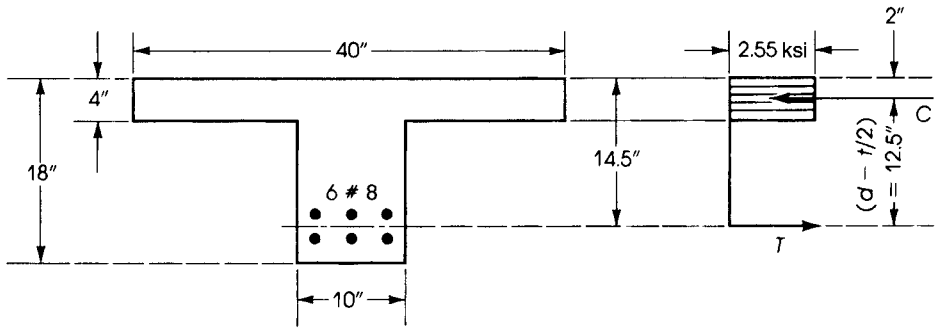


Figure 4.7 Example 4.7: T-section.

2. Determine the area of tension steel, considering a rectangular section, $b = 40$ in.

$$R_u = \phi M_n/(bd^2) = \frac{3{,}350{,}000}{40 \times 14.5^2} = 398 \text{ psi}$$

From Eq. 4.2 or from tables in Appendix A, for $R_u = 398$ psi, and $\rho = 0.00817$,

$$A_s = \rho bd = 0.00817 \times 40 \times 14.5 = 4.74 \text{ in.}^2$$

Use six no. 8 bars, $A_s = 4.74$ in.^{2} (in two rows).

3. Check that $\rho_w = A_s/b_w d \geq \rho_{\min}$; $\rho_w = 4.74/(10 \times 14.5) = 0.0327 > \rho_{\min} = 0.00333$. Note that A_s used is less than $A_{s\max}$ of 7.06 in.^{2} Calculated by Eq. 3.72.

Also, $a = 2.788$ in., $c = 3.28$ in., $d_t = 14.5$ in., and $\varepsilon_t = 0.003(d_t - c)/c = 0.01 > 0.005$, which is o.k.

Example 4.8

The floor system shown in Fig. 4.8 consists of 3-in. slabs supported by 14-ft-span beams spaced at 10 ft on centers. The beams have a web width, b_w, of 14 in. and an effective depth, d, of 18.5 in. Calculate the necessary reinforcement for a typical interior beam if the factored applied moment is 5080 K·in. Use $f'_c = 3$ ksi and $f_y = 60$ ksi.

Solution

1. Find the beam flange width: Flange width is the smallest of

$$b = 16t + b_w = 3 \times 16 + 12 = 60 \text{ in.}$$

$$b = \frac{\text{span}}{4} = \frac{14 \times 12}{4} = 42 \text{ in.}$$

Center-to-center of adjacent slabs is $10 \times 12 = 120$ in. Use $b = 42$ in.

2. Check the position of the neutral axis, assuming $a = t$.

$$\phi M_n \text{ (based on flange)} = \phi \times 0.85 f'_c bt \left(d - \frac{t}{2}\right)$$

$$= 0.9 \times 0.85 \times 3 \times 42 \times 3(18.5 - 1.5) = 4916 \text{ K·in.}$$

The applied moment is $M_u = 5080$ K·in. > 4916 K·in.; the beam acts as a T-section, so $a > t$.

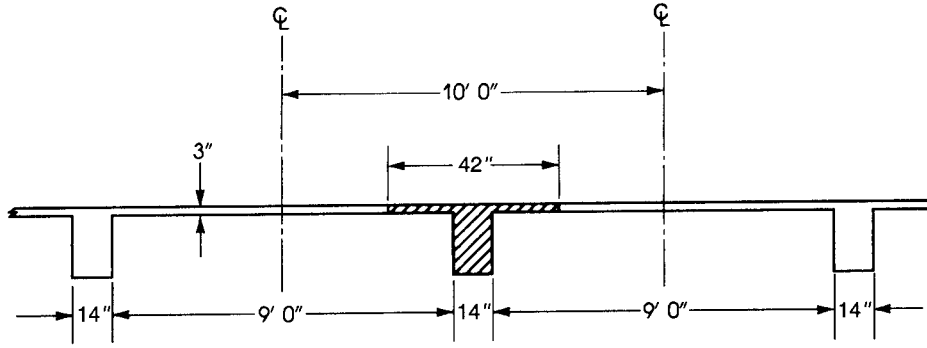


Figure 4.8 Example 4.8: effective flange width.

3. Find the portion of the design moment taken by the overhanging portions of the flange (Fig. 4.9). First calculate the area of steel required to develop a tension force balancing the compressive force in the projecting portions of the flange:

$$A_{sf} = \frac{0.85 f_c'(b - b_w)t}{f_y} = \frac{0.85 \times 3 \times (42 - 14) \times 3}{60} = 3.57 \text{ in.}$$

$\phi M_n = M_{u1} + M_{u2}$, that is, the sum of the design moment of the web and the design moment of the flanges.

$$M_{u2} = \phi A_{sf} f_y \left(d - \frac{t}{2}\right)$$

$$= 0.9 \times 3.57 \times 60 \left(18.5 - \frac{3}{2}\right) = 3272 \text{ K·in.}$$

4. Calculate the design moment of the web (as a singly reinforced rectangular section):

$$M_{u1} = M_u - M_{u2} = 5080 - 3272 = 1808 \text{ K·in.}$$

$$R_u = \frac{M_{u1}}{(b_w d^2)} = \frac{1{,}808{,}000}{14 \times (18.5)^2} = 377 \text{ psi}$$

From Eq. 4.2 or the tables in Appendix A, for $R_u = 377$ psi, $\rho_1 = 0.0077$.

$$A_{s1} = \rho_1 b_w d = 0.0077(14)(18.5) = 1.99 \text{ in.}^2$$

Total $A_s = A_{sf} + A_{s1} = 3.57 + 1.99 = 5.56 \text{ in.}^2$ (Use six no. 9 bars in two rows.)

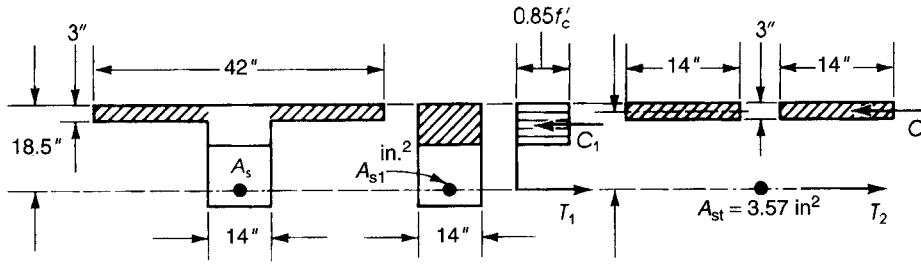


Figure 4.9 Analysis of Example 4.8.

5. Total $h = 18.5 + 3.5 = 22$ in. Calculate $A_{s\,\max}$ for T-sections using Eq. 3.72:

$$\text{Max } A_s = 7.02 \text{ in.}^2 > 5.56 \text{ in.}^2$$

6. Check ε_t: $a = 1.99 \times 60/(0.85 \times 3 \times 14) = 3.34$ in., $c = 3.93$ in., $d_t = 19.5$ in. Then $\varepsilon_t = 0.003(d_t - c)/c = 0.0119 > 0.005$, tension-controlled section ($\phi = 0.9$).

Example 4.9

In a slab-beam system, the flange width was determined to be 48 in., the web width was $b_w = 16$ in., and the slab thickness was $t = 4$ in. (Fig. 4.10). Design a T-section to resist an external factored moment of $M_u = 812$ K·ft. Use $f'_c = 3$ ksi and $f_y = 60$ ksi.

Solution

1. Because the effective depth is not given, let $a = t$ and calculate A_{sft} for the whole flange.

$$A_{sft} = \frac{0.85 f'_c bt}{f_y} = \frac{0.85(3)(48)(4)}{60} = 8.16 \text{ in.}^2$$

Let $M_u = \phi A_{sft} f_y (d - t/2)$ and calculate d:

$$812 \times 12 = 0.9(8.16)(60)\left(d - \frac{4}{2}\right) \quad d = 24.1 \text{ in.}$$

Now, if an effective $d = 24.1$ in. is chosen, then $A_s = A_{sft} = 8.16$ in.^{2}

2. If a depth $d > 24.1$ in. is chosen, say 26.5 in., then $a < t$ and it is a rectangular analysis. The steel ratio can be calculated from Eq. 4.2 with $\rho = 0.00574$ and $A_s = \rho bd = 0.00574 \times 48 \times 26.5 = 7.3$ in.^{2} (six no. 10 bars in two rows, $A_s = 7.62$ in.^{2}).

3. If a depth $d < 24.1$ in. is chosen, say, 23.5 in., then $a > t$, and the section behaves as a T-section. Calculate

$$A_{sf} = 0.85 f'_c t(b - b_w)/f_y = 0.85(3)(4)(48 - 16)/60 = 5.44 \text{ in.}^2$$

$$M_{u2} = \phi A_{sf} f_y \left(d - \frac{t}{2}\right) = 0.9(5.44)(60)\left(23.5 - \frac{4}{2}\right) = 6316 \text{ K·in.}$$

$$M_{u1} = 812 \times 12 - 6316 = 3428 \text{ K·in.}$$

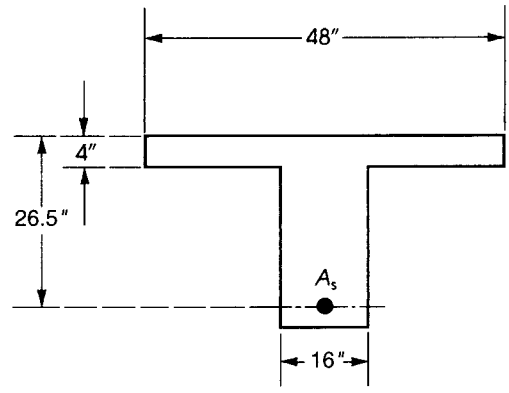


Figure 4.10 Example 4.9.

4. For the basic singly reinforced section, $b_w = 16$ in., $d = 23.5$ in., and $M_{u1} = 3428$ K·in., $R_u = 387$ psi. Calculate ρ_1 from Eq. 4.2 to get $\rho_1 = 0.0079$.

$$A_{s1} = \rho_1 b_w d = 0.0079(16)(23.5) = 2.97 \text{ in.}^2$$

$$\text{Total } A_s = A_{sf} + A_{s1} = 5.44 + 2.97 = 8.41 \text{ in.}^2 \text{(seven no. 10 bars in two rows,}$$

$$A_s = 8.89 \text{ in.}^2)$$

5. Check ε_t: $a = 2.97 \times 60/(.85 \times 3 \times 16) = 4.368$ in., $c = a/0.85 = 5.14$ in., $d_t = 24.5$ in., and $\varepsilon_t = 0.003\ (d_t - c)/c = 0.0113 > 0.005$, a tension-controlled section.
6. Calculate the total max A_s that can be used for the T-section by Eq. 3.72:

$$\text{Max } A_s =$$

$$= 0.0425[(b - bw)t + 0.319bwd] = 10.54 \text{ in.}^2$$

$$A_s \text{ (used)} \leq \max A_s$$

7. *Note* : If there are no restrictions on the total depth of the beam, it is a common practice to adopt the case when $a \leq t$ (step 2). This is because an increase in d produces a small increase in concrete in the web only while decreasing the quantity of A_s required.

4.6 ADDITIONAL EXAMPLES

The following design examples give some practical applications and combine structural analysis with concrete design of beams and frames.

Example 4.10

For the precast concrete I-section shown in Fig. 4.11, calculate the reinforcement needed to support a factored moment of 360 K·ft. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

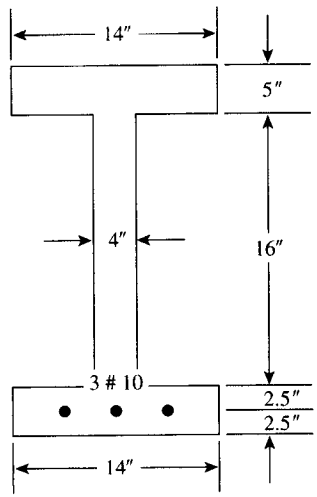


Figure 4.11 Example 4.10.

Solution

Determine if the force in the flange area 14 × 5 in. will be sufficient to resist a factored moment of 360 K-ft. Let $d = 23.5$ in. Force in flange $(C_c) = 0.85 \times f'_c$ (flange area) $= 0.85 \times 4 \times (14 \times 5) =$ 238 K, located at 2.5 in. from the top fibers, and $a = 5$ in.

$$\phi M_n = 0.9C_c\left(d - \frac{a}{2}\right) = 0.9 \times 238\frac{(23.5 - 2.5)}{12} = 374.9 \text{ K-ft}$$

which is greater than the applied moment of 360 K-ft. Therefore, a is less than 5 in.

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right), \text{ where } a = \frac{A_s f_y}{(0.85 f'_c b)}$$

$$360 \times 12 = 0.9A_s(60)\left(\frac{23.5 - 60A_s}{(1.7 \times 4 \times 14)}\right)$$

Solve to get $A_s = 3.79$ in.^{2} Or use Eq. 4.2 to get $\rho = 0.01152$ and $A_s = 0.01152 \times 14 \times 23.5 =$ 3.79 in.^{2} Use three no. 10 bars in one row, as shown in Fig. 4.11.

For similar T-sections or I-sections, it is better to adopt a section with a flange size to accommodate the compression force, C_c. In this case, a is less than or equal to the flange depth. The bottom flange is in tension and not effective.

Example 4.11

The simply supported beam shown in Fig. 4.12 carries a uniform service load of 2.8 K/ft (including self-weight) in addition to a service load of 1.6 K/ft. Also, the beam supports a concentrated dead load of 16 K and a concentrated live load of 7 K at C, 10 ft from support A.

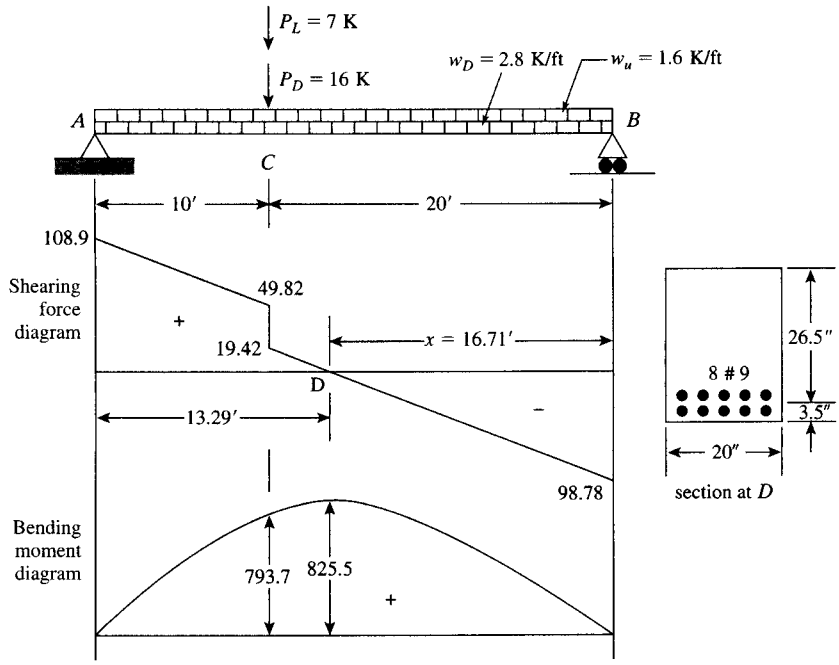


Figure 4.12 Example 4.11.

a. Determine the maximum factored moment and its location on the beam.

b. Design a rectangular section to carry the loads safely using a steel percentage of about 1.5%, $b = 20$ in., $f'_c = 4$ ksi, and $f_y = 60$ ksi.

Solution

a. Calculate the uniform factored load: $w_u = 1.2(2.8) + 1.6(1.6) = 5.91$ K/ft. Calculate the concentrated factored load: $P_u = 1.2(16) + 1.6(7) = 30.4$ K. Calculate the reaction at A by taking moments about B:

$$R_A = 5.91(30)\frac{(30/2)}{30} + \frac{30.4(20)}{30} = 108.92 \text{ K}$$

$$R_B = 5.91(30) + 30.4 - 108.92 = 98.78 \text{ K}$$

Maximum moment in the beam occurs at zero shear. Starting from B,

$$V = 0 = 98.78 - 5.91x \text{ and } x = 16.71 \text{ ft from } B \text{ at } D$$

$$M_u \text{ (at } D) = 98.72(16.71) - 5.91(16.71)\left(\frac{16.71}{2}\right) = 825.5 \text{ K·ft} \quad \text{(design moment)}$$

$$M_u \text{ (at } C) = 98.78(20) - 5.91(20)\left(\frac{20}{2}\right) = 793.6 \text{ K·ft}$$

b. Design of the section at D: For $f'_c = 4$ ksi, and $f_y = 60$ ksi, $\rho_{max} = 0.01806$ and $\rho_{min} = 0.00333$, and the design steel ratio of 1.5% is within the limits. For $\rho = 0.015$, $R_u = 700$ psi (from Table A.2) or from Eq. 3.22.

$$M_u = R_u b d^2 \quad \text{or} \quad 825.5 \times 12 = 0.7(20)d^2$$

Solve to get $d = 26.6$ in.

$$A_s = 0.015 \times 20 \times 26.6 = 7.98 \text{ in.}^2$$

Choose eight no. 9 bars in two rows (area = 8 in.^{2}), five in the lower row plus three in the upper row. Minimum b for five no. 9 bars in one row is 14 in. (Table A.7). Total depth (h) = 26.6 + 3.5 = 30.1 in. Use $h = 30$ in. Actual $d = 30 - 3.5 = 26.5$ in. Check the moment capacity of the section, $a = 8 \times 60/(0.85 \times 4 \times 20) = 7.06$ in.

$$\phi M_n = 0.9 \times 8 \times 60\frac{\left(26.5 - \frac{7.06}{2}\right)}{12} = 826.9 \text{ K·ft}$$

which is greater than 825.5 K·ft. Check that $A_s = 8$ in.^{2} is less than $A_{s\,max}$.

$$A_{s\,max} = 0.01806 \times 20 \times 26.5 = 9.57 \text{ in.}^2$$

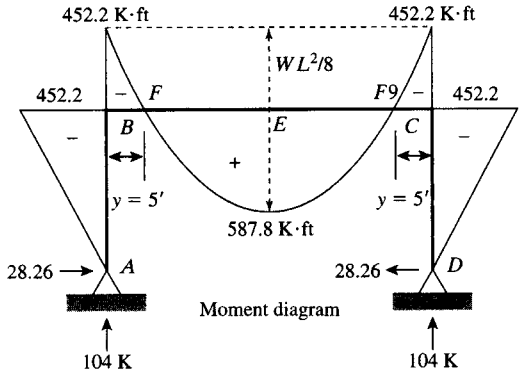
which exceeds 8 in.^{2} The final section is shown in Fig. 4.12.

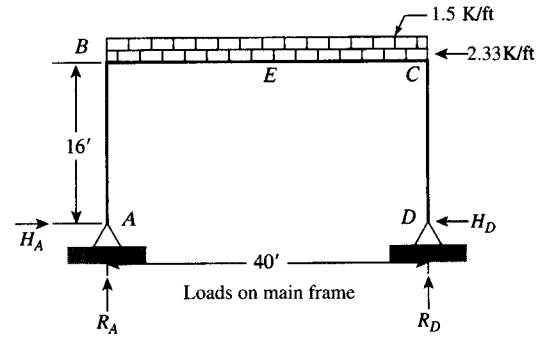
Example 4.12

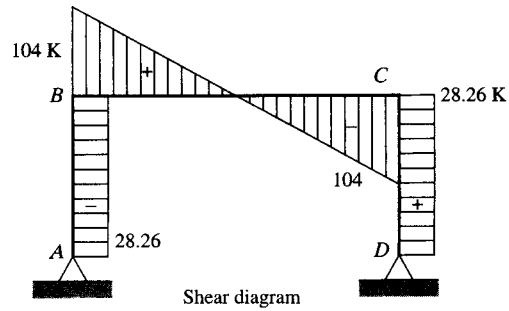
The two-hinged frame shown in Fig. 4.13 carries a uniform service dead load (including estimated self-weight) of 2.33 K/ft and a uniform service live load of 1.5 K/ft on frame beam BC. The moment at the corner B (or C) can be evaluated for this frame dimension, $M_b = M_c = -wL^2/18.4$ and the reaction at A or $D = wL/2$. A typical section of beam BC is shown, the column section is 16 × 21 in. It is required to

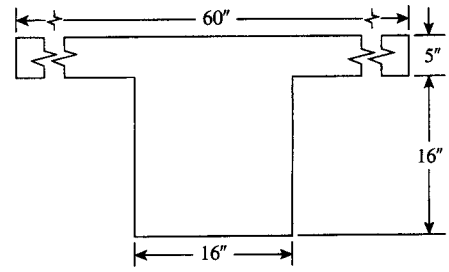
a. Draw the bending moment and shear diagrams for the frame $ABCD$ showing all critical values.

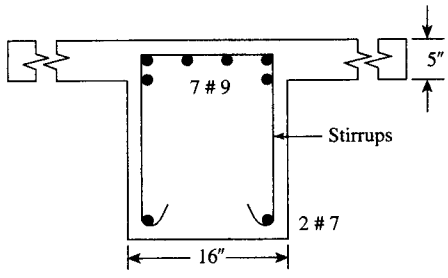
b. Design the beam BC for the factored moments, positive and negative, using $f'_c = 4$ ksi and $f_y = 60$ ksi. Show reinforcement details.

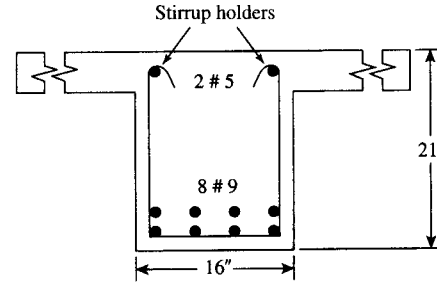












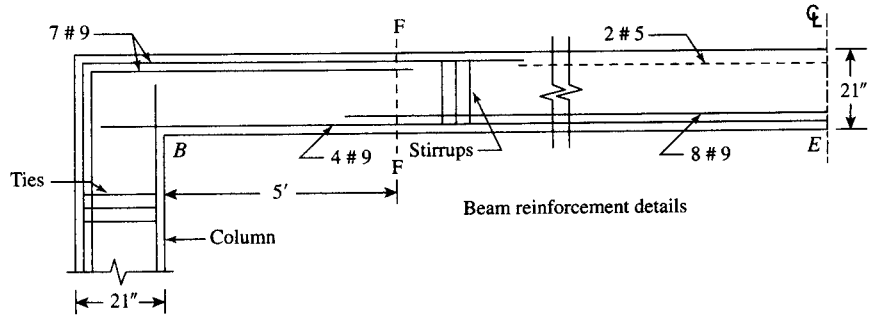


Figure 4.13 Example 4.12.

Solution

a. Calculate the forces acting on the frame using a computer program or the values mentioned previously. Factored load (w_u) = 1.2(2.33) + 1.6(1.5) = 5.2 K/ft. Because of symmetry, $M_B = M_C = -wL^2/18.4 = -5.2(40)^2/18.4 = -452.2$ K·ft. Positive moment at midspan (E) $= w_uL^2/8 + M_B = 5.2(40)^2/8 - 452.2 = 587.8$ K·ft. Vertical reaction at $A = R_A = R_D = w_uL/2 = 5.2(40)/2 = 104$ K. Horizontal reaction at $A = H_A = M_B/h = 452.2/16 = 28.26$ K. The moment and shear diagrams are shown in Fig. 4.13.

Determine the location of zero moment at section F on beam BC by taking moments about F:

$$104(y) - 28.26(16) - 5.2(y)^2/2 = 0 \qquad y = 4.963 \text{ ft}, \qquad \text{say, 5 ft from joint } B$$

b. Design of beam BC:

1. Design of section E at midspan: $M_u = +587.8$ K·ft. Assuming two rows of bars, $d = 21 - 3.5 = 17.5$ in. Calculate the moment capacity of the flange using $a = 5.0$ in.

$$\phi M_n(\text{flange}) = \phi(0.85f'_c)ab\left(d - \frac{5}{2}\right)$$

$$= 0.9(0.85 \times 4) \times (5 \times 60) \times \frac{(17.5 - 2.5)}{12} = 1147.5 \text{ K·ft}$$

which is greater than the applied moment; therefore, a is less than 5.0 in.

Assume $a = 2.0$ in. and calculate A_s.

$$M_u = \phi A_s f_y\left(d - \frac{a}{2}\right)$$

$$587.8 \times 12 = 0.9 \times 60A_s(17.5 - 1.0) \quad \text{and} \quad A_s = 7.92 \text{ in.}^2$$

Check assumed $a = A_s f_y/(0.85f'_c b) = 7.92 \times 60/(0.85 \times 4 \times 60) = 2.33$ in. Revised $A_s = 587.8 \times 12/(0.9 \times 60 \times 16.33) = 7.99$ in.^{2} Check revised a: $a = 7.99 \times 2.33/7.92 = 2.35$ in., which is very close to 2.33 in.

Alternatively, Eq. 4.2 can be used to get ρ and A_s. Choose eight no. 9 bars in two rows (area = 8.0 in.^{2}), ($b_{\min} = 11.8$ in.). Extend four no. 9 bars on both sides to the columns. The other four bars can terminate where they are not needed, beyond section F; see the longitudinal section in Fig. 4.13.

2. Design of section at B: $M_u = -452.2$ K·ft. The section acts as a rectangular section, $b = 16$ in. and $d = 17.5$ in. The main tension reinforcement lies in the flange.

$$\rho_{\max} = 0.01806 \quad \text{and} \quad R_{u\max} = 820 \text{ psi (Table 4.1)}$$

Check the maximum moment capacity of the section as singly reinforced.

$$\phi M_{n\max} = R_{u\max}bd^2 = 0.82(16)(17.5)^2/12 = 334.8 \text{ K·ft}$$

which is less than the applied moment. Compression steel is needed.

$$A_{s1} = 0.01806(16)(17.5) = 5.06 \text{ in.}^2$$

$$M_{u2} = 452.2 - 334.8 = 117.4 \text{ K·ft}$$

$$M_{u2} = \phi A_{s2} f_y(d - d'); \text{ assume } d' = 2.5 \text{ in.}$$

$$117.4 \times 12 = 0.9A_{s2}(60)(17.5 - 2.5) \quad \text{and} \quad A_{s2} = 1.74 \text{ in.}^2$$

Total tension steel = 5.06 + 1.74 = 6.8 in.^{2} Use seven no. 9 bars in two rows (area used = 7.0 in.^{2}, which is adequate). For compression steel, use two no. 9 bars (area = 2.0 in.^{2}),

extended from the positive moment reinforcement to the column. Actually, four no. 9 bars are available; see the longitudinal section in Fig. 4.13.

The seven no. 9 bars must extend in the beam BC beyond section F into the compression zone, and also must extend into the column BA to resist the column moment of 452.2 K·ft without any splices at joints B or C.

Check if compression steel yields by using Eq. 3.49 or Table 3.4. K = 0.01552 (d'/d) = 0.1552(2.5)/(17.5) = 0.02217 > ρ_1 = 0.01806. Therefore, compression steel yields, and f'_s = 60 ksi as assumed.

Stirrups are shown in the beam to resist shear (refer to Chapter 8), and two no. 5 bars were placed at the top of the beam to hold the stirrups in position. Ties are used in the column to hold the vertical bars (refer to Chapter 10). To determine the extension of the development length of bars in beams or columns, refer to Chapter 7.

4.7 EXAMPLES USING SI UNITS

Example 4.13

Design a singly reinforced rectangular section to resist a factored moment of 280 kN·m using the maximum steel percentage for tension-controlled sections. Given: $f'_c = 20$ N/mm^2, $f_y = 400$ N/mm^2, and $b = 250$ mm.

Solution

$$\rho_b = (0.85)\beta_1 \left[\frac{f'_c}{f_y}\right]\left(\frac{600}{600+f_y}\right)$$

$$= 0.85 \times 0.85 \times \frac{20}{400} \times \left(\frac{600}{600+400}\right) = 0.0217$$

$$\rho_{\max} = \left(\frac{0.003 + \frac{f_y}{E_s}}{0.008}\right)\rho_b \qquad E_s = 200{,}000 \text{ MPa} \qquad \frac{f_y}{E_s} = 0.002$$

$$= 0.625\ \rho_b = 0.01356$$

$$\phi = 0.9$$

$$R_{u\max} = \phi\rho_{\max} f_y \left(1 - \frac{\rho_{\max} f_y}{1.7 f'_c}\right)$$

$$= 0.9 \times 0.01356 \times 400\left(1 - \frac{0.01356 \times 400}{1.7 \times 20}\right) = 4.1 \text{ N/mm}^2\text{(MPa)}$$

$$M_u = R_u b d^2$$

$$d = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{280 \times 10^6}{4.1 \times 250}} = 523 \text{ mm}$$

$$A_s = \rho b d = 0.01356 \times 250 \times 523 = 1772 \text{ mm}^2 = 17.72 \text{ cm}^2$$

Choose four bars, 25 mm diameter, in two rows.

A_s provided = 4 × 4.9 = 19.6 cm^2. Total depth is

$$h = d + 25 \text{ mm} + 60 \text{ mm}$$

$$= 523 + 25 + 60 = 608 \text{ mm} \qquad \text{say, 610 mm (or 600 mm)}$$

Check minimum width:

$$b_{\min} = 2D + 1S + 95 \text{ mm} = 3 \times 25 + 95 = 170 \text{ mm} < 250 \text{ mm}$$

Bars are placed in two rows.

Example 4.14

Calculate the required reinforcement for a beam that has a section of $b = 300\,\text{mm}$ and a total depth of $h = 600\,\text{mm}$ to resist $M_u = 696$ kN·m. Given: $f'_c = 30\ \text{N/mm}^2$ and $f_y = 420\,\text{N/mm}^2$.

Solution

1. Determine the design moment strength of the section using $\rho_{\max}$ (for tension-controlled section, $\phi = 0.9$):

$$\rho_b = (0.85)\beta_1 \left[\frac{f'_c}{f_y}\right]\left(\frac{600}{600 + f_y}\right)$$

$$= 0.85 \times 0.85 \times \frac{30}{420} \times \left(\frac{600}{600 + 1020}\right) = 0.0304$$

$$\rho_{\max} = \left(\frac{0.003 + \frac{f_y}{E_s}}{0.008}\right)\rho_b = 0.6375\ \rho_b = 0.01938$$

$$R_{u\max} = \phi\rho_{\max} f_y \left(1 - \frac{\rho_{\max} f_y}{1.7 f'_c}\right)$$

$$= 0.9 \times 0.01938 \times 420\left(1 - \frac{0.01938 \times 420}{1.7 \times 30}\right) = 6.16\ \text{N/mm}^2\text{(MPa)}$$

$$d = h - 85\ \text{mm(assuming two rows of bars)}$$

$$= 600 - 85 = 515\ \text{mm}$$

$$\phi M_n = R_u b d^2 = 6.16 \times 300 \times (515)^2 \times 10^{-6} = 490\ \text{kN·m}$$

This is less than the external moment; therefore, compression reinforcement is needed.

2. Calculate A_{s1}, M_{u1}, and M_{u2}:

$$A_{s1} = \rho_{\max} bd = 0.01938 \times 300 \times 515 = 2994\ \text{mm}^2$$

$$M_{u2} = M_u - M_{u1} = 696 - 490 = 206\ \text{kN·m}$$

3. Calculate A_{s2} and A'_s due to M_{u2}. Assume d′ = 60 mm:

$$M_{u2} = \phi A_{s2} f_y (d - d')$$

$$206 \times 10^6 = 0.9 A_{s2} \times 420(515 - 60) \quad A_{s2} = 1198\ \text{mm}^2$$

Total tension steel is $2994 + 1198 = 4192\,\text{mm}^2$.

4. Compression steel yields if

$$(\rho - \rho') = \rho_1 \geq 0.85\beta_1 \times \frac{f'_c}{f_y} \times \frac{d'}{d} \times \frac{600}{600 - f_y} = K$$

$$K = (0.85)^2 \times \frac{30}{420} \times \frac{60}{515} \times \frac{600}{600 - 420} = 0.020$$

Because $(\rho - \rho') = \rho_{\max} = 0.01938 < 0.020$, compression steel does not yield.

5. Calculate

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$= \frac{2994(420)}{0.85 \times 30 \times 300} = 164.4 \text{ mm}$$

$$c = \frac{a}{0.85} = 193.4 \text{ mm} \qquad d' = 60 \text{ mm}$$

$$f'_c = 600\left(\frac{c - d'}{c}\right) = 414 \text{ N/mm}^2$$

$$A'_s = A_{s2}(420/414) = 1215 \text{ mm}^2$$

6. Choose steel bars as follows: For tension, choose six bars 30 mm in diameter (30 M). The A_s provided (4200 mm) is greater than A_s, as required. For compression steel, choose three bars 25 mm in diameter (25 M) (Table B.11).

$$A'_s = 1500 \text{ mm}^2 > 1215 \text{ mm}^2$$

SUMMARY

Sections 4.1–4.3 : Design of a Singly Reinforced Rectangular Section

Given: M_u (external factored moment), f'_c (compressive strength of concrete), and f_y (yield stress of steel).

Case 1.
When b, d, and A_s (or ρ) are *not* given:

1. Assume $\rho_{\min} \leq \rho \leq \rho_{\max}$. Choose $\rho_{\max}$ for a minimum concrete cross-section (smallest) or choose ρ between $\rho_{\max}/2$ and $\rho_b/2$ for larger sections. For example, if $f_y = 60$ ksi, you may choose

$$\rho = 1.2\% \qquad R_n = 618 \text{ psi} \quad \text{for } f'_c = 3 \text{ ksi}$$

$$\rho = 1.4\% \qquad R_n = 736 \text{ psi} \quad \text{for } f'_c = 4 \text{ ksi}$$

$$\rho = 1.4\% \qquad R_n = 757 \text{ psi} \quad \text{for } f'_c = 5 \text{ ksi}$$

   For any other value of ρ, $R_n = \rho f_y[1 - (\rho f_y/1.7 f'_c)]$, and $R_u = \phi R_n$.
2. Calculate $bd^2 = M_u/\phi R_n (\phi = 0.9)$, for tension-controlled sections.
3. Choose b and d. The ratio of d to b is approximately $1 \rightarrow 3$, or $d/b \approx 2.0$.
4. Calculate $A_s = \rho bd$; then choose bars to fit in b either in one row or two rows. (Check $b_{\min}$ from the tables.)
5. Calculate

$$h = d + 2.5 \text{ in. (for one row of bars)}$$

$$h = d + 3.5 \text{ in. (for two rows of bars)}$$

   b and h must be to the nearest higher inch. *Note:* If h is increased, calculate new $d = h - 2.5$ (or 3.5) and recalculate A_s to get a smaller value.

Case 2.
When ρ is given, d, b, and A_s are required. Repeat steps (1) through (5) from Case 1.

Case 3.
When b and d (or h) are given, A_s is required.

1. Calculate $R_n = M_u/\phi bd^2$ ($\phi = 0.9$).
2. Calculate
$$\rho = \left(\frac{0.85f'_c}{f_y}\right)\left[1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}}\right]$$
(or get ρ from tables or Eq. 4.2).
3. Calculate $A_s = \rho bd$, choose bars, and check $b_{\min}$.
4. Calculate h to the nearest higher inch (see note, Case 1 (step 5)).

Case 4.
When b and ρ are given, d and A_s are required.

1. Calculate
$$R_n = \rho f_y\left(1 - \frac{\rho f_y}{1.7f'_c}\right) \qquad R_u = \phi R_n(\phi = 0.9)$$
2. Calculate
$$d = \sqrt{\frac{M_u}{\phi R_n b}}$$
3. Calculate $A_s = \rho bd$, choose bars, and check $b_{\min}$.
4. Calculate h to the nearest higher inch (see note, Case 1 (step 5)). *Note.* Equations that may be used to check the moment capacity of the section after the final section is chosen are
$$M_u = \phi M_n = \phi A_s f_y\left(d - \frac{A_s f_y}{1.7f'_c b}\right) = \phi A_s f_y d\left(1 - \frac{\rho f_y}{1.7f'_c}\right)$$
$$= \phi\rho f_y(bd^2)\left(1 - \frac{\rho f_y}{1.7f'_c}\right) = R_u bd^2$$

Section 4.4: Design of Rectangular Sections with Compression Steel

Given: M_u, b, d, d', f'_c, f_y, and $\phi = 0.9$.
Required: A_s and A'_s.

1. *General*
   a. Calculate $\rho_{\max}$ and $\rho_{\min}$ as singly reinforced from equations (or from tables).
   b. Calculate $R_{n\max} = \rho_{\max} f_y\left[1 - \left(\frac{\rho_{\max} f_y}{1.7f'_c}\right)\right]$ (or use tables).
   c. Calculate the maximum capacity of the section as singly reinforced:
$$\phi M_n = \phi R_{n\max} bd^2.$$

d. If $M_u > \phi M_n$, then compression steel is needed. If $M_u < \phi M_n$, it is a singly reinforced section.

2. If $M_u > \phi M_n$ and compression steel is needed,
   a. Let $M_{u1} = \phi R_{n\max} bd^2$.
   b. Calculate $A_{s1} = \rho_{\max} bd$ (basic section).
   c. Calculate $M_{u2} = M_u - M_{u1}$ (for the steel section).
3. Calculate A_{s2} and A'_s as steel section.
   a. $M_{u2} = \phi A_{s2} f_y (d - d')$.
   b. Calculate total tension steel: $A_s = A_{s1} + A_{s2}$.
4. Calculate A'_s (compression steel area):
   a. Calculate $a = (A_{s1} f_y / 0.85 f'_c b)$ and $c = a/\beta_1$.
   b. Calculate $f'_s = 87[(c - d')/c] \leq f_y$.
   If $f'_s \geq f_y$, then $f'_s = f_y$ and $A'_s = A_{s2}$.
   If $f'_s < f_y$, then $A'_s = A_{s2}\left(\frac{f_y}{f'_s}\right)$.
   c. Check that total steel area $(A_s) \geq \max A_s$, or check $\varepsilon_t \geq 0.005$

$$A_s \leq \left[\rho_{\max}(bd) + A'_s\left(\frac{f'_s}{f_y}\right)\right]$$

Section 4.5: Design of T-Sections

Given: M_u, f'_c, f_y, b, t, and b_w.
Required: A_s and d (if not given).
There are two cases:

Case 1.
When d and A_s (or ρ) are *not* given:

1. Let $a \leq t$ (as singly reinforced rectangular section). If $a = t$ is assumed, then

$$M_u = (\text{total flange}) = \phi(0.85 f'_c) bt\left(d - \frac{t}{2}\right) = \phi A_s f_y\left(d - \frac{t}{2}\right)$$

Solve for d and then for A_s.

$$d = \frac{M_u}{\phi(0.85 f'_c) bt} + \frac{t}{2} \qquad A_s = \frac{M_u}{\phi f_y\left(d - \frac{t}{2}\right)}$$

2. If a is assumed to be less than t, then

$$d = \frac{M_u}{\phi(0.85 f'_c) ba} + \frac{a}{2} \quad \text{and} \quad A_s = \frac{M_u}{\phi f_y\left(d - \frac{a}{2}\right)}$$

Case 2.
When d is given and A_s is required (one unknown):

1. Check if a is greater or less than t by considering the moment capacity of the flange (bt).

$$\text{(flange) } \phi M_n = \phi(0.85 f_c')bt\left(d - \frac{t}{2}\right)$$

If $\phi M_n > M_u$ (external), then $a < t$ (rectangular section).
If $\phi M_n < M_u$ (external), then $a > t$ (T-section).

2. If $a < t$, calculate $R_n = M_u/\phi bd^2$ and then calculate ρ (or determine ρ from tables or Eq. 4.2):

$$\rho = \frac{0.85 f_c'}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{0.85 f_c'}}\right)$$

Then calculate $A_s = \rho bd$.

3. If $a > t$,
   a. Calculate C_f and A_{sf}.

$$A_{sf} = 0.85 f_c' t\frac{(b - b_w)}{f_y} = \frac{C_f}{f_y}\text{(flange)}$$

   Then calculate M_{uf} (flange) $= \phi C_f(d - t/2)$.
   b. Calculate M_{uw} (web) $= M_u - M_{uf}$. Calculate R_n (web) $= M_{uw}/(\phi b_w d^2)$; then find ρ_w (use the equation or tables). Calculate A_{sw} (web) $= \rho_w b_w d$.
   c. Total $A_s = A_{sf}$ (flange) $+ A_{sw}$ (web). Total A_s must be less than or equal to $A_{s\,max}$ and greater than or equal to $A_{s\,min}$.
   d. $\rho_w = \left(\frac{0.8 f_c'}{f_y}\right)\left(1 - \sqrt{1 - \frac{2R_n}{0.85 f_c'}}\right)$
   e. Check that $\rho_w = A_s/b_w d \geq \rho_{min}$ (ρ_w = steel ratio in web) or $A_s > A_{s\,min}$, where $A_{s\,min} = \rho_{min}(b_w d)$. Check that $A_s \leq$ max A_s; or check $\varepsilon_t = (d_t - c)/c \geq 0.005$

PROBLEMS

4.1 Based on the information given in the accompanying table and for each assigned problem, design a singly reinforced concrete section to resist the factored moment shown in boldface. Use $f_c' = 4$ ksi and $f_y = 60$ ksi, and draw a detailed, neat section.

No.	M_u (K-ft)	b (in.)	d (in.)	ρ %
a	272.7	12	21.5	—
b	969.2	18	32.0	—
c	816.0	16	—	1.70
d	657.0	16	—	1.50
e	559.4	14	—	1.75
f	254.5	10	21.5	—
g	451.4	14	—	1.80
h	832.0	18	28.0	—
i	345.0	15	—	1.77

(continues)

No.	M_u (K·ft)	b (in.)	d (in.)	ρ %
j	510.0	0.5 d	—	ρ_{max}
k	720.0	—	2.5b	1.80
l	605.0	—	1.5b	1.80

For problems in SI units, 1 in. = 25.4 mm, 1 ksi = 6.9 MPa (N/mm_2), and 1 M_u (K·ft) = 1.356 kN·m.

4.2 Based on the information given in the following table and for each assigned problem, design a rectangular section with compression reinforcement to resist the factored moment shown. Use f'_c = 4 ksi, f_y = 60 ksi, and d' = 2.5 in. Draw detailed, neat sections.

No.	M_u (K·ft)	b (in.)	d (in.)
a	554	14	20.5
b	790	16	24.5
c	448	12	18.5
d	520	12	20.5
e	765	16	20.5
f	855	18	22.0
g	555	16	18.5
h	300	12	16.5
i	400	16	16.5
j	280	12	16.5
k	290	14	14.5
l	400	14	17.5

For problems in SI units, 1 in. = 25.4 mm, 1 ksi = 6.9 MPa (N/mm2), and 1 M_u (K·ft) = 1.356 kN·m.

4.3 Based on the information given in the following table and for each assigned problem, calculate the tension steel and bars required to resist the factored moment shown. Use f'_c = 3 ksi and f_y = 60 ksi. Draw detailed, neat sections.

No.	M_u (K·ft)	b (in.)	b_w (in.)	t (in.)	d (in.)	Notes
a	394	48	14	3	18.5	
b	800	60	16	4	19.5	
c	250	44	15	3	15.0	
d	327	50	14	3	13.0	
e	577	54	16	4	18.5	
f	559	48	14	4	17.5	
g	388	44	12	3	16.0	
h	380	46	14	3	15.0	
i	537	60	16	3	16.5	
j	515	54	16	3	17.5	
k	361	44	15	3	15.0	
l	405	50	14	3	15.5	
m	378	44	16	3	—	Let $a = t$.
n	440	36	16	4	—	Let $a = t$.
o	567	48	12	3	—	Let A_s = 6.0 in^2.
p	507	46	14	3	—	Let A_s = 7.0 in^2.

For problems in SI units, 1 in. = 25.4 mm, 1 ksi = 6.9 MPa (N/mm2), and 1 1Mu (K·ft) = 1.356 kN·m.

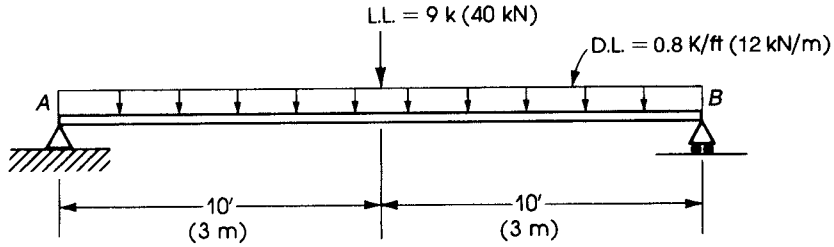


Figure 4.14 Problem 4.4.7.

4.4 Design a singly reinforced rectangular section to resist a factored moment of 232 K·ft (320 kN·m) if $f'_c = 4$ ksi (28 MPa), $f_y = 60$ ksi (420 MPa), and $b = 10$ in. (250 mm), using (a) ρ_{max}, (b) $\rho = 0.016$, and (c) $\rho = 0.012$.

4.5 Design a singly reinforced section to resist a factored moment of 186 K·ft (252 kN·m) if $b = 12$ in. (275 mm), $d = 20$ in. (500 mm), $f'_c = 3$ ksi (20 MPa), and $f_y = 40$ ksi (300 MPa).

4.6 Determine the reinforcement required for the section given in Problem 4.5 when $f'_c = 4$ ksi (30 MPa), and $f_y = 60$ ksi (400 MPa).

4.7 A simply supported beam has a 20-ft (6-m) span and carries a uniform dead load of 800 lb/ft (12 kN/m) and a concentrated live load at midspan of 9 kips (40 kN) (Fig. 4.14). Design the beam if $b = 12$ in. (300 mm), $f'_c = 4$ ksi (30 MPa), and $f_y = 60$ ksi (400 MPa). (Beam self-weight is not included in the dead load.)

4.8 A beam with a span of 24 ft (7.2 m) between supports has an overhanging extended part of 8 ft (2.4 m) on one side only. The beam carries a uniform dead load of 2.3 K/ft (30 kN/m) (including its own weight) and a uniform live load of 1.3 K/ft (18 kN/m) (Fig. 4.15). Design the smallest singly reinforced rectangular section to be used for the entire beam. Select steel for positive and negative moments. Use $f'_c = 4$ ksi (30 MPa), $f_y = 60$ ksi (400 MPa), and $b = 12$ in. (300 mm). (Determine the maximum positive and maximum negative moments by placing the live load once on the span and once on the overhanging part.)

4.9 Design a 15-ft (4.5-m) cantilever beam of uniform depth to carry a uniform dead load of 0.88 K/ft (12 kN/m) and a live load of 1.1 K/ft (15 kN/m). Assume a beam width $b = 14$ in. (350 mm), $f'_c = 4$ ksi (30 MPa), and $f_y = 60$ ksi (400 MPa).

4.10 10-ft (3-m) cantilever beam carries a uniform dead load of 1.50 K/ft (20 kN/m) (including its own weight) and a live load of 0.77 K/ft (10 kN/m) (Fig. 4.16). Design the beam using a variable depth. Draw all details of the beam and reinforcement. Given: $f'_c = 3$ ksi (20 MPa), $f_y = 40$ ksi (300 MPa), and $b = 12$ in. (300 mm). Assume h at the free end is 10 in. (250 mm).

4.11 Determine the necessary reinforcement for a concrete beam to resist an external factored moment of 290 K·ft (400 kN·m) if $b = 12$ in. (300 mm), $d = 19$ in. (475 mm), $d' = 2.5$ in. (65 mm), $f'_c = 3$ ksi (20 MPa), and $f_y = 60$ ksi (400 MPa).

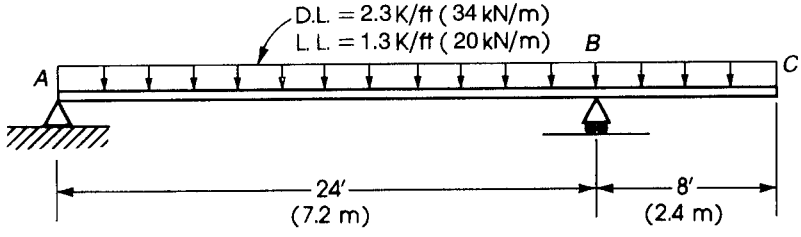


Figure 4.15 Problem 4.4.8.

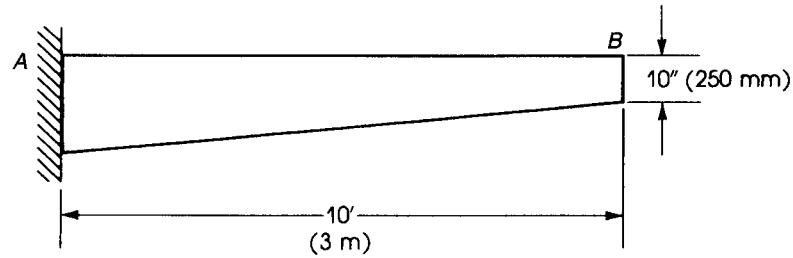


Figure 4.16 Problem 4.4.10.

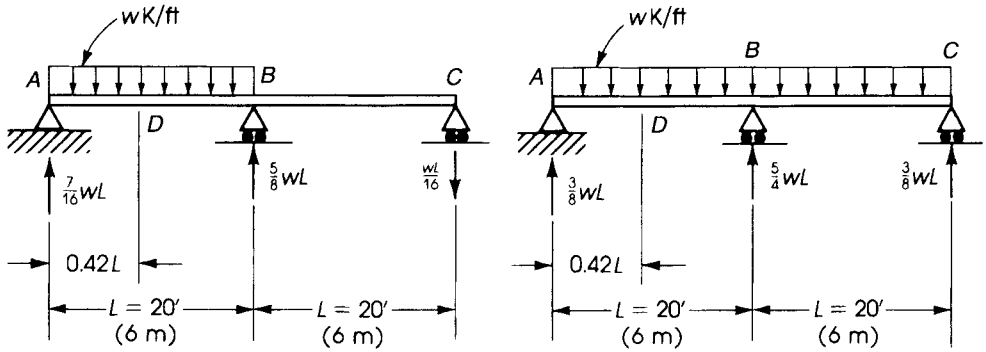


Figure 4.17 Problem 4.4.14.

4.12 Design a reinforced concrete section that can carry a factored moment of 260 K·ft (360 kN·m) as

a. Singly reinforced, $b = 10$ in. (250 mm)

b. Doubly reinforced, 25% of the moment to be resisted by compression steel, $b = 10$ in. (250 mm)

c. T-section, which has a flange thickness of 3 in. (75 mm), flange width of 20 in. (500 mm), and web width of 10 in. (250 mm)

$$f'_c = 3 \text{ ksi (20 MPa)}, \text{ and } f_y = 60 \text{ ksi (400 MPa)}, \text{ for all problems.}$$

Determine the quantities of concrete and steel designed per foot length (meter length) of beams and then determine the cost of each design if the price of the concrete equals \$50/yd^3 (67/m^3) and that of steel is \$0.30/lb (\$0.66/kg). Tabulate and compare results.

4.13 Determine the necessary reinforcement for a T-section that has a flange width of $b = 40$ in. (1000 mm), flange thickness of $t = 4$ in. (100 mm), and web width of $b_w = 10$ in. (250 mm) to carry a factored moment of 545 K·ft (750 kN·m). Given: $f'_c = 3$ ksi (20 MPa) and $f_y = 60$ ksi (400 MPa).

4.14 The two-span continuous beam shown in Fig. 4.17 is subjected to a uniform dead load of 2.6 K/ft (including its own weight) and a uniform live load of 3 K/ft. The reactions due to two different loadings are also shown. Calculate the maximum negative factored moment at the intermediate support B and the maximum positive factored moment within the span AB (at $0.42L$ from support A), design the critical section at B and D, and draw the reinforcement details for the entire beam ABC. Given: $L = 20$ ft, $b = 12$ in., $h = 24$ in. Use $d = 18$ in. for one row of bars and $d = 17$ in. for two rows. $f'_c = 4$ ksi, and $f_y = 60$ ksi.

4.15 The two-hinged frame shown in Fig. 4.18 carries a uniform dead load of 3.93 K/ft and a uniform live load of 2.4 K/ft on BC. The reactions at A and D can be evaluated as follows: $HA = HD = wL/9$ and $RA = RD = wL/2$, where w = uniform load on BC. A typical cross-section of the beam BC is also shown. It is required:

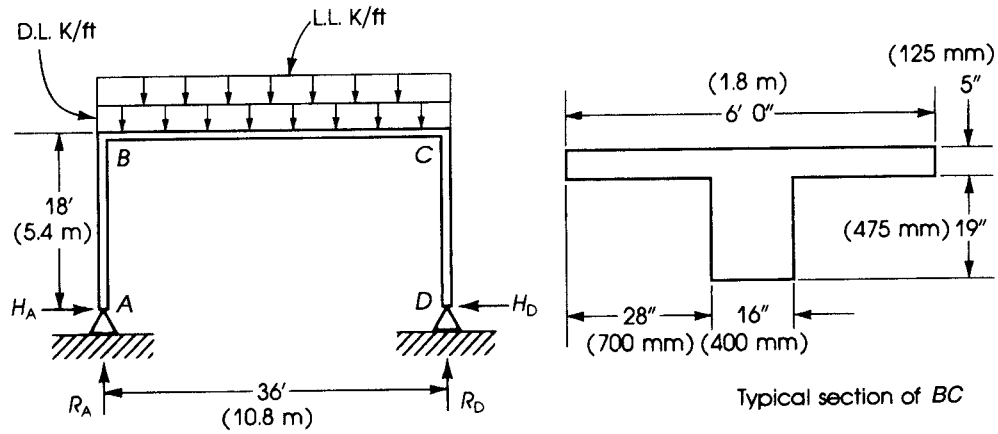


Figure 4.18 Problem 4.4.15.

a. Draw the bending moment diagram for the frame $ABCD$.

b. Design the beam BC for the applied factored moments (positive and negative).

c. Draw the reinforcement details of BC.

Given: $f'_c = 4$ ksi and $f_y = 60$ ksi.

CHAPTER 5

ALTERNATIVE DESIGN METHODS



Office building, Minneapolis, Minnesota.

5.1 INTRODUCTION

In the previous chapters, 3 and 4, the analysis and design of flexural reinforced concrete members were explained based on the provisions of the ACI Code 318-08. An alternative design approach is presented in Appendix B of the ACI Code according to the load factors given in Appendix C. This alternative design method was the basis of analysis and design in the ACI Code 318-99. It is to some extent similar to the method explained earlier except that it uses different load factors and strength reduction, ϕ. The basic analysis and design equations of the previous chapters will be used here. When Appendix B provisions are used in the design, they should replace all other corresponding provisions in the body of the Code.

5.2 LOAD FACTORS

If the required strength is denoted by U and those due to wind and seismic forces are W and E, respectively, then according to the ACI Code, Appendix C, the required strength U, shall be the most critical of the following:

1. In the case of dead, live, and wind loads,

$$U = 1.4D + 1.7L \tag{5.1a}$$

$$U = 0.75(1.4D + 1.7L) + (1.6W \text{ or } 1.0E) \tag{5.1b}$$

$$U = 0.9D + (1.6W \text{ or } 1.0E) \tag{5.1c}$$

2. When wind load, W, has not been reduced by a directionality factor, $1.3W$ can be used in place of $1.6W$. When seismic load is based on service forces, $1.4E$ can be used in place of $1.0E$.
3. In cases when earth pressure load, H, must be included in the design,

$$U = 1.4D + 1.7L + 1.7H \tag{5.2a}$$

Where dead load, D, and live load, L, reduce the effect of H, U shall be checked for

$$U = 0.9D + 1.7H \tag{5.2b}$$

For any combination of D, L, or H,

$$U = 1.4D + 1.7L$$

4. If weight and pressure loads from liquids, F, must be included in the design,

$$U = 1.4D + 1.7L + 1.4F \tag{5.3a}$$

Where dead load, D, and live load, L, reduce the effort of F,

$$U = 0.9D + 1.4F \tag{5.3b}$$

For any combination of D, L, or F,

$$U = 1.4D + 1.7L$$

The vertical pressure of liquids shall be considered as dead load with due regard to variation in liquid depth.

5. When impact effects are taken into account, they shall be included in the live load.
6. Where the structural effects, T, of differential settlement, creep, shrinkage, or temperature change may be significant, they shall be included with the most critical of

$$U = 0.75(1.4D + 1.4T + 1.7L) \tag{5.4a}$$

$$U = 1.4D + 1.4T \tag{5.4b}$$

Equation 5.1a is most generally used. The dead load factor is equal to 1.4, whereas the live load factor is equal to 1.7.

For applied concentrated dead and live loads, P_D and P_L, the factored concentrated load is $P_U = 1.4P_D + 1.7P_L$; also $M_U = 1.4M_D + 1.7M_L$, where M_D and M_L are the actual dead load and live load moments, respectively.

5.3 STRENGTH-REDUCTION FACTOR, ϕ

The nominal strength of a section is reduced by a factor ϕ to account for small adverse variations in material strengths, artisanry, dimensions, control, and degree of supervision. The factor ϕ constitutes a portion of the factor of safety, as discussed in Section 1.8.

The ACI Code, Section C.9.3 (Appendix C), specifies the following values to be used:

- Tension-controlled sections: $\phi = 0.90$
- Compression-controlled sections
  - Members with spiral reinforcement: $\phi = 0.75$
  - Other reinforced members: $\phi = 0.70$
- Shear and torsion: $\phi = 0.85$
- Bearing on concrete: $\phi = 0.70$
- Bending in plain concrete or in concrete with minimum reinforcement of $200/f_y$: $\phi = 0.65$

For sections that lie in the transition region between tension- and compression-controlled sections, ϕ may be increased linearly to 0.9.

Also, the strength eduction factor ϕ to be used for columns (or sections with $\varepsilon_t < 0.005$) may vary according to the following cases:

1. When $P_u = \phi P_n \geq 0.1 f'_c A_g$, then ϕ is 0.70 for tied columns and 0.75 for spirally reinforced columns. This case occurs generally when compression controls. A_g is the gross area of the concrete region.
2. Between values of $0.1 f'_c A_g$ or ϕP_b (whichever is smaller) and 0, P_u lies in the tension control zone and ϕ is larger than 0.7 (or 0.75). The ACI Code, Section C.9.3.2, specifies that for members in which f_y does not exceed 60 ksi, with symmetrical reinforcement and with the distance between compression and tension steel $(d - d')$ not less than $0.7h$ (h = total depth of section) and $d = h - d_s$, the value of ϕ is increased linearly to 0.9.

For this transition region, ϕ may be determined by linear interpolation between 0.7 (or 0.75) and 0.9. Figure 5.1 shows the variation of ϕ for grade 60 steel. The linear equations are as follows:

$$\phi = 0.57 + 67\varepsilon_t \qquad \text{(for tied sections)} \tag{5.5}$$

$$\phi = 0.65 + 50\varepsilon_t \qquad \text{(for spiral sections)} \tag{5.6}$$

Alternatively, ϕ in the transition region can be determined as a function of (d_t/c) for grade 60 steel as follows:

$$\phi = 0.37 + 0.20\left(\frac{d_t}{c}\right) \qquad \text{(for tied sections)} \tag{5.7}$$

$$\phi = 0.50 + 0.15\left(\frac{d_t}{c}\right) \qquad \text{(for spiral sections)} \tag{5.8}$$

where c is the depth of the neutral axis at nominal strength.

5.4 RECTANGULAR SECTIONS WITH TENSION REINFORCEMENT

From the analysis of rectangular singly reinforced section (Section 3.9), the following equations were derived, where f'_c and f_y are in ksi:

$$\rho_b = 0.85\beta_1 \frac{f'_c}{f_y}\left(\frac{87}{87 + f_y}\right) \tag{3.18}$$

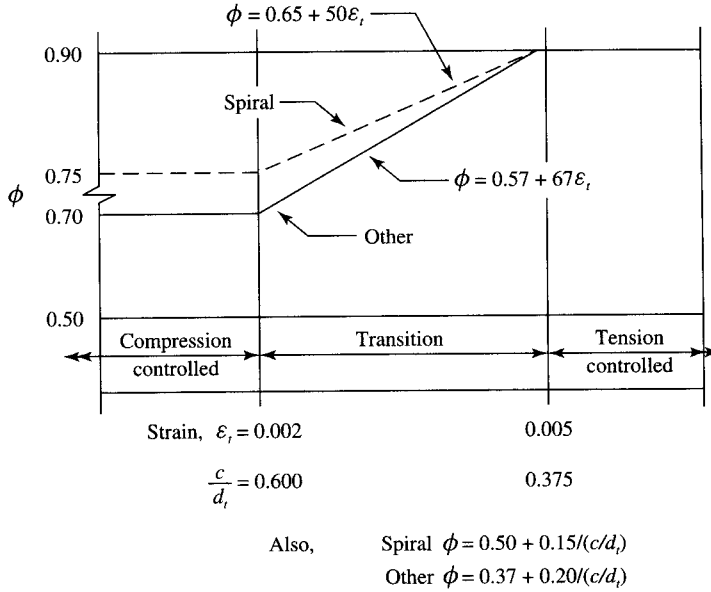


Figure 5.1 Variation of ϕ with the net tensile strain for grade 60 steel [1]. Courtesy of ACI 318-08.

If the maximum percentage of reinforcement is limited to $0.75\rho_b$, then

$$\rho_b = 0.75\rho_b = 0.6375\beta_1 \frac{f'_c}{f_y}\left(\frac{87}{87 + f_y}\right) \tag{5.9}$$

It is to be noted that $\rho_{max} = 0.75\rho_b$ is greater than that of $0.634\rho_b$ as given earlier in Chapter 3, (Eq. 3.30 for $f_y = 60$ ksi).

For $f'_c \leq 4000$ psi,

$$\rho_{max} = 0.542\frac{f'_c}{f_y}\left(\frac{87}{87 + f_y}\right) \tag{5.10}$$

The value of β_1 is 0.85 when $f'_c \leq 4000$ psi (30 N/mm^2) and decreases by 0.05 for every increase of 1000 psi (7 N/mm^2) in concrete strength, or $\beta_1 = 0.85 - 0.05(f'_c - 4) \geq 0.65$.

The steel percentage of a balanced section, ρ_b, and the maximum allowable steel percentage, ρ_{max}, can be calculated for different values of f'_c and f_y, as shown in Table 5.1. Suggested design steel ratios for $\rho \leq \rho_{max}$ are also shown in Table 5.1.

The design moment equations were derived in the previous chapter in the following forms:

$$\phi M_n = M_u = R_u b d^2 \tag{3.21}$$

where

$$R_u = \phi\rho f_y\left(1 - \frac{\rho f_y}{1.7f'_c}\right) = \phi R_n \tag{3.22}$$

Table 5.1 Suggested Design Steel Ratios ρ_s

f'_c (ksi)	f_y (ksi)	% ρ_b	% ρ_{max}	% ρ_s	Ratio ρ_s/ρ_b	Ratio ρ_s/ρ_{max}	R_u (psi)	$R_{u\,max}$ (psi)
3	40	3.71	2.78	1.4	0.38	0.50	450	783
	60	2.15	1.61	1.2	0.56	0.75	556	702
4	60	2.85	2.14	1.4	0.49	0.65	662	936
	75	2.07	1.55	1.2	0.58	0.77	702	867
5	60	3.36	2.52	1.4	0.42	0.56	681	1120
	75	2.44	1.83	1.2	0.49	0.66	722	1033

and $\phi = 0.9$. For tension-controlled sections, $\varepsilon_t \geq 0.005$:

$$\phi M_n = M_u = \phi A_s f_y \left(d - \frac{A_s f_y}{1.7 f'_c b}\right) \tag{3.19a}$$

Also,

$$\phi M_n = M_u = \phi \rho f_y b d^2 \left(1 - \frac{\rho f_y}{1.7 f'_c}\right) \tag{3.20}$$

We can see that for a given factored moment and known f'_c and f_y, there are three unknowns in these equations: the width, b, the effective depth of the section, d, and the steel ratio, ρ. A unique solution is not possible unless values of two of these three unknowns are assumed. Usually ρ is assumed (using ρ_{max}, for instance), and b can also be assumed.

Based on the preceding discussion, the following cases may develop for a given M_u, f'_c and f_y:

1. If ρ is assumed, then R_u can be calculated from Eq. 3.19, giving $bd^2 = M_u/R_u$. The designer may use ρ up to ρ_{max}, which produces the minimum size of the singly reinforced concrete section. Using ρ_{min} will produce the maximum concrete section. If b is assumed in addition to ρ, then d can be determined as follows:

$$d = \sqrt{\frac{M_u}{R_u b}} \tag{5.11}$$

If $d/b = 2$, then $d = \sqrt[3]{(2M_u/R_u)}$ and $b = d/2$, rounded to the nearest higher inch.

2. If b and d are given, the required reinforcement ratio, ρ, can be determined by rearranging Eq. 3.20 to obtain

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{4M_u}{1.7\phi f'_c b d^2}}\right] \tag{5.12}$$

$$= \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right]$$

and

$$A_s = \rho b d$$

where all units are in kips (or pounds) and inches. For example, if $M_u = 2440\,\text{K·in.}$, $b = 12\,\text{in.}$, $d = 18\,\text{in.}$, $f'_c = 3\,\text{ksi}$, and $f_y = 60\,\text{ksi}$, then $\rho = 0.01389$ (from Eq. 5.22) and $A_s = \rho bd = 0.01389(12)(18) = 3.0\,\text{in}^2$. When b and d are given, it is better to check if compression steel is or is not required because of a small d. This can be achieved as follows:

a. Calculate $\rho_{\max}$ and $R_{u\max} = \phi\rho_{\max} f_y[1 - (\rho_{\max} f_y/1.7f'_c)]$.

b. Calculate $\phi M_{n\max} = R_{u\max} bd^2 =$ the maximum moment strength of a singly reinforced concrete section.

c. If $M_u < \phi M_{n\max}$, then no compression reinforcement is needed. Calculate ρ and A_s from the preceding equations.

d. If $M_u > \phi M_{n\max}$, then compression steel is needed.

3. If ρ and b are given, calculate R_u:

$$R_u = \phi\rho f_y\left(1 - \frac{\rho f_y}{1.7f'_c}\right)$$

The calculate d from Eq. 5.21:

$$d = \sqrt{\frac{M_u}{R_u b}} \quad \text{and} \quad A_s = \rho bd$$

Example 5.1

Find the necessary reinforcement for a given section 10 in. wide and 28 in. total depth (Fig. 5.2) if it is subjected to an external factored moment of 245 K·ft. Given: $f'_c = 4$ ksi and $f_y = 60\,\text{ksi}$.

Solution

1. Assuming one layer of no. 8 steel bars (to be checked later), $d = 28 - 2.5\,\text{in.} = 25.5\,\text{in.}$
2. Check if the section is adequate without compression reinforcement. Compare design moment strength of the section (using $\rho_{\max}$) with the design moment. For $f'_c = 4$ ksi and $f_y = 60\,\text{ksi}$, $\rho_{\max} = 0.02138$.

$$R_u = \phi\rho_{\max} f_y\left(1 - \frac{\rho_{\max} f_y}{1.7f'_c}\right) = 937\ \text{psi}$$

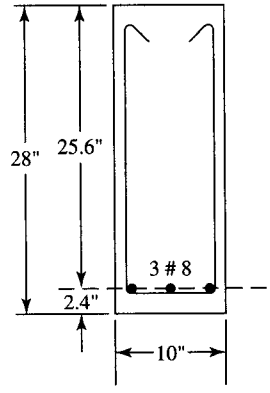


Figure 5.2 Example 5.1.

The design moment strength of a singly reinforced basic section is

$$\phi M_{n\max} = R_{u\max} bd^2 = 0.937(10)(25.5)^2$$
$$= 6093 \text{ K·in.} > 245 \times 12 = 2940 \text{ K·in.}$$

Therefore, $\rho < \rho_{\max}$ and the section is singly reinforced.

3. Calculate ρ from Eq. 5.12 to get $\rho = 0.009$. $A_s = \rho bd = 0.009(10)(25.5) = 2.30 \text{ in.}^2$ Use three no. 8 bars ($A_s = 2.35 \text{ in.}^2$) in one row, $b_{\min} < 10$ in. The final section is shown in Fig. 5.2.
4. Check ε_t:

$$a = \frac{2.35(60)}{0.85(4)(10)} = 4.15 \text{ in.}$$

$$c = \frac{a}{0.85} = 4.88 \text{ in.}$$

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003 = 0.0127 > 0.005 \quad \phi = 0.9$$

5.5 RECTANGULAR SECTIONS WITH COMPRESSION REINFORCEMENT

A singly reinforced section has a maximum design moment strength when $\rho_{\max}$ of steel is used. If the applied factored moment is greater than the internal moment strength, as in the case of a limited cross-section, a doubly reinforced section may be used, adding steel bars in both the compression and tension zones.

The procedure for designing a rectangular section with compressive steel when M_u, f'_c, b, d, and d' are given was summarized in Section 4.4. The only difference is that $\rho_{\max} = 0.75\,\rho_b$ is used in this design approach here.

$$\rho_{\max} = 0.6375\beta_1 \frac{f'_c}{f_y}\left(\frac{87}{87 + f_y}\right) \tag{5.9}$$

Also, check that $\varepsilon_t \geq 0.005$ for $\phi = 0.9$.

Example 5.2

A beam section is limited to $b = 12$ in. and to a total depth of $h = 20$ in. and is subjected to a factored moment of $M_u = 330$ K·ft. Determine the necessary reinforcement using $f'_c = 4$ ksi and $f_y = 60$ ksi. (Refer to Fig. 5.3)

Solution

1. Determine the design moment strength of the section as singly reinforced. Assume $\rho = 0.018$. Therefore, $R_u = 818$ psi (Table A2). For two rows of bars, $d = 20 - 3.5 = 16.5$ in.

$$M_{u1} = R_u bd^2 = 0.818(12)(16.5)^2 = 2672 \text{ K·in.}$$

The design moment is $M_u = 330 \times 12 = 3960$ K·in. $> M_{u1}$; therefore, compression steel is needed.

2. Calculate A_{s1}, M_{u2}, A_{s2}, and total A_s.

$$A_{s1} = \rho bd = 0.018(12)(16.5) = 3.56 \text{ in.}$$

$$M_{u2} = M_u - M_{u1} = 3960 - 2672 = 1288 \text{ K·in.}$$

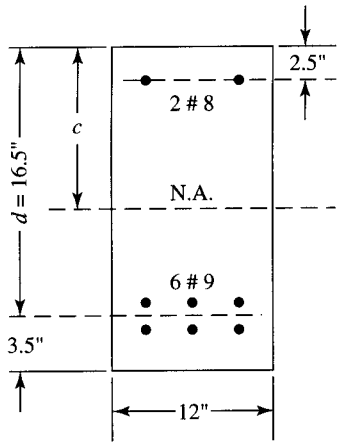


Figure 5.3 Example 5.2.

$$M_{u2} = \phi A_{s2} f_y (d - d'), \text{ assume } d' = 2.5 \text{ in.}$$

$$1288 = 0.9 A_{s2}(60)(16.5 - 2.5) \quad A_{s2} = 1.7 \text{ in.}^2$$

$$\text{Total } A_s = A_{s1} + A_{s2} = 3.56 + 1.7 = 5.26 \text{ in.}^2 \text{ (six no. 9 bars)}$$

3. Check if compression steel yields by Eq. 3.49. Compression steel yields if

$$\rho - \rho' \geq K = 0.85\beta_1 \frac{f'_c}{f_y}\left(\frac{d'}{d}\right)\left(\frac{87}{87 - f_y}\right)$$

$$K = (0.85)^2\left(\frac{4}{60}\right)\left(\frac{2.5}{16.5}\right)\left(\frac{87}{27}\right) = 0.0235$$

$$\rho - \rho' = \frac{A_{s1}}{bd} = \frac{3.56}{(12)(16.5)} = 0.018 \leq K$$

Therefore, compression steel does not yield: $f'_s < f_y$.

4. Calculate f'_s : $f'_s = 87[(c - d')/c] \leq f_y$. Determine c from A_{s1}: $A_{s1} = 3.56 \text{ in.}^2$

$$a = \frac{A_{s1} f_y}{0.85 f'_c b}$$

$$= \frac{3.56 \times 60}{0.85 \times 4 \times 12} = 5.24 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{5.24}{0.85} = 6.16 \text{ in.}$$

$$f'_s = 87 \times \left(\frac{6.16 - 2.5}{6.16}\right) = 52 \text{ ksi} < 60 \text{ ksi}$$

5. Calculate A'_s from $M_{u2} = \phi A'_s f'_s (d - d')$:

$$1288 = 0.9 A'_s (52)(16.5 - 2.5)$$

Thus, $A'_s = 1.97 \text{ in.}^2$, or calculate A'_s from $A'_s = A_{s2}(f_y/f'_s) = 1.97 \text{ in.}^2$ (two no. 9 bars).

6. Check

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003$$

$$d_t = h - 2.5 \text{ in.} = 17.5 \text{ in.}$$

$$\varepsilon_t = \left(\frac{17.5 - 6.16}{6.16}\right) 0.003$$

$$= 0.0055 > 0.005 \quad \phi = 0.9$$

or

$$\frac{c}{dt} = \frac{6.16}{17.5} = 0.352 < 0.375 \quad \text{(o.k.)}$$

7. Check final ϕM_n. $A_s = 6.0\,\text{in.}^2$, $A'_s = 2.0\,\text{in.}^2$, $A_{s1} = 4.0\,\text{in.}^2$, $a = 5.88\,\text{in.}$, and $c = 6.92\,\text{in.}$

$$M_n = 4 \times 60\left(16.5 - \frac{5.88}{2}\right) + 2 \times 52(16.5 - 2.5) = 4710 \text{ K·in.}$$

Check ε_t, $d_t = 17.5\,\text{in.}$

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003 = 0.0459 < 0.005$$

$$\phi = 0.57 + 67(0.0459) = 0.88$$

$$\phi M_n = 4145 \text{ K·in.} > M_u = 3960 \text{ K·in.}$$

5.6 DESIGN OF T-SECTIONS

In the design of a T-section for a given factored moment, M_u, the flange thickness t and width b would have been already established from the design of the slab and the ACI Code limitations for the effective flange width b, as given in Section 3.14. The web thickness, b_w, can be assumed to vary between 8 in. and 20 in., with a practical width of 12 to 16 in. Two more unknowns still need to be determined, d and A_s. The design procedure was summarized in Section 4.5.

Example 5.3

The T-beam section shown in Fig. 5.4 has a web width, b_w, of 10 in., a flange width, b, of 40 in., a flange thickness of 4 in., and an effective depth, d, of 14.5 in. Determine the necessary reinforcement if the applied factored moment is 3800 K·in. Given: $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. Check the position of the neutral axis; the section may be rectangular. Assume the depth of compression block a is 4 in.; that is, $a = t = 4\,\text{in.}$ Then

$$\phi M_n = \phi(0.85 f'_c) bt \left(d - \frac{t}{2}\right) = 6120 \text{ K·in.} > M_u = 3800 \text{ K·in.}$$

The design moment that the concrete flange can resist is greater than the factored moment applied. Therefore, the section behaves as a rectangular section.

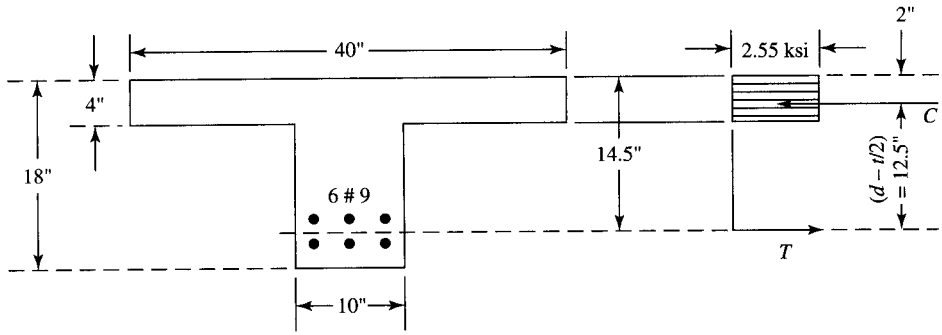


Figure 5.4 Example 5.3: T-section.

2. Determine the area of tension steel, considering a rectangular section, $b = 40$ in.

$$R_u = \frac{M_u}{(bd^2)} = \frac{3{,}800{,}000}{40 \times 14.5^2} = 452 \text{ psi}$$

From Eq. 5.22, for $R_u = 452$ psi, $\rho = 0.0091$.

$$A_s = \rho bd = 0.0091 \times 40 \times 14.5 = 5.28 \text{ in.}^2$$

Use six no. 9 bars, $A_s = 6.00$ in.^{2} (in two rows).

3. Check that $\rho_w = A_s/b_w\, d \geq \rho_{\min}$; $\rho_w = 5.28/(10 \times 14.5) = 0.0364 > \rho_{\min} = 0.00333$.
4. Check

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003 \qquad d_t = 14.5$$

$$a = \frac{5.28(60)}{0.85 \times 4 \times 40} = 2.33 \text{ in.} \qquad c = 2.74 \text{ in.}$$

$$\varepsilon_t = 0.0129 > 0.005 \qquad \phi = 0.9$$

Note that other examples will be similar to those in Chapters 3 and 4.

5.7 STRUT AND TIE METHOD

5.7.1 Introduction

The ACI Code, Appendix A, introduces an alternative approach to the method explained earlier in Chapter 3, called the strut and tie models. This alternative method can be applied effectively in regions of discontinuity in the structural member, such as support areas, zones of load application, or areas with sudden change in the geometrical dimensions as brackets and portal frames. In these regions, the plane sections do not remain plane after bending (as was assumed in Chapter 3, Section 3.2), and they are called D-regions (Fig. 5.5*a*). The other regions of a standard beam, the basic beam theory, and a linear strain relationship apply. These regions are called B-regions (Fig. 5.5*a*).

The discontinuity in the stress distribution in region D (due to geometry or loading condition), based on St. Venants principles, indicates that the stresses due to axial load and bending approach a linear distribution at a distance approximately equal to the height of the member, h,

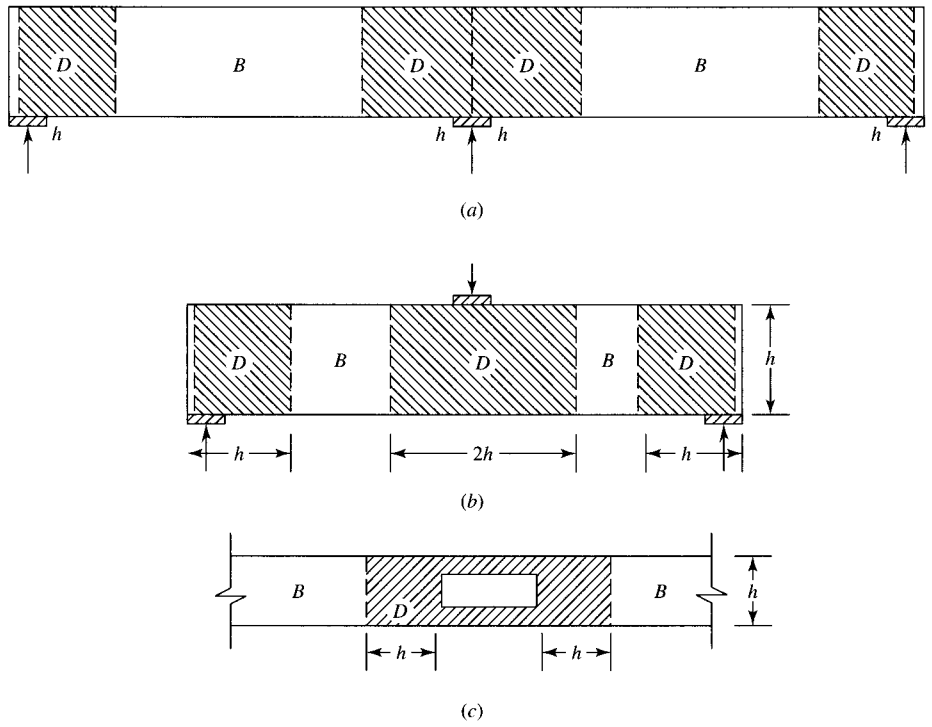


Figure 5.5 D- and B-regions in beam. (*a*) Continuous beam, (*b*) beam with concentrated load, (*c*) beam with an opening [1]. Courtesy of ACI 318-05.

away from the discontinuity (Fig. 5.5*b* and *c*) [1]. If two D-regions overlap or meet, they can be considered a single D-region. The maximum length to depth ratio would be equal to two, producing a minimum angle of 26.5° (tan $\frac{1}{2}$) between the strut and tie (or approximately 25°).

In a strut and tie model (Fig. 5.6), the point where the three forces meet at joint D is called a node, and the volume of concrete around a node is called a nodal zone. Forces at a node can vary between different combinations of compression and tensile forces, C-C-C, C-C-T, C-T-T, or T-T-T (Fig. 5.7). Figure 5.8 shows typical nodal zones for different load applications, while Fig. 5.9 shows extended nodal zones for one or more layers of reinforcing bars [6].

5.7.2 Strut and Tie Models

A strut and tie model can be represented by an idealized truss model with forces acting at the different nodes. Now consider the steel truss shown in Fig. 5.10. Due to symmetry, the reactions at A and B are equal, $R_A = R_B$ 20 K, and from the equilibrium of joints A and D, the tensile force in AB = 20 K, while the compressive force in AD or BD = 28.3 K. Member AB is considered a tie, while AD and BD act as struts. The forces in the other members are equal to 0. Comparing this truss with concrete beam in Fig. 5.6*a*, it can be seen that most of the areas ACD and BED and below the nodal zone D are not effective and act as fillers. The forces in the struts, for this loading condition, are greater than the force in the tie. In this case, adequate concrete areas are available to act as idealized struts (Fig. 5.6*a*). Steel reinforcement is needed to act as a tie for AB. Proper anchoring of the ties are essential for a safe design and should be anchored in a nodal zone.

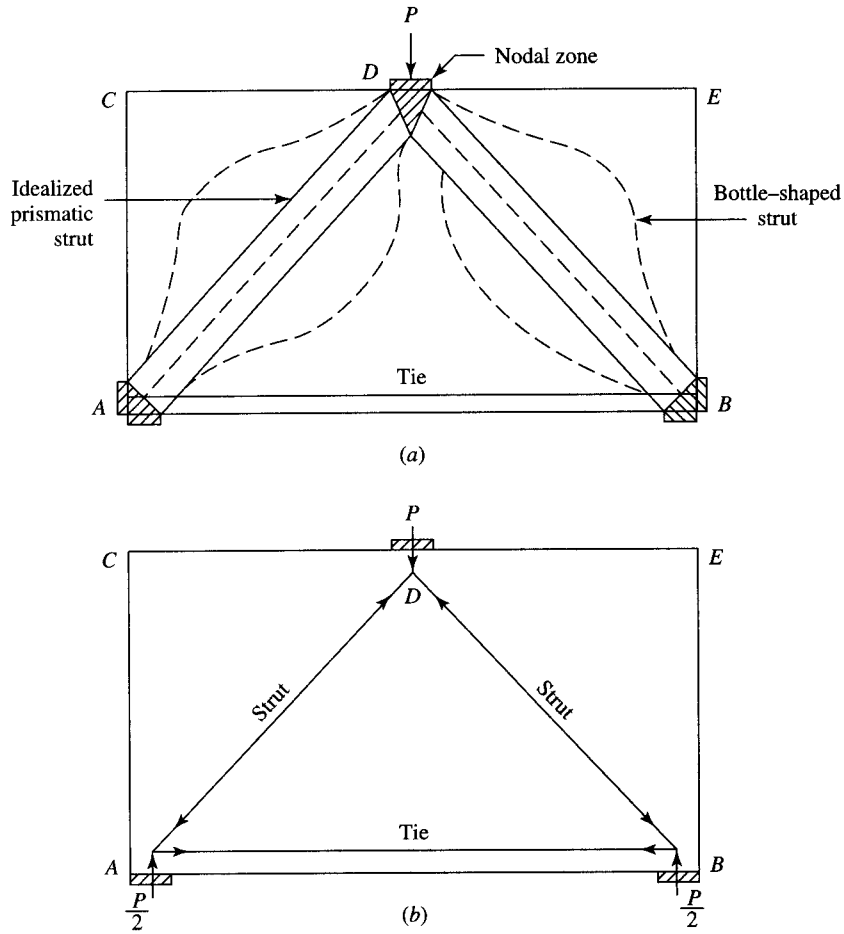


Figure 5.6 (*a*) Strut and tie model, (*b*) idealized model [1]. Courtesy of ACI 318-05.

5.7.3 ACI Design Procedure

Based on the ACI Code, Section A.2, the design of a D-region includes the following steps [1]:

1. Define and isolate each region.
2. Determine the resultant forces acting on each D-region boundary.
3. Select a truss model to transfer the resultant forces across the D-region. The axes of the struts and ties should coincide, approximately, with the compression and tension fields.
4. Determine the effective widths of the struts and nodal zones based on the concrete and steel strengths and the truss model chosen.
5. Check serviceability conditions according to the ACI Code requirements. Deflections of deep beams can be estimated using an elastic analysis. Crack control conditions of the ACI Code, Section 10.6.4, should be checked assuming the tie is encased in a prism of concrete according to RA.4.2.

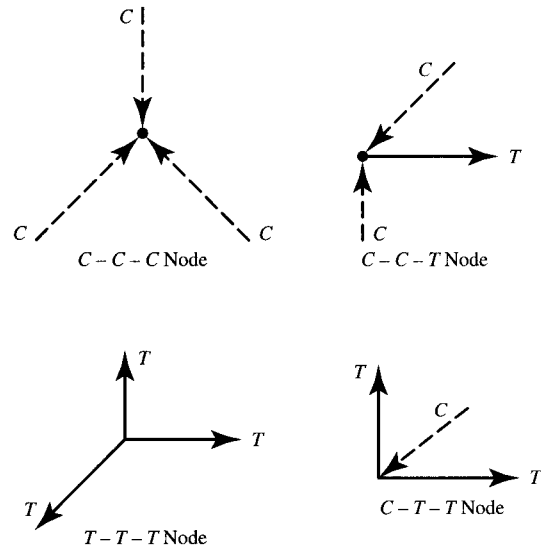


Figure 5.7 Classification of nodes.

5.7.4 Design Requirements

The design requirements for struts and ties can be summarized as follows:

1. Design of struts, ties, and nodal zones:

$$\phi F_n \geq F_u \tag{5.13}$$

where

F_u = force in a strut, tie, or nodal zone due to factored loads
F_n = nominal strength of a strut, tie, or nodal zone
ϕ = 0.75 for both struts and ties

2. Strength of struts: The nominal compressive strengths of a strut without longitudinal reinforcement, F_{ns}, shall be the smaller value of F_{ns} at the two ends of the strut such that:

$$F_{ns} = f_{ce} A_{cs} \tag{5.14}$$

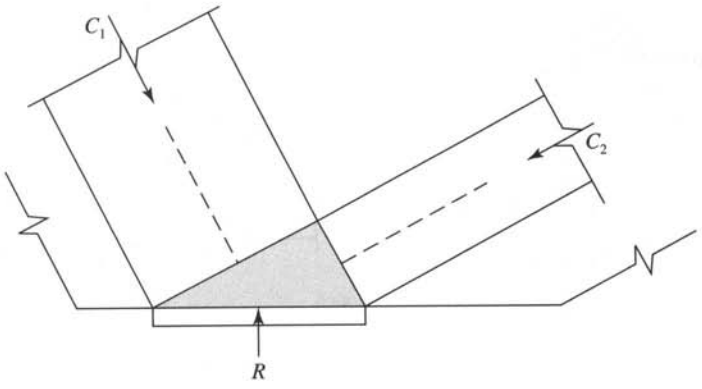
where

A_{cs} = cross-sectional area at one end of a strut
f_{ce} = the smaller effective compressive strength of concrete in a strut or nodal zone

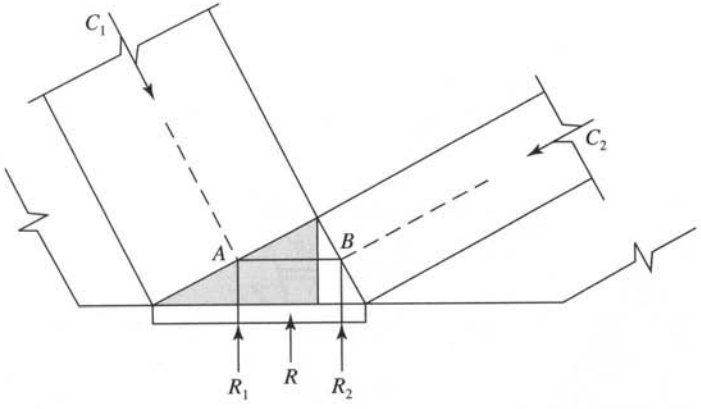
$$f_{ce} = 0.85\beta_s f'_c \tag{5.15}$$

where

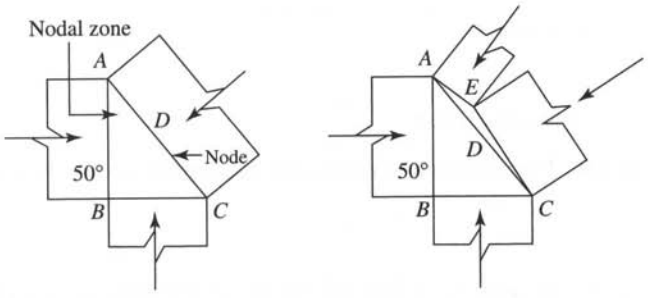
a. β_s = 1.0 for a strut of uniform cross section (ACI A.3.2.1)
b. β_s = 0.4 for struts in tension members, or the tension flanges of members (ACI A.3.2.3)



(*a*) Nodal zone



(*b*) Subdivided nodal zone



(*c*) Three struts acting on a nodal zone

(*d*) Struts *AE* and *CE* may be replaced by *AC*

Figure 5.8 Nodal zones [1]. (*a*, *b*) Subdivision of nodal zone, (*c*, *d*) resolution of forces on a nodal zone.

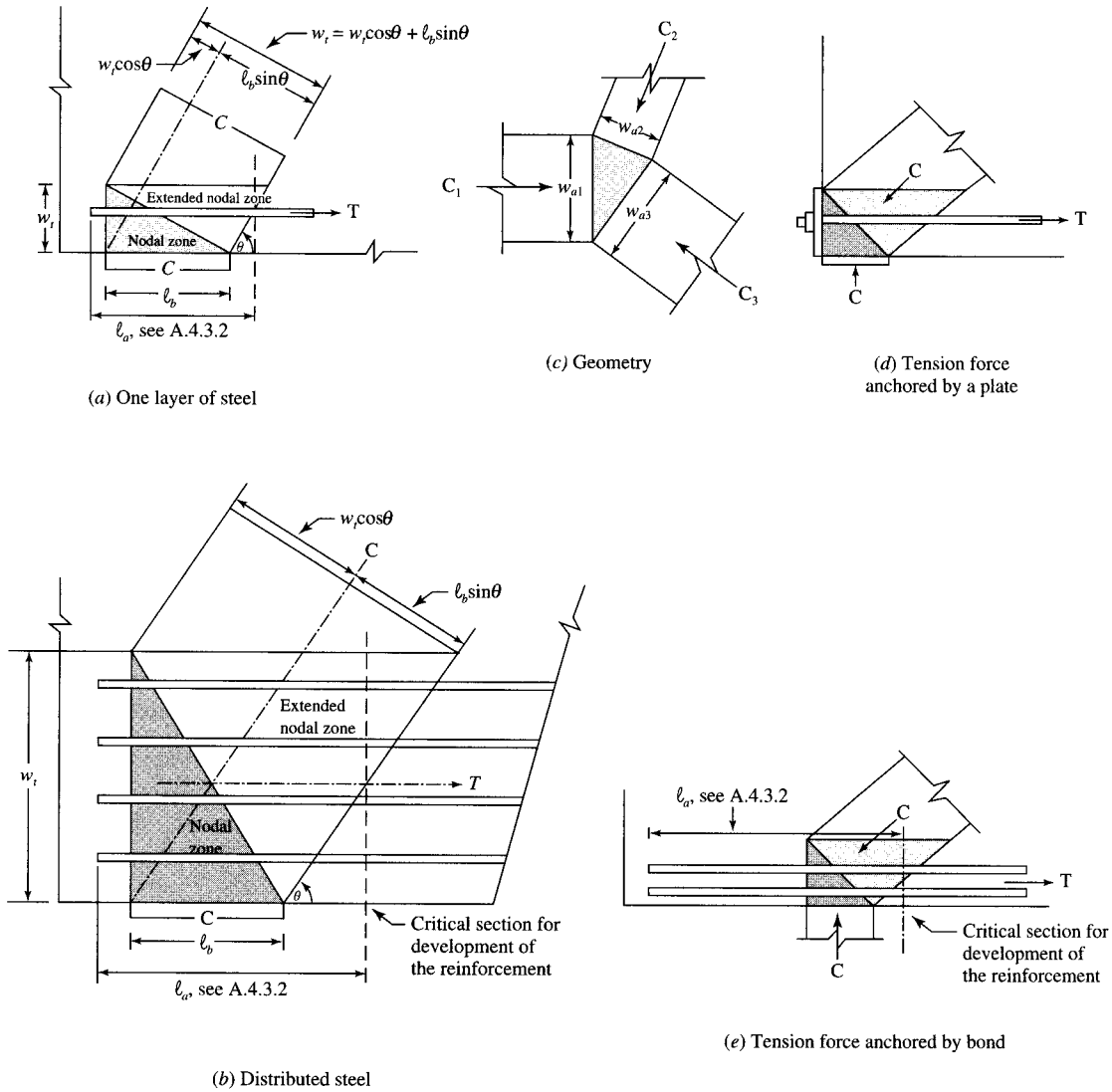


Figure 5.9 Extended nodal zones and hydrostatic nodes [1]. Courtesy of ACI 318-05.

c. For struts located such that the width of the midsection of the strut is larger than the width of the nodes (bottle-shaped struts) (ACI 3.2.2):
$\beta_s = 0.75$ with reinforcement satisfying ACI A.3.3
$\beta_s = 0.6\lambda$ without reinforcement satisfying ACI A.3.3

d. $\beta_s = 0.6\lambda$ for all other cases (ACI 3.2.4)

$\lambda = 1.00$ normal-weight concrete

$= 0.85$ sand lightweight concrete

$= 0.75$ for all other lightweight concrete

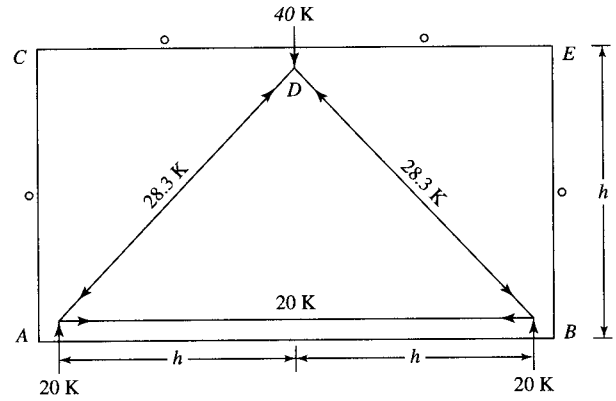


Figure 5.10 Example of a steel truss.

Linear interpolation between 0.75 and 0.85 shall be permitted, on the basis of volumetric fraction, when a portion of the lightweight fine aggregate is replaced with normal-weight fine aggregate. Linear interpolation between 0.85 and 1.0 shall be permitted for concrete containing normal-weight fine aggregate and a blend of light- and normal-weight coarse aggregate.

3. Reinforcement crossing struts (Fig. 5.11): For $f'_c \leq 6$ ksi, the value $\beta_s = 0.75$ can be used if the axis of the strut is crossed by layers of bars such that

$$\sum \frac{A_{si}}{b_s s_i} \sin \gamma_i \geq 0.003 \tag{5.16}$$

where

A_{si} = total area of surface reinforcement at a spacing s_i in the ith layer crossing a strut with reinforcement at an angle α_i to the axis of the strut

s_i = spacing of reinforcement in the ith layer crossing a strut at an angle α_i to the axis of the strut member

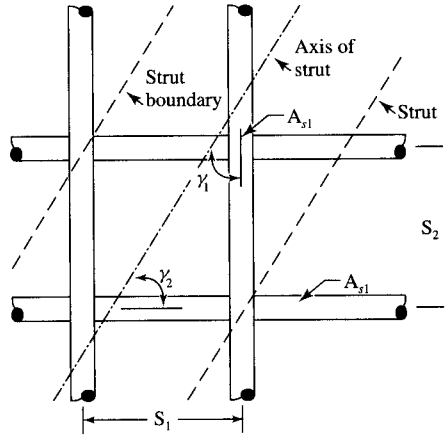


Figure 5.11 Reinforcing bars crossing a strut [1]. Courtesy of ACI 318-08.

b_s = width of member

α_i = angle between the axis of the strut and the bars in ith layer of bars crossing the strut

4. Compression reinforcement in struts: Compression reinforcement can be used to increase the strength of a strut such that

$$F_{ns} = f_{ce}A_{cs} + A'_s f'_s \tag{5.17}$$

where

F_{ns} = strength of a longitudinal reinforced strut

A'_s = area of the compression reinforcement in a strut

f'_s = stress in A'_s ($f'_s = f_y$ for grades 40 to 60)

5. Strength of ties: The nominal strength of a tie, F_{nt} is:

$$F_{nt} = A_{ts}f_y + A_{tp}(f_{se} + \Delta f_p) \tag{5.18}$$

where

A_{ts} = area of nonprestressed reinforcement in the tie

A_{tp} = area of prestressing reinforcement

f_{se} = effective stress after losses in prestressed reinforcement

Δf_p = increase in prestressing stress due to factored loads

A_{tp} = 0 for nonprestressed members

$$(f_{se} + \Delta f_p) \leq f_{py} \tag{5.19}$$

It is permitted to take $\Delta f_p = 60$ ksi for bonded prestressed reinforcement or 10 ksi for unbonded prestressed reinforcement. Also, a practical upper limit of the tie width, $w_{t,max}$ can be taken as follows:

$$w_{t,max} = F_{nt}/(f_{ce}b_s) \tag{5.20}$$

6. Strength of nodal zones: The nominal compression strength of a nodal zone, F_{nn}, shall be

$$F_{nn} = f_{ce}A_{nz} \tag{5.21}$$

where A_{nz} = the area of the face of the nodal zone or a section through a nodal zone perpendicular to the resultant force on the section.

7. Confinement in nodal zones: Unless confining reinforcement is provided within the nodal zone and its effect is supported by tests and analysis, the calculated effective compressive stress on a face of a nodal zone due to the strut and tie forces should not exceed the following:

$$f_{ce} = 0.85\beta_n f'_c \tag{5.22}$$

where

β_n = 1.0 in nodal zones bounded by struts or bearing areas, or both, C-C-C node.

β_n = 0.80 in nodal zones anchoring one tie, C-C-T node.

β_n = 0.60 in nodal zones anchoring two or more ties, C-T-T node.

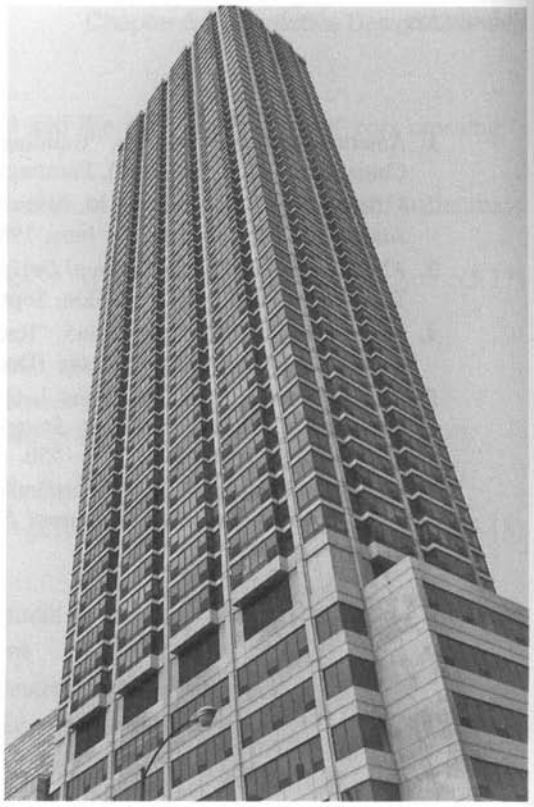
The application of the strut and tie method to a deep beam is given in Example 8.6, Section 8.11.

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CHAPTER 6

DEFLECTION AND CONTROL OF CRACKING



High-rise building, Chicago, Illinois.

6.1 DEFLECTION OF STRUCTURAL CONCRETE MEMBERS

Flexural concrete members must be designed for safety and serviceability. The members will be safe if they are designed according to the ACI Code equations and limitations. Consequently, as explained in previous chapters, the size of each member is determined as well as the reinforcement required to maintain an internal moment capacity equal to or greater than that of the external moment. Once the final dimensions are determined, the beam must be checked for serviceability: cracks and deflection. Adequate stiffness of the member is necessary to prevent excessive cracks and deflection.

The use of the ACI Code provisions, taking into consideration the nonlinear relationship between stress and strain in concrete, has resulted in smaller sections than those designed by the elastic theory. The ACI Code, Section 9.4, recognizes the use of steel up to a yield strength of 80 ksi (560 MPa) and the use of high-strength concrete. The use of high-strength steel and concrete results in smaller sections and a reduction in the stiffness of the flexural member and consequently increases its deflection.

The permissible deflection is governed by many factors, such as the type of the building, the appearance of the structure, the presence of plastered ceilings and partitions, the damage expected due to excessive deflection, and the type and magnitude of live load.

The ACI Code, Section 9.5, specifies minimum thickness for one-way flexural members and one-way slabs, as shown in Table 6.1 in this chapter. The values are for members not supporting or attached to partitions or other constructions likely to be damaged by large deflections.

Table 6.1 Minimum Thickness of Beams and One-Way Slabs (L = Span Length)

Member	Yield Strength f_y (ksi)	Simply Supported	One End Continuous	Both Ends Continuous	Cantilever
Solid one-way slabs	40	$L/25$	$L/30$	$L/35$	$L/12.5$
	50	$L/22$	$L/27$	$L/31$	$L/11$
	60*	$L/20$	$L/24$	$L/28$	$L/10$
Beams or ribbed one-way slabs	40	$L/20$	$L/23$	$L/26$	$L/10$
	50	$L/18$	$L/20.5$	$L/23.5$	$L/9$
	60*	$L/16$	$L/18.5$	$L/21$	$L/8$

*Values reported in ACI Table 9.5(a).

The minimum thicknesses indicated in Table 6.1 are used for members made of normal-weight concrete, and for steel reinforcement with yield strengths as mentioned in the table. The values are modified for cases of lightweight concrete or a steel yield strength different from 60 ksi as follows:

- For lightweight concrete having unit weights in the range of 90 to 115 pcf, the values in the tables for $f_y = 60$ ksi (420 MPa) shall be multiplied by the greater of $(1.65 - 0.005 W_c)$ but not less than 1.09, where W_c is the unit weight of concrete in pounds per cubic foot.
- For yield strength of steel different from 60 ksi (420 MPa), the values in the tables for 60 ksi shall be multiplied by $(0.4 + f_y/100)$, where f_y is in ksi.

6.2 INSTANTANEOUS DEFLECTION

The deflection of structural members is due mainly to the dead load plus a fraction of or all the live load. The deflection that occurs immediately upon the application of the load is called the *immediate*, or *instantaneous, deflection*. Under sustained loads, the deflection increases appreciably with time. Various methods are available for computing deflections in statically determinate and indeterminate structures. The instantaneous deflection calculations are based on the elastic behavior of the flexural members. The elastic deflection, Δ, is a function of the load, W, span, L, moment of inertia, I, and the modulus of elasticity of the material, E:

$$\Delta = f\left(\frac{WL}{EI}\right) = \alpha\left(\frac{WL^3}{EI}\right) = K\left(\frac{ML^2}{EI}\right) \tag{6.1}$$

where W = total load on the span and α and K are coefficients that depend on the degree of fixity at the supports, the variation of moment of inertia along the span, and the distribution of load. For example, the maximum deflection on a uniformly loaded simply supported beam is

$$\Delta = \frac{5WL^3}{384EI} = \frac{5wL^4}{384EI} \tag{6.2}$$

where W = the total load on the span = wL (uniform load per unit length × span). Deflections of beams with different loadings and different end conditions as a function of the load, span, and EI are given in Appendix C and in books of structural analysis.

Because W and L are known, the problem is to calculate the modulus of elasticity, E, and the moment of inertia, I, of the concrete member or the flexural stiffness of the member EI.

6.2.1 Modulus of Elasticity

The ACI Code, Section 8.5,specifies that the modulus of elasticity of concrete, E_c, may be taken as

$$E_c = 33W_c^{1.5}\sqrt{f'_c} \text{ psi} \tag{6.3}$$

for values of W_c between 90 and 160 pcf. For normal-weight concrete ($W_c = 145$ pcf),

$$E_c = 57{,}400\sqrt{f'_c} \text{ psi} \quad (\text{or } 57{,}000\sqrt{f'_c})$$

The modulus of elasticity is usually determined by the short-term loading of a concrete cylinder. In actual members, creep due to sustained loading, at least for the dead load, affects the modulus on the compression side of the member. For the tension side, the modulus in tension is assumed to be the same as in compression when the stress magnitude is low. At high stresses the modulus decreases appreciably. Furthermore, the modulus varies along the span due to the variation of moments and shear forces.

6.2.2 Modular Ratio

The modular ratio, $n = Es/E_c$, which is used in the transformed area concept was explained in Section 2.10. It may be used to the nearest whole number but may not be less than 6. For example,

when $f'_c = 2500$ psi (17.5 MPa), $n = 10$

when $f'_c = 3000$ psi (20 MPa), $n = 9$

when $f'_c = 4000$ psi (30 MPa), $n = 8$

when $f'_c = 5000$ psi (17.5 MPa), $n = 7$

For normal-weight concrete, n may be taken as $500/\sqrt{f'_c}$, (psi units).

6.2.3 Cracking Moment

The behavior of a simply supported structural concrete beam loaded to failure was explained in Section 3.3. At a low load, a small bending moment develops, and the stress at the extreme tension fibers will be less than the modulus of rupture of concrete, $f_r = 7.5\lambda\sqrt{f'_c}$. If the load is increased until the tensile stress reaches an average stress of the modulus of rupture, f_r, cracks will develop. If the tensile stress is higher than f_r, the section will crack, and a cracked section case will develop. This means that there are three cases to be considered:

1. When the tensile stress, f_t, is less than f_r, the whole-uncracked section is considered to calculate the properties of the section. In this case, the gross moment of inertia, I_g, is used: $I_g = bh^3/12$, where bh = the whole concrete section.
2. When the tensile stress, f_t, is equal to the modulus of rupture, $f_r = 7.5\lambda\sqrt{f'_c}$, a crack may start to develop, and the moment that causes this stress is called the cracking moment. Using the flexural formula;

$$f_r = M_{\text{cr}}\frac{c}{I_g} \quad \text{or} \quad M_{\text{cr}} = f_r \cdot \frac{I_g}{c} \tag{6.4}$$

where $f_r = 7.5\lambda\sqrt{f'_c}$, I_g = the gross moment of inertia, and c = the distance from the neutral axis to the extreme tension fibers. For example, for a rectangular section, $I_g = bh^3/12$ and $c = h/2$, and where

λ is a modification factor for type of concrete (ACI 8.6.1)

$\lambda = 1.0$ Normal-weight concrete

$\lambda = 0.85$ Sand-lightweight concrete

$\lambda = 0.75$ For all-lightweight concrete

Linear interpolation shall be permitted between 0.85 and 1.0 on the basis of volumetric fractions, for concrete containing normal-weight fine aggregate and a blend of lightweight and normal-weight coarse aggregate.

3. When the applied external moment exceeds the cracking moment, M_{cr}, a cracked section case is developed, and the concrete in the tension zone is neglected. A transformed cracked section is used to calculate the cracking moment of inertia, I_{cr}, using the concrete area in compression and the transformed steel area nA_s.

Example 6.1

A rectangular concrete section is reinforced with three no. 9 bars in one row and has a width of 12 in., a total depth of 25 in., and $d = 22.5$. (Fig. 6.1. Calculate the modulus of rupture, f_r, the gross moment of inertia, I_g, and the cracking moment, M_{cr}. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. The modulus of rupture is $f_r = 7.5\lambda\sqrt{f'c} = 7.5 \times 1 \times \sqrt{4000} = 474$ psi. ($\lambda = 1$ normal-weight concrete)
2. The gross moment of inertia for a rectangular section is

$$bh^3/12 = \frac{12(25)^3}{12} = 15{,}625 \text{ in.}^4$$

3. The cracking moment is $M_{cr} = f_r \cdot I_g/c$

$$f_r = 474 \text{ psi} \qquad I_g = 15{,}625 \text{ in.}^4 \qquad c = h/2 = 12.5 \text{ in.}$$

Therefore, $M_{cr} = 474 \times 15{,}625/(12.5 \times 1000) = 592.5$ K·in. $= 49.38$ K·ft

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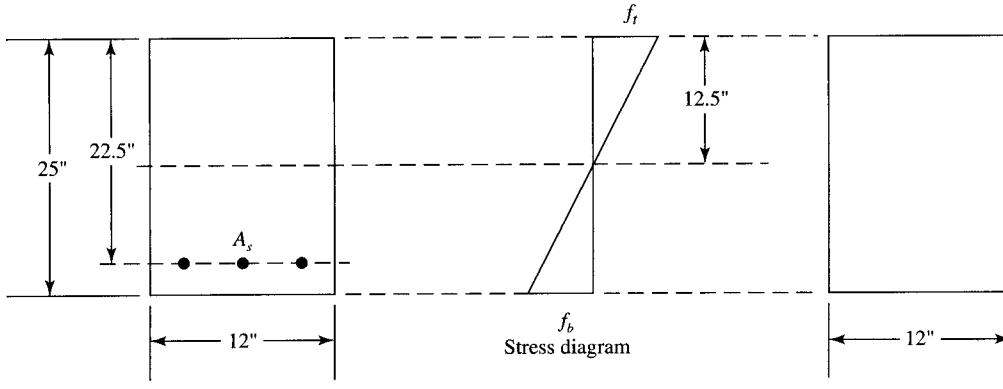


Figure 6.1 Example 6.1.

6.2.4 Moment of Inertia

The moment of inertia, in addition to the modulus of elasticity, determines the stiffness of the flexural member. Under small loads, the produced maximum moment will be small, and the tension stresses at the extreme tension fibers will be less than the modulus of rupture of concrete; in this case, the gross transformed cracked section will be effective in providing the rigidity. At working loads or higher, flexural tension cracks are formed. At the cracked section, the position of the neutral axis is high, whereas at sections midway between cracks along the beam, the position of the neutral axis is lower (nearer to the tension steel). In both locations only the transformed cracked sections are effective in determining the stiffness of the member; therefore, the effective moment of inertia varies considerably along the span. At maximum bending moment, the concrete is cracked, and its portion in the tension zone is neglected in the calculations of moment of inertia. Near the points of inflection the stresses are low, and the entire section may be uncracked. For this situation and in the case of beams with variable depth, exact solutions are complicated.

Figure 6.2*a* shows the load–deflection curve of a concrete beam tested to failure. The beam is a simply supported 17-ft span and loaded by two concentrated loads 5 ft apart, symmetrical about the centerline. The beam was subjected to two cycles of loading: In the first (curve *cy* 1), the load–deflection curve was a straight line up to a load $P = 1.7$ K when cracks started to occur in the beam. Line *a* represents the load–deflection relationship using a moment of inertia for the uncracked transformed section. It can be seen that the actual deflection of the beam under loads less than the cracking load, based on a homogeneous uncracked section, is very close to the calculated deflection (line *a*). Curve *cy* 1 represents the actual deflection curve when the load is increased to about one-half the ultimate load. The slope of the curve, at any level of load, is less than the slope of line *a* because cracks developed, and the cracked part of the concrete section reduces the stiffness of the beam. The load was then released, and a residual deflection was observed at midspan. Once cracks developed, the assumption of uncracked section behavior under small loads did not hold.

In the second cycle of loading, the deflection (curve *c*) increased at a rate greater than that of line *a*, because the resistance of the concrete tension fibers was lost. When the load was increased, the load–deflection relationship was represented by curve *cy* 2. If the load in the first cycle is increased up to the ultimate load, curve *cy* 1 will take the path *cy* 2 at about 0.6 of the ultimate load. Curve *c* represents the actual behavior of the beam for any additional loading or unloading cycles.

Line *b* represents the load–deflection relationship based on a cracked transformed section; it can be seen that the deflection calculated on that basis differs from the actual deflection. Figure 6.2*c* shows the variation of the beam stiffness *EI* with an increase in moment. ACI Code, Section 9.5, presents an equation to determine the effective moment of inertia used in calculating deflection in flexural members. The effective moment of inertia given by the ACI Code (Eq. 9.8) is based on the expression proposed by Branson [3] and calculated as follows:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g \tag{6.5}$$

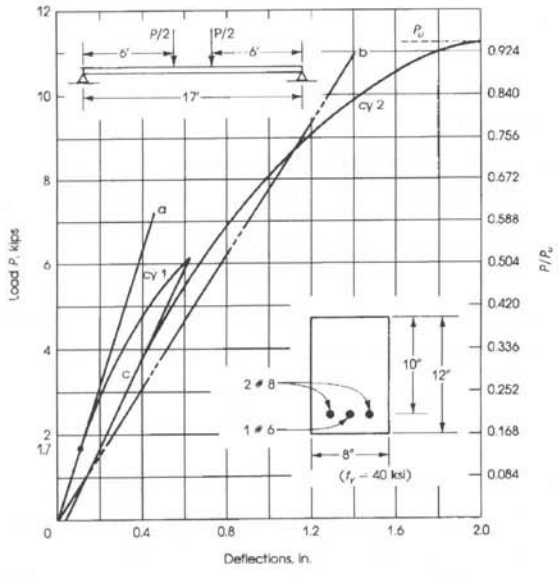
where

I_e = effective moment of inertia

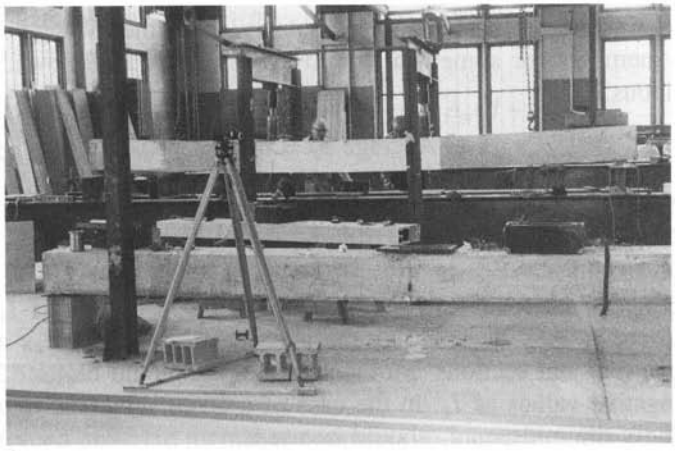
M_{cr} = cracking moment, $\left(\frac{f_r I_g}{Y_t}\right)$ (6.6)

f_r = modulus of rupture of concrete

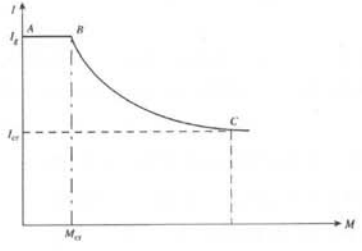
$= 7.5\lambda\sqrt{f'_c}$ psi, $(0.623\lambda\sqrt{f'_c}$ MPa) (6.7)



(*a*)



(*b*)



(*c*)

Figure 6.2 (*a*) Experimental and theoretical load—deflection curves for a beam of the section and load illustrated, (*b*) deflection of a reinforced concrete beam, and (*c*) variation of beam moment of inertia, I, with an increase in moment (E = constant). BC is a transition curve between I_g and I_{cr}.

M_a = maximum unfactored moment in member at stage for which deflection is being computed

I_g = moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement

I_{cr} = moment of inertia of cracked transformed section

Y_t = distance from centroidal axis of cross-section, neglecting steel, to the tension face

The following limitations are specified by the code:

1. For continuous spans, the effective moment of inertia may be taken as the average of the moment of inertia of the critical positive and negative moment sections.
2. For prismatic members, I_e may be taken as the value obtained from Eq. 6.5 at midspan for simple and continuous spans and at the support section for cantilevers (ACI Code, Section 9.5.2).

   Note that I_e, as computed by Eq. 6.5, provides a transition between the upper and lower bounds of the gross moment of inertia, I_g, and the cracked moment of inertia, I_{cr}, as a function of the level of M_{cr}/M_a. Heavily reinforced concrete members may have an effective moment of inertia, I_e, very close to that of a cracked section, I_{cr}, whereas flanged members may have an effective moment of inertia close to the gross moment of inertia, I_g.
3. For continuous beams, an approximate value of the average I_e for prismatic or nonprismatic members for somewhat improved results is as follows: For beams with both ends continuous,

$$\text{Average } I_e = 0.70I_m + 0.15(I_{e1} + I_{e2}) \tag{6.8}$$

For beams with one end continuous,

$$\text{Average } I_e = 0.85I_m + 0.15(I_{con}) \tag{6.9}$$

where I_m = midspan I_e, I_{e1}, I_{e2} = I_e at beam ends, and I_{con} = I_e at the continuous end. Also, I_e may be taken as the average value of the I_es at the critical positive- and negative-moment sections. Moment envelopes should be used in computing both positive and negative values of I_e. In the case of a beam subjected to a single heavy concentrated load, only the midspan I_e should be used.

6.2.5 Properties of Sections

To determine the moment of inertia of the gross and cracked sections, it is necessary to calculate the distance from the compression fibers to the neutral axis (x or kd).

1. Gross moment of inertia, I_g (neglect all steel in the section)
   a. For a rectangular section of width b and a total depth h, $I_g = bh^3/12$.
   b. For a T-section, flange width b, web width b_w, and flange thickness t, calculate y, the distance to the centroidal axis from top of flange:

$$y = \frac{\left(\frac{bt^2}{2}\right) + b_w(h-t)\left[\frac{(h-t)}{2+t}\right]}{bt + b_w(h-t)} \tag{6.10}$$

Then calculate I_g:

$$I_g = \left[\frac{bt^3}{12} + bt\left(y - \frac{t}{2}\right)^2\right] + \left[b_w\frac{(y-t)^3}{3}\right] + \left[b_w\frac{(h-y)}{3}\right] \tag{6.10a}$$

2. Cracked moment of inertia, I_{cr}: Let x = the distance of the neutral axis from the extreme compression fibers ($x = kd$).
   - **a.** Rectangular section with tension steel, A_s, only
     - **i.** Calculate x from the following equation:

$$\frac{bx^2}{2} - nA_s(d-x) = 0 \tag{6.11}$$

     - **ii.** Calculate $I_{cr} = bx^3/3 + nA_s\,(d-x)^2$ (6.11a)
   - **b.** Rectangular section with tension steel A_s and compression steel A'_s
     - **i.** Calculate x:

$$\frac{bx^2}{2} + (n-1)A'_s(x-d') - nA_s(d-x) = 0 \tag{6.12}$$

     - **ii.** Calculate $I_{cr} = (bx^3/3) + (n-1)A'_s(x-d')^2 + nA_s(d-x)^2$. (6.12a)
   - **c.** T-sections with tension steel A_s
     - **i.** Calculate x : $bt\left(x - \frac{t}{2}\right) + b_w\frac{(x-t)^2}{2} - nA_s(d-x) = 0$ (6.13)
     - **ii.** Calculate I_{cr}:

$$I_{cr} = \left[\frac{bt^3}{12} + bt\left(x - \frac{t}{2}\right)^2\right] + \left[b_w\frac{(x-t)^3}{3}\right] + nA_s(d-x)^2 \tag{6.13a}$$

6.3 LONG-TIME DEFLECTION

Deflection of reinforced concrete members continues to increase under sustained load, although more slowly with time. Shrinkage and creep are the cause of this additional deflection, which is called long-time deflection [1]. It is influenced mainly by temperature, humidity, age at time of loading, curing, quantity of compression reinforcement, and magnitude of the sustained load. The ACI Code, Section 9.5.2.5, suggests that unless values are obtained by a more comprehensive analysis, the additional long-term deflection for both normal and lightweight concrete flexural members shall be obtained by multiplying the immediate deflection by the factor

$$\lambda_\Delta = \frac{\zeta}{1 + 50\rho'} \tag{6.14}$$

where

λ_Δ = multiplier for additional deflection due to long-term effect.

ρ' = A'_s/bd for the section at midspan of a simply supported or continuous beam or at the support of a cantilever beam

ζ = time-dependent factor for sustained loads that may be taken as shown in Table 6.2.

Table 6.2 Multipliers for Long-time Deflections

Period (months)	1	3	6	12	24	36	48	60 &over
ζ	0.5	1.0	1.2	1.4	1.7	1.8	1.9	2.0

The factor λ_Δ is used to compute deflection caused by the dead load and the portion of the live load that will be sustained for a sufficient period to cause significant time-dependent deflections. The factor λ_Δ is a function of the material property, represented by ζ, and the section property, represented by $(1 + 50\rho')$. In Eq. 6.14, the effect of compression reinforcement is related to the area of concrete rather than the ratio of compression to tension steel.

The ACI Code Commentary, Section 9.5.2.5, presents a curve to estimate ζ for periods less than 60 months. These values are estimated as shown in Table 6.2.

The total deflection is equal to the immediate deflection plus the additional long-time deflection. For instance, the total additional long-time deflection of a flexural beam with $\rho' = 0.01$ at a 5-year period is equal to λ_Δ times the immediate deflection, where $\lambda_\Delta = 2/(1 + 50 \times 0.01) = 1.33$.

6.4 ALLOWABLE DEFLECTION

Deflection shall not exceed the following values according to the ACI Code, Section 9.5:

- $L/180$ for immediate deflection due to live load for flat roofs not supporting elements that are likely to be damaged
- $L/360$ for immediate deflection due to live load for floors not supporting elements likely to be damaged
- $L/480$ for the part of the total deflection that occurs after attachment of elements, that is, the sum of the long-time deflection due to all sustained loads and the immediate deflection due to any additional live load, for floors or roofs supporting elements likely to be damaged
- $L/240$ for the part of the total deflection occurring after elements are attached, for floors or roofs not supporting elements likely to be damaged

6.5 DEFLECTION DUE TO COMBINATIONS OF LOADS

If a beam is subjected to different types of loads (uniform, nonuniform, or concentrated loads) or subjected to end moments, the deflection may be calculated for each type of loading or force applied on the beam separately and the total deflection calculated by superposition. This means that all separate deflections are added up algebraically to get the total deflection. The deflections of beams under individual loads are shown in Table 6.3.

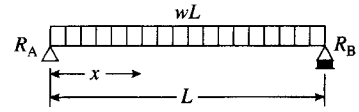
Example 6.2

Calculate the instantaneous midspan deflection for the simply supported beam shown in Fig. 6.3, which carries a uniform dead load of 0.4 K/ft and a live load of 0.6 K/ft in addition to a concentrated dead load of 5 kips at midspan. Given: $f'_c = 4$ ksi normal-weight concrete, $f_y = 60$ ksi, $b = 13$ in., $d = 21$ in., and total depth $= 25$ in. $(n = 8)$.

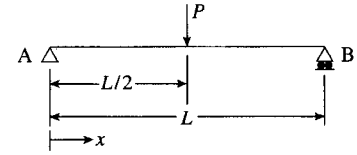
Table 6.3 Deflection of Beams

$$\Delta_{max} = \frac{5}{384} \times \frac{WL^3}{EI} \quad \text{(at center)}$$

W = total load = wL



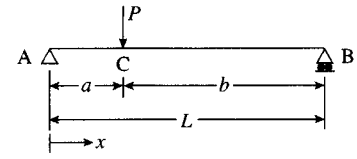
$$\Delta_{max} = \frac{PL^3}{48EI} \quad \text{(at midspan)}$$



$$\Delta_C = \frac{Pa^2b^2}{3EIL} \quad \text{(at point load)}$$

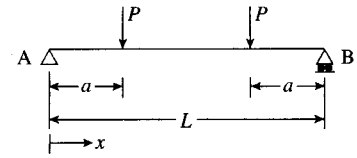
$$\Delta_{max} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right] \quad \text{(when } a \geq b\text{)}$$

at $x = \sqrt{a(b + L)/3}$



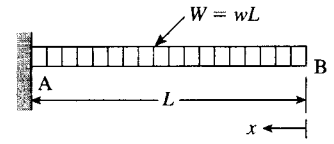
$$\Delta_{max} = \frac{PL^3}{6EI}\left[\frac{3a}{4L} - \left(\frac{a}{L}\right)^3\right] \quad \text{(at midspan)}$$

$$\Delta_{max} = \frac{23PL^3}{648EI} \quad \text{(at midspan) when } a = L/3$$



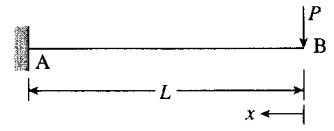
$$\Delta_B max = \frac{WL^3}{8EI} \quad (W = wL)$$

$$\Delta_x = \frac{w}{24EIL}(x^4 - 4L^3x + 3L^4)$$



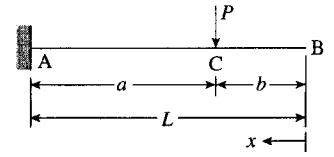
$$\Delta_B max = \frac{PL^3}{3EI}$$

$$\Delta_x = \frac{P}{6EI}(2L^3 - 3L^2x + x^3)$$



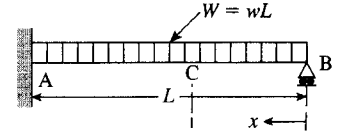
$$\Delta_C = Pa^3/3EI$$

$$\Delta_B max = \frac{Pa^3}{3EI}\left(1 + \frac{3b}{2a}\right) \quad \text{(at free end)}$$

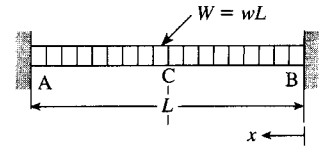


$$\Delta_{max} = \frac{WL^3}{185EI}$$

at a distance $x = 0.4215L$ (from support B)



$$\Delta_{max} = \frac{WL^3}{384EI} \quad \text{(at midspan)}$$



$$\Delta_1 = \frac{5wL^4}{384E_cI_e}$$

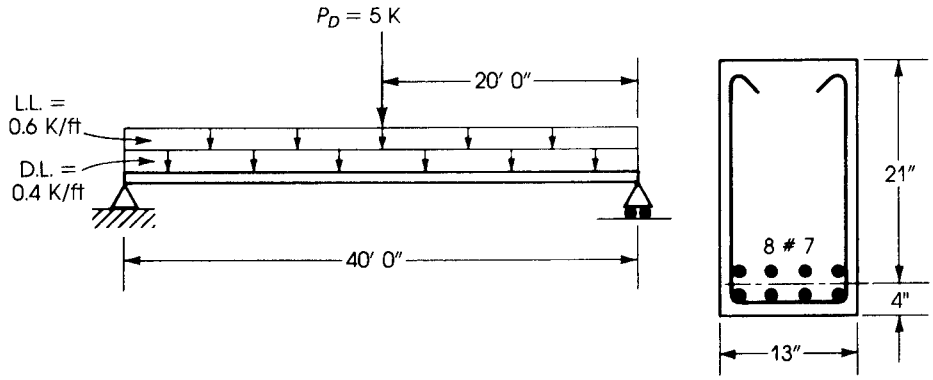


Figure 6.3 Example 6.2.

Solution

1. Check minimum depth according to the ACI Code, Table 6.1.

$$\text{Minimum total depth} = \frac{L}{16} = \frac{40 \times 12}{16} = 30 \text{ in.}$$

The total thickness used in 25 in. < 30 in.; therefore, deflection must be checked.

2. The deflection at midspan due to a distributed load is

$$\Delta_1 = \frac{5wL^4}{384E_cI_e}$$

The deflection at midspan due to a concentrated load is

$$\Delta_2 = \frac{PL^3}{48E_cI_e}$$

Because w, P, and L are known, we must determine the modulus of elasticity, E_c, and the effective moment of inertia, I_e.

3. The modulus of elasticity of concrete is

$$E_c = 57{,}400\sqrt{f'_c} = 57{,}400\sqrt{4000} = 3.63 \times 10^6 \text{psi}$$

4. The effective moment of inertia is equal to

$$\Delta_1 = \frac{5wL^4}{384E_cI_e}$$

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g$$

Determine values of all terms on the right-hand side:

$$M_a = \frac{wL^2}{8} + \frac{PL}{4} = \frac{(0.6 + 0.4)}{8}(40)^2 \times 12 + \frac{5 \times 40}{4} \times 12 = 3000 \text{ K·in.}$$

$$I_g = \frac{bh^3}{12} = \frac{13(25)^3}{12} = 16{,}927 \text{ in.}^4$$

$$M_{cr} = \frac{f_r I_g}{Y_t} \qquad Y_t = \frac{h}{2} = 12.5 \text{ in.} \qquad f_r = 7.5\lambda\sqrt{f'_c} = 474 \text{ psi} \quad \lambda = 1 \text{ (normal-weight)}$$

$$M_{cr} = \frac{0.474 \times 16{,}927}{12.5} = 642 \text{ K·in.}$$

The moment of inertia of the cracked transformed area, I_{cr}, is calculated as follows: Determine the position of the neutral axis for a cracked section by equating the moments of the transformed area about the neutral axis to 0, letting $x = kd$ = distance to the neutral axis:

$$\frac{bx^2}{2} - nA_s(d - x) = 0 \quad n = \frac{E_s}{E_c} = 8.0 \quad A_s = 4.8 \text{ in.}^2$$

$$\frac{13}{2}x^2 - (8)(4.8)(21 - x) = 0$$

$$x^2 + 5.9x - 124 = 0 \quad x = 8.8 \text{ in.}$$

$$I_{cr} = \frac{bx^3}{3} + nA_s(d - x)^2 = \frac{13(8.8)^3}{3} + 38.4(21 - 8.8)^2 = 8660 \text{ in.}^4$$

With all terms calculated,

$$I_e = \left(\frac{642}{3000}\right)^3 \times 16{,}927 + \left[1 - \left(\frac{642}{3000}\right)^3\right] \times 8660 = 8740 \text{ in.}^4$$

5. Calculate the deflections from the different loads:

$$\Delta_1(\text{due to distributed load}) = \frac{5wL^4}{384E_cI_e}$$

$$\Delta_1 = \left(\frac{5}{384}\right) \times \left(\frac{1000}{12}\right) \times \frac{(40 \times 12)^4}{3.63 \times 10^6 \times 8740} = 1.82 \text{ in.}$$

$$\Delta_2(\text{due to concentrated load}) = \frac{PL^3}{48E_cI_e}$$

$$\Delta_2 = \frac{5000 \times (40 \times 12)^3}{48 \times 3.63 \times 10^6 \times 8740} = 0.36 \text{ in.}$$

Total immediate deflection $= \Delta_1 + \Delta_2 = 1.82 + 0.36 = 2.18$ in.

6. Compare the calculated values with the allowable deflection: The immediate deflection due to a uniform live load of 0.6 K/ft is equal to $0.6(1.82) = 1.09$ in. If the member is part of a floor construction not supporting or attached to partitions or other elements likely to be damaged by large deflection, the allowable immediate deflection due to live load is equal to

$$\frac{L}{360} = \frac{40 \times 12}{360} = 1.33 \text{ in.} > 1.09 \text{ in.}$$

If the member is part of a flat roof and similar to the preceding, the allowable immediate deflection due to live load is $L/180 = 2.67$ in. Both allowable values are greater than the actual deflection of 1.09 in. due to the uniform applied live load.

Example 6.3

Determine the long-time deflection of the beam in Example 6.2 if the time-dependent factor equals 2.0.

Solution

1. The sustained load causing long-time deflection is that due to dead load, consisting of a distributed uniform dead load of 0.4 K/ft and a concentrated dead load of 5 K at midspan.

$$\text{Deflection due to uniform load} = 0.4 \times 1.82 = 0.728 \text{ in.}$$

Deflection is a linear function of load, w, all other values (L, E_c, I_e) being the same.

$$\text{Deflection due to concentrated load} = 0.36 \text{ in.}$$

$$\text{Total immediate deflection due to sustained loads} = 0.728 + 0.36$$

$$= 1.088 \text{ in.}$$

2. For additional long-time deflection, the immediate deflection is multiplied by the factor λ_Δ:

$$\lambda_\Delta = \frac{\zeta}{1 + 50\rho'} = \frac{2}{1 + 0}$$

In this problem, $A_s' = 0$; therefore, $\lambda_\Delta = 2.0$.

$$\text{Additional long-time deflection} = 2 \times 1.088 = 2.176 \text{ in.}$$

3. Total long-time deflection is the immediate deflection plus additional long-time deflection: 2.18 + 2.176 = 4.356 in.
4. Deflection due to dead load plus additional long-time deflection due to shrinkage and creep is 1.088 + 2.176 = 3.264 in.

Example 6.4

Calculate the instantaneous and 1-year long-time deflection at the free end of the cantilever beam shown in Fig. 6.4. The beam has a 20-ft span and carries a uniform dead load of 0.4 K/ft, a uniform live load of 0.4 K/ft, a concentrated dead load, P_D, of 3 K at the free end, and a concentrated live load, P_L, of 4 K placed at 10 ft from the fixed end. Given: $f_c' = 4$ ksi, $f_y = 60$ ksi, $b = 12$ in., $d = 21.5$ in., and total depth of section = 25 in. (Tension steel is six no. 8 bars and compression steel is two no. 8 bars.). Assume normal-weight concrete.

Solution

1. Minimum depth = $L/8 = \frac{20}{8} = 2.5$ ft = 30 in., which is greater than the 25 in. used. Therefore, deflection must be checked. The maximum deflection of a cantilever beam is at the free end. The deflection at the free end is as follows.

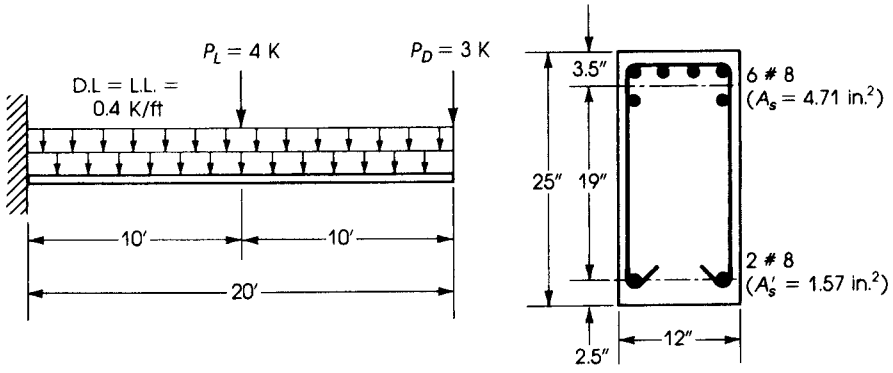


Figure 6.4 Example 6.4.

Deflection due to distributed load:

$$\Delta_1 = \frac{wL^4}{8EI}$$

Deflection due to a concentrated dead load at the free end:

$$\Delta_2 = \frac{P_D L^3}{3EI}$$

Deflection due to concentrated live load at $a = 10\,\text{ft}$ from the fixed end is maximum at the free end:

$$\Delta_3 = \frac{P_L(a)^2}{6EI}(3L - a) \quad \text{or} \quad \frac{Pa^3}{3EI}\left(1 + \frac{3b}{2a}\right)$$

2. The modulus of elasticity of normal-weight concrete is

$$E_c = 57{,}400\sqrt{f'_c} = 57{,}400\sqrt{4000} = 3.63 \times 10^6 \text{ psi}$$

3. Maximum moment at the fixed end is

$$M_a = \frac{wL^2}{2} + P_D \times 20 + P_L \times 10$$

$$= \frac{(0.4 + 0.4)(400)}{2} + 3 \times 20 + 4 \times 10 = 260 \text{ K·ft}$$

4. I_g = gross moment of inertia (concrete only)

$$= \frac{bh^3}{12} = \frac{12 \times (25)^3}{12} = 15{,}625 \text{ in.}^4$$

5. $$M_{cr} = \frac{f_r I_g}{Y_t} = \frac{((7.5)(1)\sqrt{4000}) \times 15{,}625}{\frac{25}{2}} = 592.9 \text{ K·in.} = 49.40 \text{ K·ft}$$

6. Determine the position of the neutral axis; then determine the moment of inertia of the cracked transformed section. Take moments of areas about the neutral axis and equate them to 0. Use $n = 8$ to calculate the transformed area of A_s and use $(n - 1) = 7$ to calculate the transformed area of A'_s. Let $kd = x$.

$$b\frac{(x^2)}{2} + (n - 1)A'_s(x - d') - nA_s(d - x) = 0$$

For this section, $x = 8.44\,\text{in.}$

$$I_{cr} = \frac{b}{3}x^3 + (n - 1)A'_s(x - d')^2 + nA_s(d - x)^2 = 9220 \text{ in.}^4$$

7. Effective moment of inertia is

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g$$

$$= \left(\frac{49.40}{260}\right)^3 \times 15{,}625 + \left[1 - \left(\frac{49.40}{260}\right)^3\right] \times 9220 = 9264 \text{ in.}^4$$

8. Determine the components of the deflection:

$$\Delta_1 \text{ (due to uniform load of 0.8K/ft)} = \frac{800}{12} \times \frac{(20 \times 12)^4}{8 \times 3.63 \times 10^6 \times 9264} = 0.82 \text{ in.}$$

$$\Delta_1 \text{ (due to dead load)} = 0.82 \times \frac{0.4}{0.8} = 0.41 \text{ in.}$$

$$\Delta_2 \text{ (due to concentrated dead load) at free end} = \frac{3000(20 \times 12)^3}{3 \times 3.63 \times 10^6 \times 9264} = 0.41 \text{ in.}$$

$$\Delta_3 \text{ (due to concentrated live load at 10 ft from fixed end)} = \frac{4000(10 \times 12)^2 \times (3 \times 20 \times 12 - 10 \times 12)}{6 \times 3.63 \times 10^6 \times 9264} = 0.17 \text{ in.}$$

The total immediate deflection is

$$\Delta = \Delta_1 + \Delta_2 + \Delta_3 = 0.82 + 0.41 + 0.17 = 1.40 \text{ in.}$$

9. For additional long-time deflection, the immediated deflection is multiplied by the factor λ_Δ. For a 1-year period, $\zeta = 1.4$.

$$\rho' = \frac{A_s'}{bd} = \frac{1.57}{12 \times 21.5} = 0.0061$$

$$\lambda_\Delta = \frac{1.4}{1 + 50 \times 0.0061} = 1.073$$

Total immediate deflection Δ_s due to sustained load (here only the dead load of 0.4 K/ft and $P_D = 3$ K at free end): $\Delta_s = (0.41 + 0.41) = 0.82$ in. Additional long-time deflection $= 1.073 \times 0.82 = 0.88$ in.

10. Total long-time deflection is the immediate deflection plus long-time deflection due to shrinkage and creep.
Total $\Delta = 1.40 + 0.88 = 2.28$ in.

Example 6.5

Calculate the instantaneous midspan deflection of beam *AB* in Fig. 6.5, which has a span of 32 ft. The beam is continuous over several supports of different span lengths. The absolute bending moment diagram and cross-sections of the beam at midspan and supports are also shown. The beam carries a uniform dead load of 4.2 K/ft and a live load of 3.6 K/ft. Given: $f_c' = 3$ ksi normal-weight concrete, $f_y = 60$ ksi, and $n = 9.2$.

Moment at midspan:	$M_D = 192$ K·ft	$M_{(D+L)} = 480$ K·ft
Moment at left support *A*:	$M_D = 179$ K·ft	$M_{(D+L)} = 420$ K·ft
Moment at right support *B*:	$M_D = 216$ K·ft	$M_{(D+L)} = 542$ K·ft

Solution

1. The beam *AB* is subjected to a positive moment that causes a deflection downward at midspan and negative moments at the two ends, causing a deflection upward at midspan. As was explained earlier, the deflection is a function of the effective moment of inertia, I_e. In a continuous beam, the value of I_e to be used is the average value for the positive and negative moment regions. Therefore, three sections will be considered: the section at midspan and the sections at the two supports.
2. Calculate I_e: For the gross area of all sections, $kd = 13.5$ in. and $I_g = 114{,}300$ in.^{4} Also, $f_r = 7.5\lambda\sqrt{f_c'} = 410$ psi and $E_e = 57{,}400\sqrt{f_c'} = 3.15 \times 10^6$ for all sections. The values of kd, I_{cr}, and M_{cr} for each cracked section, I_e for dead load only (using M_a of dead load), and

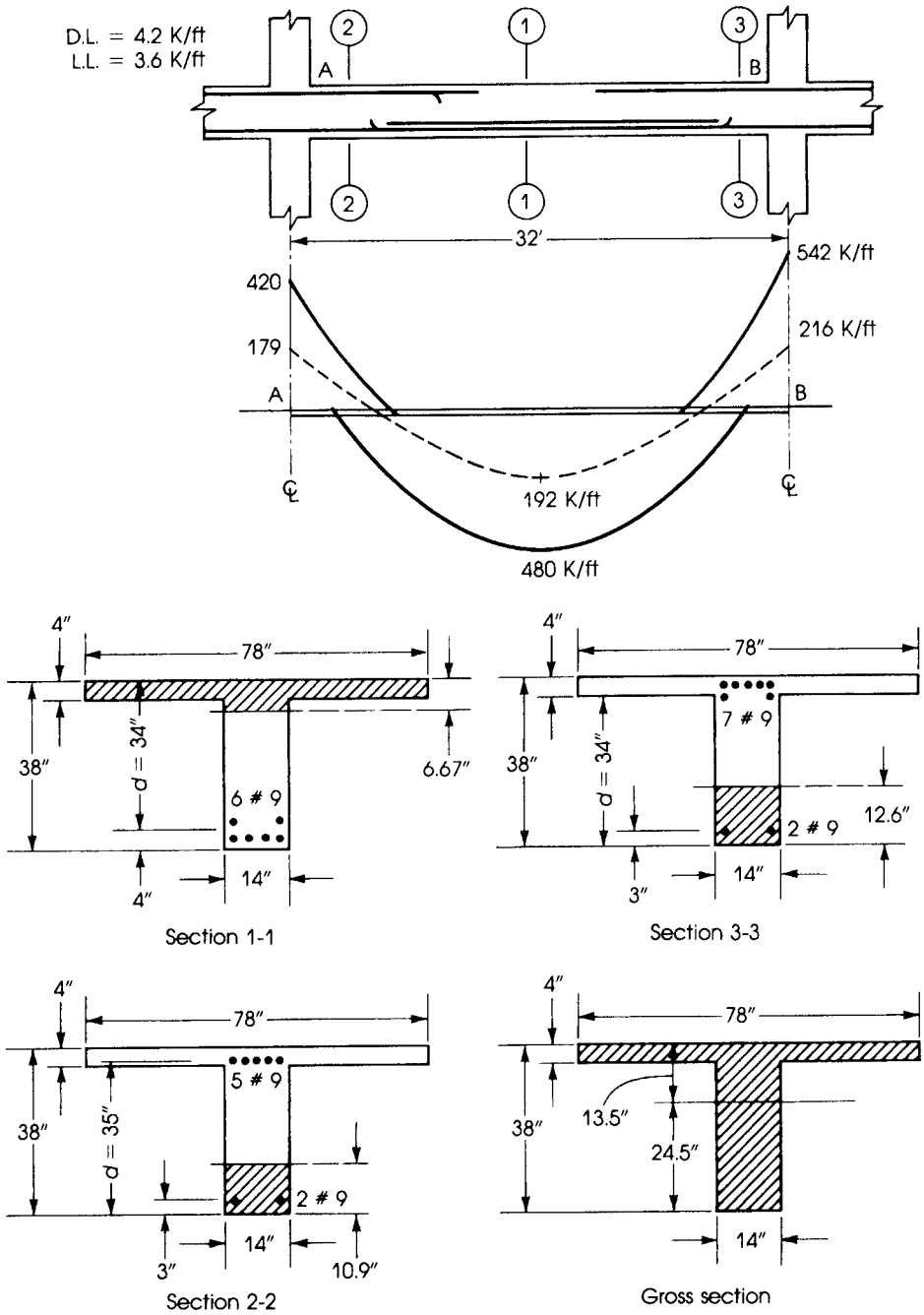


Figure 6.5 Example 6.5: deflection of a continuous beam.

I_e for dead and live loads (using M_a for dead and live loads) are calculated and tabulated as follows.

Section	kd (in.)	I_{cr} (in.^{4})	M_{cr} (K·ft)	I_e (in.^{4}) (Dead load)	I_e (in.^{4}) ($D + L$)
Midspan	6.67	48,550	159.4	86,160	50,960
Support A	10.9	34,930	289.3	114,300	60,880
Support B	12.6	44,860	289.3	114,300	55,415

Note that when the beam is subjected to dead load only and the ratio M_{cr}/M_a is greater than 1.0, I_e is equal to I_g.

3. Calculate average I_e from Eq. 6.8:

$$I_{e1}(\text{average}) = 0.7(50{,}960) + 0.15(60{,}880 + 55{,}415)$$
$$= 53{,}116 \text{ in.}^4$$

For dead and live loads,

$$\text{Average } I_e \text{ for end sections} = \tfrac{1}{2}(60{,}880 + 55{,}415)$$
$$= 58{,}150 \text{ in.}^4$$
$$I_{e2} \text{ (average)} = \tfrac{1}{2}(50{,}960 + 58{,}150) = 54{,}550 \text{ in.}^4$$

For dead loads only,

$$\text{Average } I_e \text{ for end sections} = 114{,}300 \text{ in.}^4$$
$$I_{e3} \text{ (average)} = \tfrac{1}{2}(86{,}160 + 114{,}300) = 100{,}230 \text{ in.}^4$$

4. Calculate immediate deflection at midspan:

$$\Delta_1 \text{ (due to uniform load)} = \frac{5wL^4}{384EI_e} \quad \text{(downward)}$$

$$\Delta_2 \text{ (due to a moment at } A, M_A) - \frac{M_A L^2}{16EI_e} \quad \text{(upward)}$$

$$\Delta_3 \text{ (due to a moment at } B, M_B) = -\frac{M_B L^2}{16EI_e} \quad \text{(upward)}$$

$$\text{Total deflection } \Delta = \Delta_1 - \Delta_2 - \Delta_3$$

The dead-load deflection for a uniform dead load of 4.2 K/ft, taking M_A(D.L.) = 179 K·ft, M_B (D.L.) = 216 K·ft, and I_{e3} = 100,230 in.^{4} and then substituting in the preceding equations, is

$$\Delta = 0.314 - 0.063 - 0.075 = 0.176 \text{ in.} \quad \text{(downward)}$$

The deflection due to combined dead and live loads is found by taking dead plus live load = 7.8 K/ft, M_A = 420 K·ft, M_B = 542 K·ft, and I_{e2} = 54,550 in.^{4}:

$$\Delta = 1.071 - 0.270 - 0.349 = 0.452 \text{ in.} \quad \text{(downward)}$$

The immediate deflection due to live load only is 0.542 − 0.176 = 0.276 in. (downward). If the limiting permissible deflection is $L/480$ = (32 × 12)/480 = 0.8 in., then the section is adequate.

There are a few points to mention about the results.

a. If I_e of the midspan section only is used ($I_e = 50{,}960\,\text{in.}^4$) then the deflection due to dead plus live loads is calculated by multiplying the obtained value in step 4 by the ratio of the two I_e:

$$\Delta \text{ (dead + live)} = 0.452 \times \left(\frac{54{,}550}{50{,}960}\right) = 0.484 \text{ in.}$$

The difference is small, about 7% on the conservative side.

b. If I_e 1 (average) is used ($I_{e1} = 53{,}116\,\text{in.}^4$), then Δ (dead + live) = 0.471 in. The difference is small, about 4% on the conservative side.

c. It is believed that it is more convenient to use I_e at midspan section unless a more rigorous solution is required.

6.6 CRACKS IN FLEXURAL MEMBERS

The study of crack formation, behavior of cracks under increasing load, and control of cracking is necessary for proper design of reinforced concrete structures. In flexural members, cracks develop under working loads, and because concrete is weak in tension, reinforcement is placed in the cracked tension zone to resist the tension force produced by the external loads.

Flexural cracks develop when the stress at the extreme tension fibers exceeds the modulus of rupture of concrete. With the use of high-strength reinforcing bars, excessive cracking may develop in reinforced concrete members. The use of high-tensile steel has many advantages, yet the development of undesirable cracks seems to be inevitable. Wide cracks may allow corrosion of the reinforcement or leakage of water structures and may spoil the appearance of the structure.

A crack is formed in concrete when a narrow opening of indefinite dimension has developed in the concrete beam as the result of internal tensile stresses. These internal stresses may be due to one or more of the following:

- External forces such as direct axial tension, shear, flexure, or torsion
- Shrinkage
- Creep
- Internal expansion resulting from a change of properties of the concrete constituents

In general, cracks may be divided into two main types: secondary cracks and main cracks.

6.6.1 Secondary Cracks

Secondary cracks, very small cracks that develop in the first stage of cracking, are produced by the internal expansion and contraction of the concrete constituents and by low flexural tension stresses due to the self-weight of the member and any other dead loads. There are three types of secondary cracks.

Shrinkage cracks. *Shrinkage cracks* are important cracks, because they affect the pattern of cracking that is produced by loads in flexural members. When they develop, they form a weak path in the concrete. When load is applied, cracks start to appear at the weakest sections, such as along the reinforcing bars. The number of cracks formed is limited by the amount of shrinkage in concrete and the presence of restraints. Shrinkage cracks are difficult to control.

Secondary flexural cracks. Usually *secondary flexural cracks* are widely spaced, and one crack does not influence the formation of others [8]. They are expected to occur under low loads, such as dead loads. When a load is applied gradually on a simple beam, tensile stress develops at the bottom fibers, and when it exceeds the flexural tensile stress of concrete, cracks start to develop. They widen gradually and extend toward the neutral axis. It is difficult to predict the sections at which secondary cracks start because concrete is not a homogeneous, isotropic material.

Salinger [9] and Billing [10] estimated the steel stress just before cracking to be from about 6000 to 7000 psi (42 to 49 MPa). An initial crack width of the order of 0.001 in. (0.025 mm) is expected at the extreme concrete tensile fibers. Once cracks are formed, the tensile stress of concrete at the cracked section decreases to 0, and the steel bars take all the tensile force. At this moment, some slip occurs between the steel bars and the concrete due to the differential elongation of concrete and steel and extends to a section where the concrete and steel strains are equal. Figure 6.6 shows the typical stress distribution between cracks in a member under axial tension.

Corrosion secondary cracks. *Corrosion secondary cracks* form when moisture containing deleterious agents such as sodium chloride, carbon dioxide, and dissolved oxygen penetrates the concrete surface, corroding the steel reinforcement [11]. The oxide compounds formed by deterioration of steel bars occupy a larger volume than the steel and exert mechanical pressure that perpetuates extensive cracking [12,13]. This type of cracking may be severe enough to result in eventual failure of the structure. The failure of a roof in Muskegan, Michigan, in 1955 due to the corrosion of steel bars was reported by Shermer [13]. The extensive cracking and spalling of concrete in the San Mateo–Hayward Bridge in California within 7 years was reported by Stratful [12]. Corrosion cracking may be forestalled by using proper construction methods and high-quality concrete. More details are discussed by Evans [14] and Mozer and others [15].

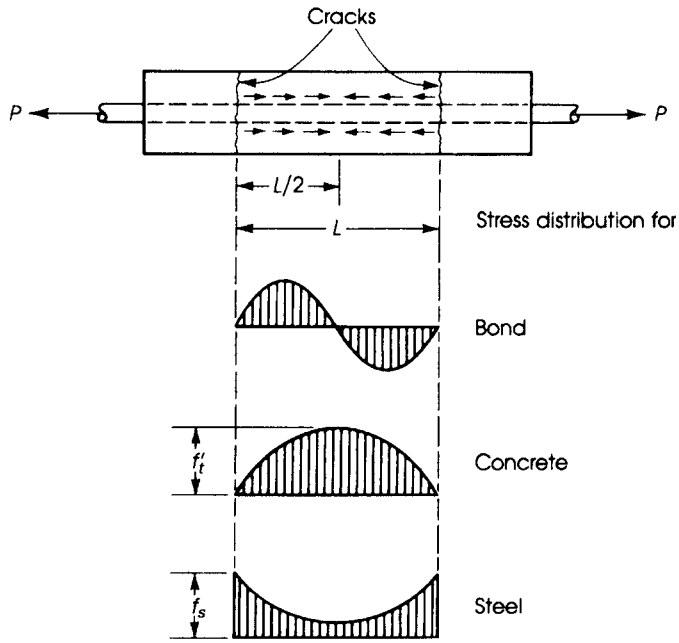


Figure 6.6 Typical stress distribution between cracks.

6.6.2 Main Cracks

Main cracks develop at a later stage than secondary cracks. They are caused by the difference in strains in steel and concrete at the section considered. The behavior of main cracks changes at two different stages. At low tensile stresses in steel bars, the number of cracks increases, whereas the widths of cracks remain small; as tensile stresses are increased, an equilibrium stage is reached. When stresses are further increased, the second stage of cracking develops, and crack widths increase without any significant increase in the number of cracks. Usually one or two cracks start to widen more than the others, forming critical cracks (Fig. 6.7).

Main cracks in beams and axially tensioned members have been studied by many investigators; prediction of the width of cracks and crack control were among the problems studied. These are discussed here, along with the requirements of the ACI Code.

Crack width. *Crack width* and *crack spacing*, according to existing crack theories, depend on many factors, which include steel percentage, its distribution in the concrete section, steel flexural stress at service load, concrete cover, and properties of the concrete constituents. Different equations for predicting the width and spacing of cracks in reinforced concrete members were presented at the Symposium on Bond and Crack Formation in Reinforced Concrete in Stockholm, Sweden, in 1957. Chi and Kirstein [16] presented equations for the crack width and spacing as a function of an effective area of concrete around the steel bar: A concrete circular area of diameter equal to four times the diameter of the bar was used to calculate crack width. Other equations were presented over the next decade [17–23].

Gergely and Lutz [23] presented the following formula for the limiting crack width:

$$W = 0.076\beta f_s \sqrt[3]{Ad_c} \times 10^{-6} \text{ (in.)} \tag{6.15}$$

where β, A, and f_s are as defined previously and d_c = thickness of concrete cover measured from the extreme tension fiber to the center of the closest bar. The value of β can be taken to be approximately equal to 1.2 for beams and 1.35 for slabs. Note that f_s is in psi and W is in inches.

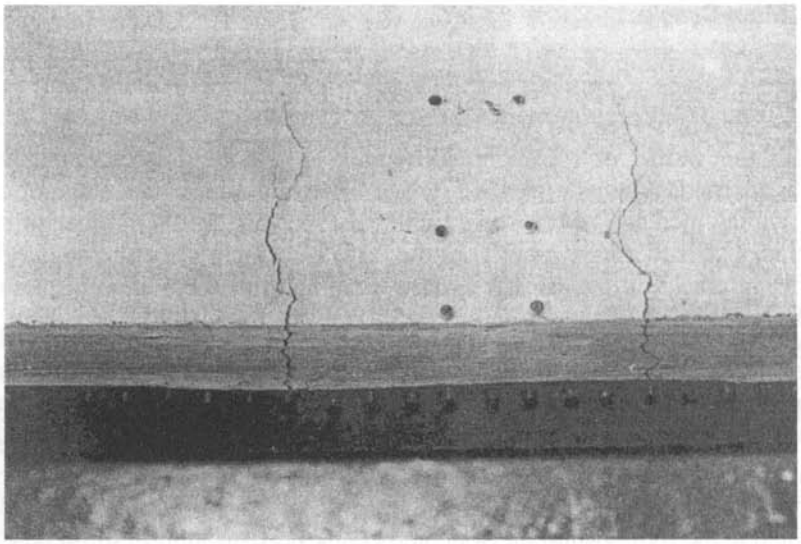
The mean ratio of maximum crack width to average crack width was found to vary between 1.5 and 2.0, as reported by many investigators. An average value of 1.75 may be used.

In SI units (mm and MPa), Eq. 6.15 is

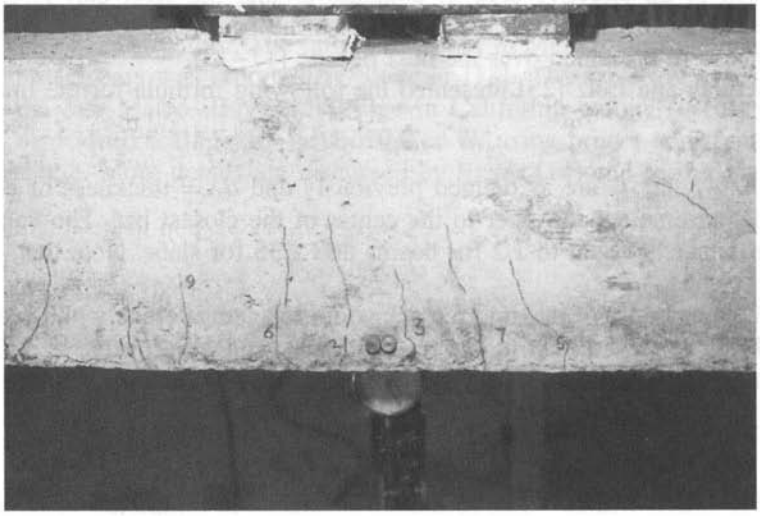
$$W = 11.0\beta f_s \sqrt[3]{Ad_c} \times 10^{-6} \tag{6.16}$$

Tolerable crack width. The formation of cracks in reinforced concrete members is unavoidable. Hairline cracks occur even in carefully designed and constructed structures. Cracks are usually measured at the face of the concrete, but actually they are related to crack width at the steel level, where corrosion is expected. The permissible crack width is also influenced by aesthetic and appearance requirements. The naked eye can detect a crack about 0.006 in. (0.15 mm) wide, depending on the surface texture of concrete. Different values for permissible crack width at the steel level have been suggested by many investigators, ranging from 0.010 to 0.016 in. (0.25–0.40 mm) for interior members and from 0.006 to 0.010 in. (0.15–0.25 mm) for exterior exposed members. A limiting crack width of 0.016 in. (0.40 mm) for interior members and 0.013 in. (0.32 mm) for exterior members under dry conditions can be tolerated.

Crack control. *Control* grows in importance with the use of high-strength steel in reinforced concrete members, as larger cracks develop under working loads because of the high allowable stresses. Control of cracking depends on the permissible crack width: It is always preferable to



(*a*)



(*b*)

Figure 6.7 (*a*) Main cracks in a reinforced concrete beam. (*b*) Spacing of cracks in a reinforced concrete beam.

have a large number of fine cracks rather than a small number of large cracks. Secondary cracks are minimized by controlling the total amount of cement paste, water–cement ratio, permeability of aggregate and concrete, rate of curing, shrinkage, and end-restraint conditions.

The factors involved in controlling main cracks are the reinforcement stress, the bond characteristics of reinforcement, the distribution of reinforcement, the diameter of the steel bars used, the steel percentage, the concrete cover, and the properties of concrete constituents. Any improvement in these factors will help in reducing the width of cracks.

6.7 ACI CODE REQUIREMENTS

To control cracks in reinforced concrete members, the ACI Code, Chapter 10, specifies the following:

1. Only deformed bars are permitted as main reinforcement.
2. Tension reinforcement should be well distributed in the zones of maximum tension (Section 10.6.3).
3. When the flange of the section is under tension, part of the main reinforcement should be distributed over the effective flange width or one-tenth of the span, whichever is smaller. Some longitudinal reinforcement has to be provided in the outer portion of the flange (Section 10.6.6).
4. The design yield strength of reinforcement should not exceed 80 ksi (560 MPa) (Section 9.4).
5. The maximum spacing **s** of reinforcement closest to a concrete surface in tension in reinforced concrete beams and one-way slabs is limited to

$$\mathbf{s}\ (\text{in.}) = \left[15\left(\frac{40}{f_s}\right) - 2.5C_c\right] \tag{6.17}$$

but not greater than 12 $(40/f_s)$, where

f_s = calculated stress (ksi) in reinforcement at service load computed as the unfactored moment divided by the product of steel area and the internal moment arm, $f_s = M/(A_s jd)$. (Alternatively, $f_s = \frac{2}{3}f_y$ may be used; an approximate value of $jd = 0.87d$ may be used.)

C_c = clear cover from the nearest surface in tension to the surface of the flexural tension reinforcement (in.).

s = center to center spacing of flexural tension reinforcement nearest to the extreme concrete tension face (in.).

The preceding limitations are applicable to reinforced concrete beams and one-way slabs subject to normal environmental condition and do not apply to structures subjected to aggressive exposure. The spacing limitation just given is independent of the bar size, which may lead to the use of smaller bar sizes to satisfy the spacing criteria. For the case of concrete beams reinforced with grade 60 steel bars and $C_c = 2$ in., clear cover to the tension face, the maximum spacing is calculated as follows: Assume $f_s = 2/3\ f_y = (2/3) \times 60 = 40$ ksi and $\mathbf{s} = 15\left(\frac{40}{40}\right) - 2.5 \times 2 = 10$ in. (controls), which is less than 12(40/40) = 12 in.

6. In SI units, Eq. 6.17 becomes

$$\mathbf{s}\ (\text{mm}) = 105{,}000/f_s - 2.5C_c \tag{6.18}$$

but not greater than 300 $(280/f_s)$, where f_s is in MPa and C_c is in mm. For example, if bars with a clear cover equal to 50 mm are used, then the maximum spacing, **s**, is calculated as follows:

$$\mathbf{s} = (105{,}000/280) - 2.5 \times 50 = 250 \text{ mm (controls)},$$

which is less than 300(280/280) = 300 mm in this example. This is assuming that $f_s = \frac{2}{3} \times 420 = 280$ MPa.

7. In the previous Codes, control of cracking was based on a factor Z defined as follows:

$$Z = f_s\sqrt[3]{Ad_c} \leq 175 \text{ K/in.} \quad (31 \text{ kN/mm}) \quad \text{for interior members}$$

$$Z \leq 140 \text{ K/in. } (26 \text{ kN/mm}) \quad \text{for exterior members.} \qquad (6.19)$$

where f_s = flexural stress at service load (ksi) and may be taken as 0.6 f_y. A and d_c are the effective tension area of concrete and thickness of concrete cover, respectively. This expression is based on Eq. 6.15 assuming a limiting crack width of 0.016 in. for interior members and 0.013 in. for exterior members. It encouraged a decrease in the reinforcement cover to achieve a smaller Z, while unfortunately it penalized structures with concrete cover that exceeded 2 in.

8. *Skin reinforcement*: For relatively deep girders, with a total depth, h, equal to or greater than 36 in. (900 mm), light reinforcement should be added near the vertical faces in the tension zone to control cracking in the web above the main reinforcement. The ACI Code, Section 10.6.7, referred to this additional steel as skin reinforcement. The skin reinforcement should be uniformly distributed along both side faces of the member for a distance $h/2$ from the tension face.

The spacing S between the longitudinal bars or wires of the skin reinforcement shall be as provided in Eq. 6.17 where C_c is the least distance from the skin reinforcement to the side face.

Referring to Figure 6.8, if b = 16 in., h = 40 in., fy = 60 ksi and choosing no. 3 bars spaced at 6.0 in. as skin reinforcement (3 spaces on each side), then the height covered = 3 × 6 + 2.5 = 20.5 in., which is greater than $h/2$ = 40/2 = 20 in.

Checking the spacing S by Eq. 6.18 and assuming f_s = 2/3, f_y = 2/3 × 60 = 40 ksi, and C_c = 2 in., then S = 15(40/40) − 2.5 × 2 = 10 in., which is less than 12(40/40) = 12 in. The spacing used is adequate. Note that C_c = 1.5 in. may be used for the skin reinforcement concrete cover.

It is recommended to use smaller spacing to control the propagation of tensile cracks along the side of the tension zone with the first side bar to be placed at 4 to 6 in. from the main tensile steel.

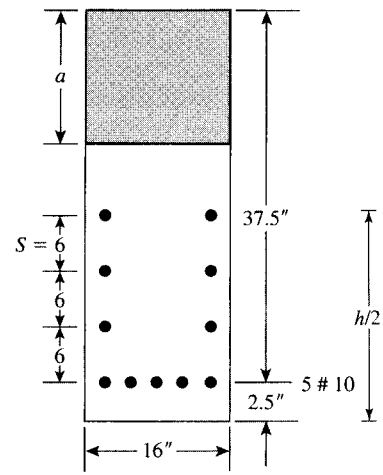


Figure 6.8 Skin reinforcement (6 no. 3 bars).

Example 6.6

The sections of a simply supported beam are shown in Fig. 6.9.

a. Check if the bar arrangement satisfies the ACI Code requirements.
b. Determine the expected crack width.
c. Check the Z-factor based on Eq. 6.19.

Given: $f'_c = 4$ ksi, $f_y = 60$ ksi, and no. 3 stirrups.

Solution

1. Fig. 6.9, section *a*:
   a. For three no. 8 bars, $A_s = 2.35\,\text{in.}^2$, clear cover, C_c, $= 2.5 - 8/16 = 2.0$ in. Assume $f_s = \frac{2}{3} f_y = 2/3 \times 60 = 40$ ksi. Maximum spacing $\mathbf{s} = 600/40 - 2.5 \times 2 = 10$ in., which is less than $12(40/40) = 12$ in. Spacing provided $= 0.5(12 - 2.5 - 2.5) = 3.5$ in., center to center of bars, which is less than 10 in.
   b. For this section, $d_c = 2.5$ in. The effective tension area of concrete for one bar is

$$A = 12(2 \times 2.5)/3 = 20\ \text{in.}^2$$

   Estimated crack width using Eq. 6.16 is

$$W = 0.076(1.2)(36{,}000)\sqrt[3]{20 \times 2.5} \times 10^{-6} = 0.0121\ \text{in.}$$

   This is assuming $\beta = 1.2$ for beams and $f_s = 36$ ksi. The crack width above is less than 0.016 in. and 0.013 in. for interior and exterior members.

2. Fig. 6.9, section *b*:
   a. Calculations of spacing of bars are similar to those in section *a*.
   b. For this section, $d_c = 2.5$ in., and the steel bars are placed in two layers. The centroid of the steel bars is 3.5 in. from the bottom fibers. The effective tension concrete area is $A = 12(2 \times 3.5)/6 = 14\,\text{in.}^2$

$$W = .076 \times 1.2 \times 36{,}000\sqrt[3]{14 \times 2.5} \times 10^{-6} = 0.0107\ \text{in.}$$

   which is adequate.

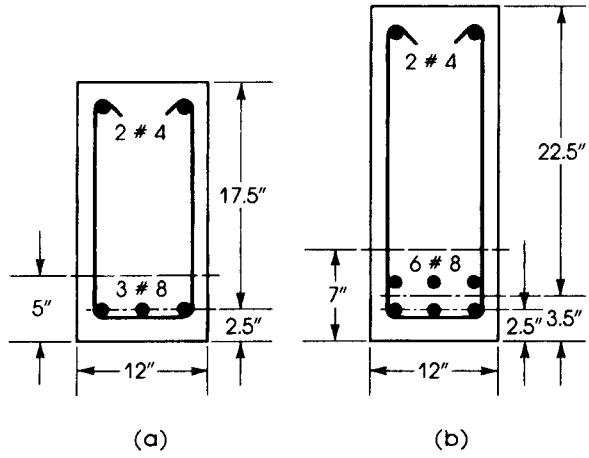


Figure 6.9 Two sections for Example 6.6.

Discussion

It can be seen that the spacing, **s**, in Eq. 6.17 is a function of the stress in the tension bars or, indirectly, is a function of the strain in the tension steel, $f_s = E_s \times \varepsilon_s$, and E_s for steel is equal to 29,000 ksi. The spacing also depends on the concrete cover, C_c. An increase in the concrete cover will reduce the limited spacing **s**, which is independent on the bar size used in the section.

In this example, the expected crack width was calculated by Eq. 6.17 to give the student or the engineer a physical feeling for the crack width and crack control requirement. The crack width is usually measured in beams when tested in the laboratory or else in actual structures under loading when serious cracks develop in beams or slabs and testing is needed. If the crack width measured before and after loading is greater than the yield strain of steel, then the main reinforcement is in the plastic range and ineffective. Sheets with lines of different thickness representing crack widths are available in the market for easy comparisons with actual crack widths. In addition to the steel stress and the concrete cover, W depends on a third factor, A, representing the tension area of concrete surrounding one bar in tension.

Example 6.7

Design a simply supported beam with a span of 24 ft to carry a uniform dead load of 1.5 K/ft and a live load of 1.18 K/ft. Choose adequate bars; then check their spacing arrangement to satisfy the ACI Code. Given: $b = 16$ in., $f'_c = 4$ ksi, $f_y = 60$ ksi, a steel percentage = 0.8%, and a clear concrete cover of 2 in.

Solution

1. For a steel percentage of 0.8%, $R_u = 400$ psi $= 0.4$ ksi ($\phi = 0.9$). The external factored moment is $M_u = w_u \times L^2/8$, and $w_u = 1.2(1.5) + 1.6(1.18) = 3.69$ K/ft.

$$M_u = 3.69(24)^2/8 = 265.68 \text{ K·ft} = 3188.2 \text{ K·in.}$$

$$M_u = R_u \cdot bd^2 \qquad d = 22.32 \qquad A_s = 0.008 \times 16 \times 22.32 = 2.86 \text{ in.}^2$$

Choose three no. 9 bars (area $= 3.0$ in.^{2}) in one row, and a total depth of $h = 25.0$ in. Actual $d = 25 - 2 - 9/16 = 22.44$ in. (Fig. 6.10).

2. Check spacing of bars using Eq. 6.18. Calculate the service load and moment: $w = 1.5 + 1.18 = 2.68$ K/ft.

$$M = 2.68(24)^2/8 = 193 \text{ K·ft} = 2315 \text{ K·in.}$$

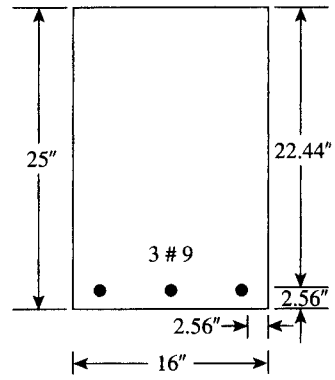


Figure 6.10 Example 6.7.

3. Calculate the neutral axis depth kd and the moment arm jd (Eq. 6.12).

$$b(kd)^2/2 - nA_s(d - kd) = 0 \qquad n = 8 \qquad A_s = 3.0 \qquad d = 22.44 \text{ in.}$$

$$kd = 6.85 \text{ in.} \qquad jd = d - kd/3 = 20.16 \text{ in.} \qquad j = 20.16/22.44 = 0.898$$

Note that an approximate value of $j = 0.87$ may be used if kd is not calculated.

4. Calculate the stress f_s:

$$M = A_s \cdot f_s \cdot jd \quad 2315 = 3(f_s)(20.16) \quad f_s = 38.3 \text{ ksi}$$

5. Calculate the spacing **s** by Eq. 6.18:

$$\mathbf{s} = 600/38.3 - 2.5 \times 2 = 10.7 \text{ in. (controls)}$$

which is less than 12(40/40) = 12.0 in. Spacing provided, = 0.5 (16 − 2.56 − 2.56) = 5.44 in., which is less than 10.7 in.

Example 6.8: SI Units

Design a simply supported beam of 7.2-m span to carry a uniform dead load of 22.2 kN/m and a live load of 17 kN/m. Choose adequate bars, and check their spacing arrangement to satisfy the ACI Code.

Given: $b = 400$ mm, $f'_c = 30$ MPa, $f_y = 400$ MPa, a steel percentage of 0.8%, and a clear concrete cover of 50 mm.

Solution

1. For a steel percentage of 0.008 and from Eq. 3.22, $R_u = 2.7$ MPa. Factored load $w_u = 1.2(22.2) + 1.6(17) = 53.8$ kN/m. $M_u = w_u \cdot L^2/8 = 53.8(7.2)^2/8 = 348.6$ kN·m. $M_u = R_u \cdot bd^2$, or $348.6 \times 10^6 = 2.7 \times 400d^2$ then $d = 568$ mm $A_s = \rho bd = 0.008 \times 400 \times 568 = 1818$ mm^2. Choose four bars, 25 mm (no. 25 M), $A_s = 2040$ mm^2, in one row ($b_{\min} = 220$ mm). Let $h = 650$ mm, the actual $d = 650 - 50 - 25/2 = 587.5$ mm, say 585 mm. Final section: $b = 400$ mm, $h = 650$ mm, with four no. 25 mm bars (Fig. 6.11).
2. Check spacing of bars using Eq. 6.17. Calculate the service load moment, $w = 22.2 + 17 = 39.2$ kN/m.

$$M = 39.2(7.2)^2/8 = 254 \text{ kN·m}$$

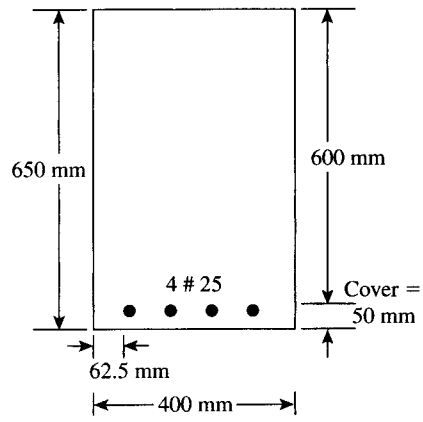


Figure 6.11 Example 6.8.

Calculate kd and jd as in the previous example. Alternatively, use a moment arm, $jd = 0.87d = 0.87(585) = 509$ mm and $f_s = M/(A_s \cdot jd) = 254(10)^6/(2040 \times 585) = 213$ MPa. From Eq. 6.19, maximum $s = (105{,}000/213) - 2.5(50) = 368$ mm (controls), which is less than $300(280/f_s) = 300(280)/213 = 394$ mm. Note that if $f_s = 0.6\ f_y = 0.6(400) = 240$ MPa is used, then maximum $s = 312$ mm. It is preferable to calculate f_s from the moment equation to reflect the actual stress in the bars. Spacing provided $= (1/3)(400 - 50 - 25) = 92$ mm, which is adequate.

SUMMARY

Sections 6.1–6.2

1. Deflection $\Delta = \alpha(WL^3/EI) = 5WL^3/384EI = 5\ wL^4/384EI$ for a simply supported beam subjected to a uniform total load of $W = wL$.

$$E_c = 33w^{1.5}\sqrt{f'_c} = 57{,}400\ f'_c \text{ psi}$$

for normal-weight concrete.

2. Effective moment of inertia is

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g$$

$$M_{cr} = f_r \times \frac{I_g}{y_t} \quad \text{and} \quad f_r = 7.5\lambda\sqrt{f'_c} \tag{6.5}$$

Section 6.3

The deflection of reinforced concrete members continues to increase under sustained load. Additional long-time deflection $= \zeta_\Delta \times$ instantaneous deflection:

$$\zeta_\Delta = \frac{\zeta}{1 + 50\rho'} \tag{6.14}$$

$\zeta = 1.0$, 1.2, 1.4, and 2.0 for periods of 3, 6, 12, and 60 months, respectively.

Sections 6.4–6.5

1. The allowable deflection varies between $L/180$ and $L/480$.
2. Deflections for different types of loads may be calculated for each type of loading separately and then added algebraically to obtain the total deflection.

Section 6.6

1. Cracks are classified as secondary cracks (shrinkage, corrosion, or secondary flexural cracks) and main cracks.
2. Maximum crack width is

$$W = 0.076\beta f_s \sqrt[3]{Ad_c} \times 10^{-6} \text{ (in.)} \tag{6.15}$$

Approximate values for β, f_s, and d_c are $\beta = 1.2$ for beams and 1.35 for slabs, $d_c = 2.5$ in., and $f_s = (2/3)f_y$.

3. The limiting crack width is 0.016 in. for interior members and 0.013 in. for exterior members.

Section 6.7

The maximum spacing **s** of bars closest to a concrete surface in tension is limited to

$$\mathbf{s} = 600/f_s - 2.5C_c \tag{6.17}$$

but not more than $12(40/f_s)$. Note that f_s may be taken as $2/3 f_y$.

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PROBLEMS

6.1 Determine the instantaneous and long-time deflection of a 20-ft-span simply supported beam for each of the following load conditions. Assume that 10% of the live loads are sustained and the dead loads include the self-weight of the beams. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, $d' = 2.5$ in., and a time limit of 5 years. Refer to Fig. 6.12.

No.	b (in.)	d (in.)	h (in.)	A_s (in.^{2})	A'_s (in.^{2})	W_D (K/ft)	W_L (K/ft)	P_D (K)	P_L (K)
a	14	17.5	20	5 no. 9	—	2.2	1.8	—	—
b	20	27.5	30	6 no. 10	—	7.0	3.6	—	—
c	12	19.5	23	6 no. 8	—	3.0	1.5	—	—
d	18	20.5	24	6 no. 10	2 no. 9	6.0	2.0	—	—
e	16	22.5	26	6 no. 11	2 no. 10	5.0	3.2	12	10
f	14	20.5	24	8 no. 9	2 no. 9	3.8	2.8	8	6

h-d = 2.5 in. indicates one row of bars, whereas h-d = 3.5 in. indicates two rows of bars. Concentrated loads are placed at midspan.

6.2 Determine the instantaneous and long-term deflection of the free end of a 12-ft-span cantilever beam for each of the following load conditions. Assume that only dead loads are sustained, and the dead loads include the self-weight of the beams. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, and a time limit of more than 5 years. Refer to Fig. 6.13.

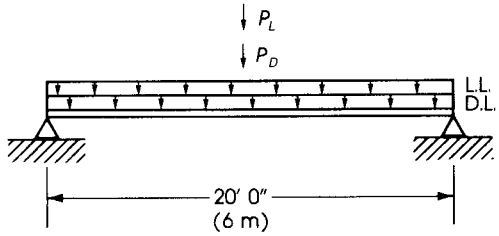


Figure 6.12 Problem 6.1.

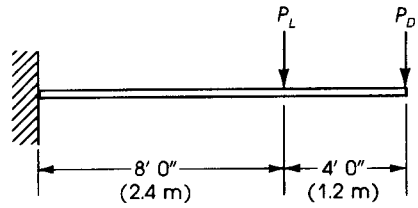


Figure 6.13 Problem 6.2.

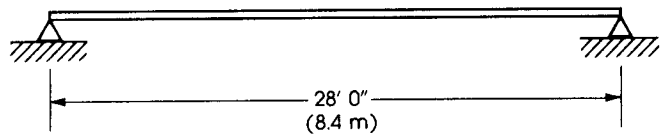


Figure 6.14 Problem 6.3: Dead load = 2 K/ft (30 kN/m) and live load = 1.33 K/ft (20 kN/m).

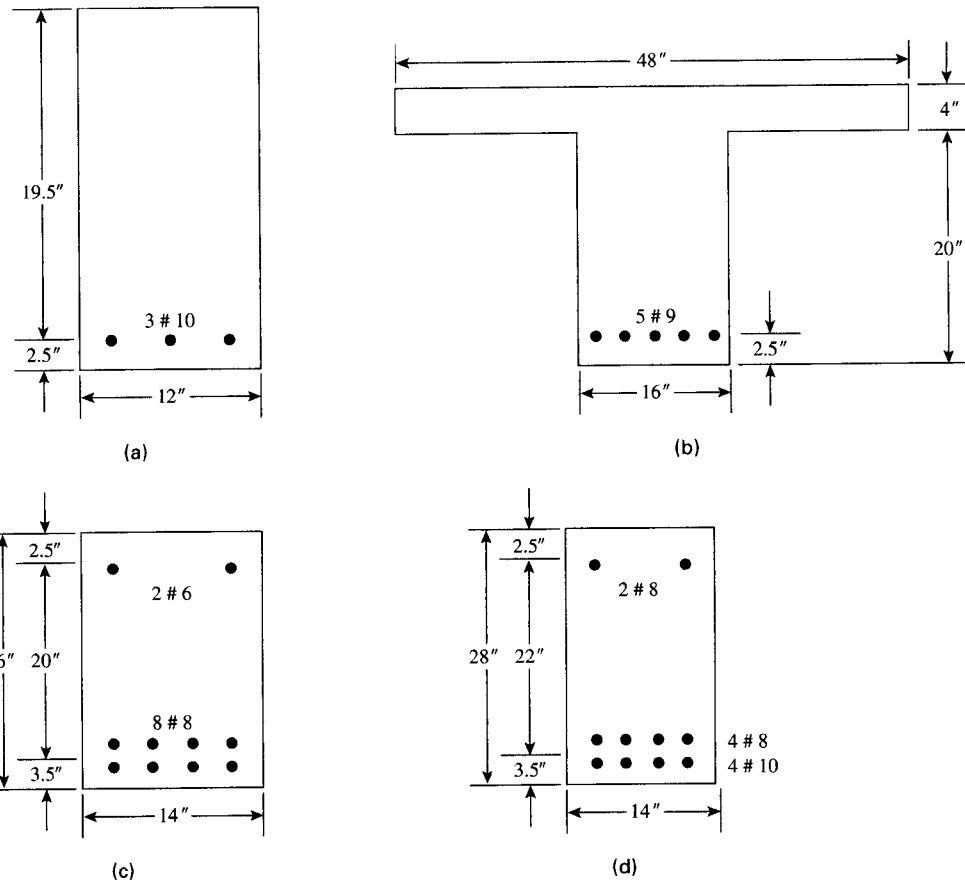


Figure 6.15 Problem 6.5.

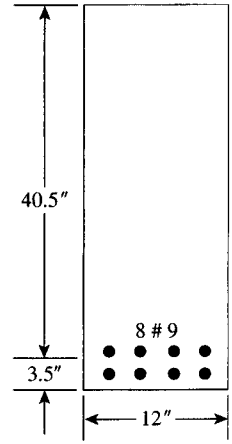


Figure 6.16 Problem 6.6 (skin reinforcement).

No.	b (in.)	d (in.)	h (in.)	A_s (in.^{2})	A'_s (in.^{2})	W_D (K/ft)	W_L (K/ft)	P_D (K)	P_L (K)
a	15	20.5	24	8 no. 9	2 no. 9	3.5	2.0	—	—
b	18	22.5	26	6 no. 10	—	2.0	1.5	7.4	5.0
c	12	19.5	23	8 no. 8	2 no. 8	2.4	1.6	—	—
d	14	20.5	24	8 no. 9	2 no. 9	3.0	1.1	5.5	4.0

h-d = 2.5 in. indicates one row of bars, whereas h-d = 3.5 in. indicates two rows of bars. Concentrated loads are placed as shown

6.3 A 28-ft simply supported beam carries a uniform dead load of 2 K/ft (including self-weight) and a live load of 1.33 K/ft. Design the critical section at midspan using the maximum steel ratio allowed by the ACI Code and then calculate the instantaneous deflection. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, and $b = 12$ in. See Fig. 6.14.

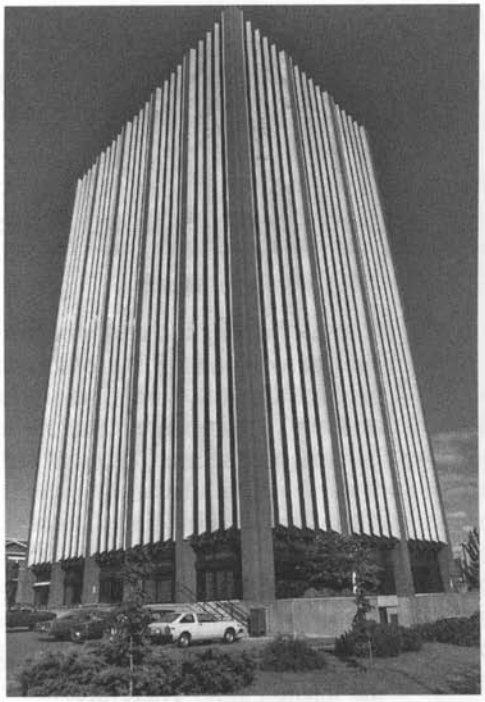
6.4 Design the beam in Problem 6.3asdoubly reinforced, considering that compression steel resists 20% of the maximum bending moment. Then calculate the maximum instantaneous deflection.

6.5 The four cross-sections shown in Fig. 6.15 belong to four different beams with $f'_c = 4$ ksi and $f_y = 60$ ksi. Check the spacing of the bars in each section according to the ACI Code requirement using $f_s = 0.6 f_y$. Then calculate the tolerable crack width, W.

6.6 Determine the necessary skin reinforcement for the beam section shown in Fig. 6.16. Then choose adequate bars and spacings. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

CHAPTER 7

DEVELOPMENT LENGTH OF REINFORCING BARS



Reinforced concrete columns supporting an office building, Toronto, Canada.

7.1 INTRODUCTION

The joint behavior of steel and concrete in a reinforced concrete member is based on the fact that a bond is maintained between the two materials after the concrete hardens. If a straight bar of round section is embedded in concrete, a considerable force is required to pull the bar out of the concrete. If the embedded length of the bar is long enough, the steel bar may yield, leaving some length of the bar in the concrete. The bonding force depends on the friction between steel and concrete. It is influenced mainly by the roughness of the steel surface area, the concrete mix, shrinkage, and the cover of concrete. Deformed bars give a better bond than plain bars. Rich mixes have greater adhesion than weak mixes. An increase in the concrete cover will improve the ultimate bond stress of a steel bar [2].

In general, the bond strength is influenced by the following factors:

1. Yield strength of reinforcing bars, f_y. Longer development length is needed with higher f_y.
2. Quality of concrete and its compressive strength, f'_c. An increase in f'_c reduces the required development length of reinforcing bars.
3. Bar size, spacing, and location in the concrete section. Horizontal bars placed with more than 12 in. of concrete below them have lower bond strength due to the fact that concrete shrinks and settles during the hardening process. Also, wide spacings of bars improve the bond strength, giving adequate effective concrete area around each bar.
4. Concrete cover to reinforcing bars. A small cover may cause the cracking and spalling of the concrete cover.

5. Confinement of bars by lateral ties. Adequate confinement by ties or stirrups prevents the spalling of concrete around bars.

7.2 DEVELOPMENT OF BOND STRESSES

7.2.1 Flexural Bond

Consider a length dx of a beam subjected to uniform loading. Let the moment produced on one side be M_1 and on the other side be M_2 with M_1 being greater than M_2. The moments will produce internal compression and tension forces, as shown in Fig. 7.1. Because M_1 is greater than M_2, T_1 is greater than T_2; consequently, C_1 is greater than C_2.

At any section, $T = M/jd$, where jd is the moment arm:

$$T_1 - T_2 = dT = \frac{dM}{jd}$$

but

$$T_1 = T_2 + u\Sigma O dx$$

where u is the average bond stress and ΣO is the sum of perimeters of bars in the section at the tension side. Therefore,

$$T_1 - T_2 = u\Sigma O dx = \frac{dM}{jd}$$

$$u = \frac{dM}{dx} \times \frac{1}{jd\Sigma O}$$

The rate of change of the moment with respect to x is the shear, or $dM/dx = V$. Therefore,

$$u = \frac{V}{jd\Sigma O} \tag{7.1}$$

The value u is the average bond stress; for practical calculations, j can be taken to be approximately equal to 0.87:

$$u = \frac{V}{0.87\, d\Sigma O}$$

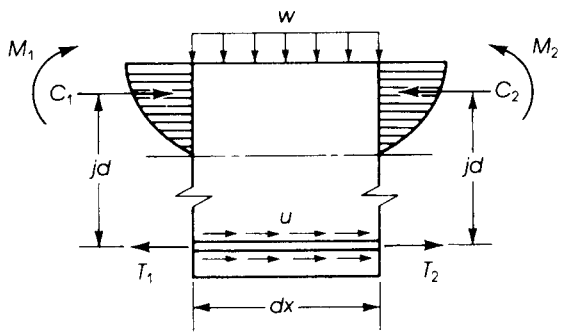


Figure 7.1 Flexural bond.

In the strength design method, the nominal bond strength is reduced by the capacity reduction factor, $\phi = 0.85$. Thus,

$$U_u = \frac{V_u}{\phi(0.87)d\Sigma O} \tag{7.2}$$

Based on the preceding analysis, the bond stress is developed along the surface of the reinforcing bar due to shear stresses and shear interlock.

7.2.2 Tests for Bond Efficiency

Tests to determine the bond stress capacity can be made using the pullout test (Fig. 7.2). This test evaluates the bond capacity of various types of bar surfaces relative to a specific embedded length. The distribution of tensile stresses will be uniform around the reinforcing bar at a specific section and varies along the anchorage length of the bar and at a radial distance from the surface of the bar (Fig. 7.2). However, this test does not represent the effective bond behavior in the surface of the bars in flexural members, because stresses vary along the depth of the concrete section. A second type of test can be performed on an embedded rod (Fig. 7.3). In these tests, the tensile force, P, is increased gradually and the number of cracks and their spacings and widths are recorded. The bond stresses vary along the bar length between the cracks. The strain in the steel bar is maximum at the cracked section and decreases toward the middle section between cracks.

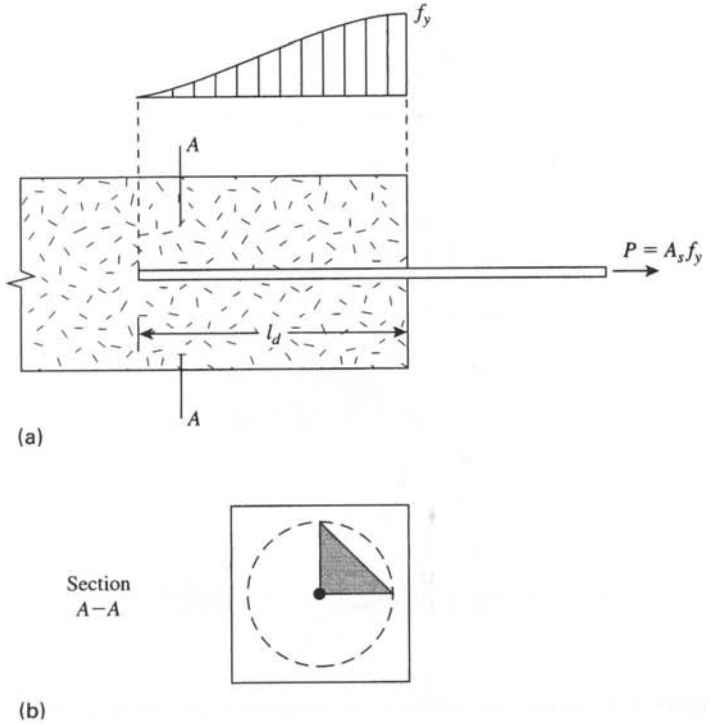


Figure 7.2 Bond stresses and development length. (*a*) Distribution of stress along l_d and (*b*) radial stress in concrete around the bar.

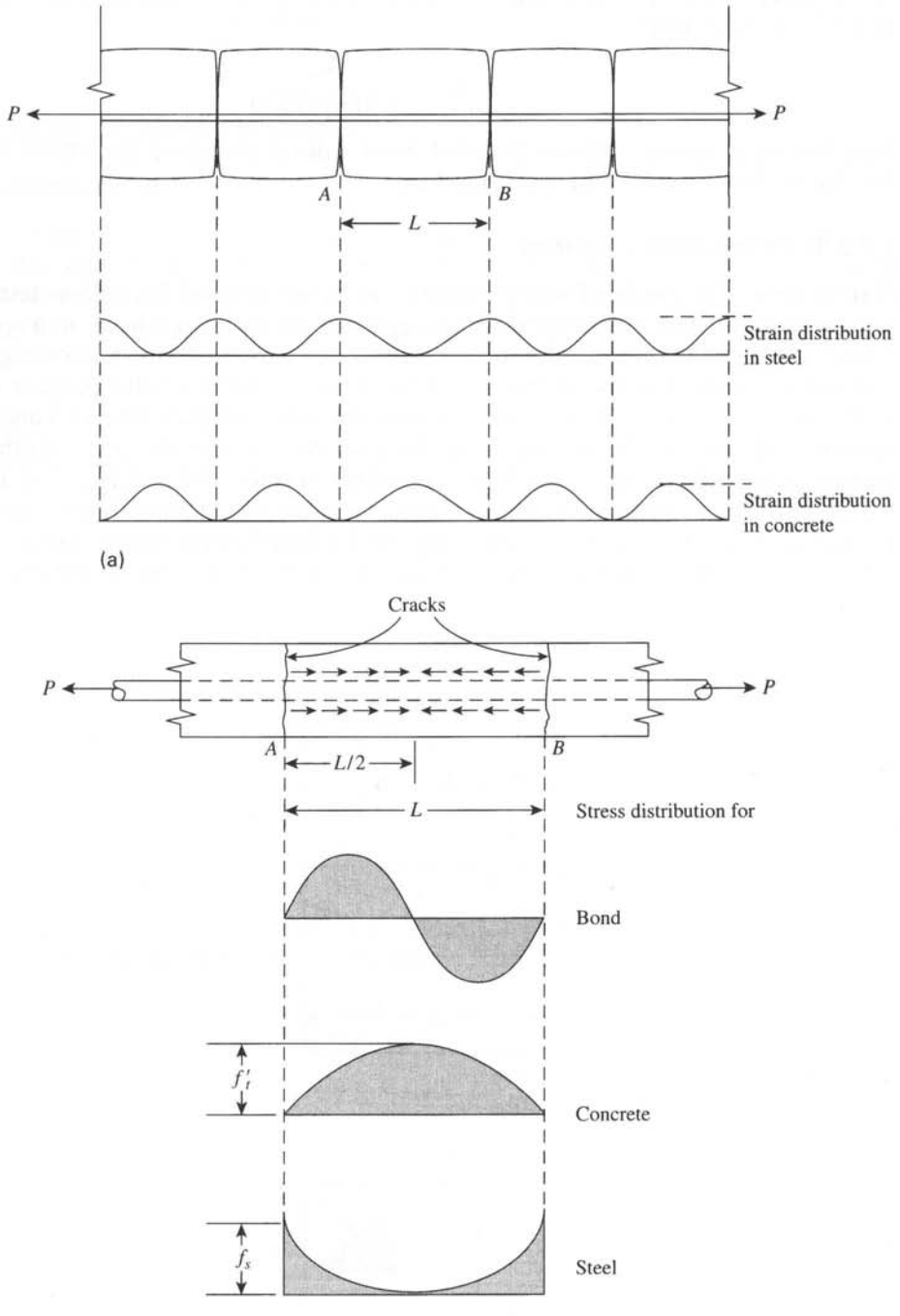


Figure 7.3 Bond mechanism in an embedded bar. Strain (*a*) and stress (*b*) distribution between cracks.

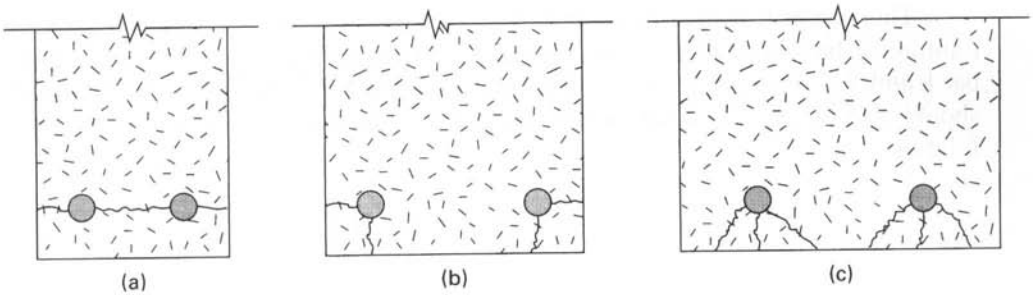


Figure 7.4 Examples of spalling of concrete cover. (*a*) High bottom cover, (*b*) wide spacing, and (*c*) small bottom cover.

Tests on flexural members are also performed to study the bond effectiveness along the surface of the tension bars. The analysis of bond stresses in the bars of these members was explained earlier, and they are represented by Eq. 7.2.

Based on this discussion, it is important to choose an appropriate length in each reinforcing bar to develop its full yield strength without a failure in the bond strength. This length is called the *development length,* l_d. If this length is not provided, the bond stresses in the tension zone of a beam become high enough to cause cracking and splitting in the concrete cover around the tension bars (Fig. 7.4). If the split continues to the end of the bar, the beam will eventually fail. Note that small spacings between tensile bars and a small concrete cover on the sides and bottom will reduce the bond capacity of the reinforcing bars (Fig. 7.4).

7.3 DEVELOPMENT LENGTH IN TENSION

7.3.1 Development Length, l_d

If a steel bar is embedded in concrete, as shown in Fig. 7.2, and is subjected to a tension force T, then this force will be resisted by the bond stress between the steel bar and the concrete. The maximum tension force is equal to $A_s f_y$, where A_s is the area of the steel bar. This force is resisted by another internal force of magnitude $U_u O l_d$, where U_u is the ultimate average bond stress, l_d is the embedded length of the bar, and O is the perimeter of the bar (πD). The two forces must be equal for equilibrium:

$$A_s f_y = U_u O l_d \quad \text{and} \quad l_d = \frac{A_s f_y}{U_u O}$$

For a combination of bars,

$$l_d = \frac{A_s f_y}{U_u \Sigma O} \tag{7.3}$$

The length l_d is the minimum permissible anchorage length and is called the development length.

$$l_d = \frac{\pi d_b^2 f_y}{4U_u(\pi d_b)} = \frac{d_b f_y}{4U_u} \tag{7.4}$$

where d_b = diameter of reinforcing bars.

This means that the development length is a function of the size and yield strength of the reinforcing bars in addition to the ultimate bond stress, which in turn is a function of $\sqrt{f'_c}$. The bar length l_d given in Eq. 7.4 is called the *development length*, l_d. The final development length should also include the other factors mentioned in Section 7.1. Equation 7.4 may be written as follows:

$$\frac{l_d}{d_b} = K\left(\frac{f_y}{\sqrt{f'_c}}\right) \tag{7.5}$$

where K is a general factor that can be obtained from tests to include factors such as the bar characteristics (bar size, spacing, epoxy coated or uncoated, location in concrete section, and bar splicing), amount of transverse reinforcement, and the provision of excess reinforcement compared to that required from design.

The ACI Code, Section 12.2.3, evaluated K as follows:

$$K = \left(\frac{3}{40\lambda}\right)\frac{\psi_t\psi_e\psi_s}{\dfrac{(c_b + K_{tr})}{d_b}} \tag{7.6}$$

and Eq. 7.5 becomes

$$\frac{l_d}{d_b} = \frac{3}{40\lambda}\frac{f_y}{\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\dfrac{c_b + K_{tr}}{d_b}\right)} \tag{7.7}$$

where

ψ_t = bar location
ψ_e = coating factor
ψ_s = bar-size factor
λ = lightweight aggregate concrete factor (ACI Code, Section 8.6.1)
= 1.0 normal-weight concrete
= shall not exceed 0.75 unless splitting tensile strength is specified, then
$\lambda = f_{ct}/(6.7\sqrt{f'_c}) \leq 1$
c_b = spacing or cover dimension (in.), whichever is smaller
K_{tr} = transverse reinforcement index
$= 40A_{tr}/sn$
n = number of bars or wires being developed along the plane of splitting
s = maximum spacing of transverse reinforcement within l_d, center to center (in.).
f_{yt} = yield strength of transverse reinforcement (psi)
A_{tr} = total sectional area of all transverse reinforcement within spacing s that crosses the potential plane of splitting through to the reinforcement being developed (in.^{2})

Notes:

1. $(c_b + K_{tr})/d_b$ shall not exceed 2.5 to safeguard against pullout-type failures.
2. The value of $\sqrt{f'_c}$ shall not exceed 100 psi (ACI Code, Section 12.1.2).
3. $K_{tr} = 0$ can be used as a design simplification (ACI Code, Section 12.2.3).

7.3.2 ACI Code Factors for Calculating l_d for Bars in Tension

1. ψ_t = bar location factor
   ψ_t = 1.3 for top bars defined as horizontal reinforcement, placed so that more than 12 in. of fresh concrete is below the development length, or splice
   ψ_t = 1.0 for all other reinforcement
2. ψ_e = coating factor
   ψ_e = 1.5 for epoxy-coated bars or wires with cover less than $3d_b$ or clear spacing less than $6d_b$
   ψ_e = 1.2 for all other epoxy coated bars or wires
   ψ_e = 1.0 for uncoated and zinc-coated (galvanized) reinforcement (However, the value of the product $\psi_t\psi_e$ should not exceed 1.7)
3. ψ_s = bar size factor
   ψ_s = 0.8 for no. 6 bars or smaller bars and deformed wires
   ψ_s = 1.0 for no. 7 bars and larger bars
4. λ = lightweight aggregate concrete factor
   λ = λ shall not exceed 0.75 unless f_{ct} is specified
   λ = 1.0 for normal-weight concrete
5. The ACI Code permits using $K_{tr} = 0$ even if transverse reinforcement is present. In this case,

$$\frac{l_d}{d_b} = \left(\frac{3}{40\lambda}\right)\left(\frac{f_y}{\sqrt{f'_c}}\right)\left(\frac{\psi_t\psi_e\psi_s}{(c_b/d_b)}\right) \tag{7.7a}$$

   The value of $\sqrt{f'_c}$ should not exceed 100 psi.
6. R_s is the reduction factor due to excess reinforcement. The ACI Code, Section 12.2.5, permits the reduction of l_d by the factor R_s when the reinforcement in a flexural member exceeds that required by analysis, except where anchorage or development for f_y is specifically required or the reinforcement is designed considering seismic effects.

$$R_s = \frac{A_s \text{ (required)}}{A_s \text{ (provided)}}$$

7. The development length, l_d, in all cases shall not be less than 12 in.

7.3.3 Simplified Expressions for l_d

The ACI Code, Section 12.2.2, permits the use of simplified expressions to calculate the ratio l_d/d_b. This is based on the fact that current practical construction cases utilize spacing and cover values along with confining reinforcement, such as stirrups and ties, that produce a value of $(c_b + K_{tr})/d_b \geq 1.5$. Moreover, tests indicated that the development length, l_d, can be reduced by 20% for no. 6 and smaller bars. Based on these assumptions and assuming $(c_b + K_{tr})/d_b = 1.5$, Eq. 7.7 can be reduced to the following expressions:

1. For no. 7 and larger bars,

$$\frac{l_d}{d_b} = \left(\frac{f_y}{\sqrt{f'_c}}\right)\frac{\psi_t\psi_e}{20\lambda} \tag{7.8}$$

   For no. 6 and smaller bars and deformed wires,

$$\frac{l_d}{d_b} = \left(\frac{f_y}{\sqrt{f'_c}}\right)\frac{\psi_t\psi_e}{25\lambda} \tag{7.9}$$

The ratio l_d/d_b in Eq. 7.9 represents 80% of that in Eq. 7.8. These equations are used when one of the following conditions is met:

a. 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.

b. Clear spacing of bars or wires being developed or spliced not less than $2d_b$ and clear cover not less than d_b.

2. For all other cases, the value of l_d/d_b in Eqs. 7.8 and 7.9 must be multiplied by 1.5 to restore them to equivalence with Eq. 7.7.

These equations are relatively simple to use for the general conditions involved in practical design and construction. For example, in all structures with normal-weight concrete ($\psi_t = 1.0$), uncoated reinforcement ($\psi_e = 1.0$), no. 7 *or larger* bars ($\psi_s = 1.0$), Eq. 7.8 becomes

$$\frac{l_d}{d_b} = \frac{f_y}{(20\lambda\sqrt{f'_c})} \tag{7.10}$$

This equation is used when conditions *a* and *b* are met, whereas for all other cases, l_d/d_b is multiplied by 1.5, or

$$\frac{l_d}{d_b} = \frac{3f_y}{(40\lambda\sqrt{f'_c})} \tag{7.11}$$

Similarly, for the same conditions and for no. 6 *or smaller* bars, Eq. 7.9 becomes

$$\frac{l_d}{d_b} = \frac{f_y}{(25\lambda\sqrt{f'_c})} \tag{7.12}$$

This is used when conditions *a* and *b* are met; for all other cases, l_d/d_b is multiplied by 1.5, or

$$\frac{l_d}{d_b} = \frac{3f_y}{(50\lambda\sqrt{f'_c})} \tag{7.13}$$

It is quite common to use $f'_c = 4$ ksi and $f_y = 60$ ksi in the design and construction of reinforced concrete buildings. If these values are substituted in the preceding equations, and assuming normal-weight concrete ($\lambda = 1.0$) then

Equation 7.10 becomes	$l_d = 47.5d_b$	($\geq$ no. 7 bars).	(7.10a)
Equation 7.11 becomes	$l_d = 71.2d_b$	($\geq$ no. 7 bars).	(7.11a)
Equation 7.12 becomes	$l_d = 38d_b$	($\leq$ no. 6 bars).	(7.12a)
Equation 7.13 becomes	$l_d = 57d_b$	($\leq$ no. 6 bars).	(7.13a)

Other values of l_d/d_b ratios are shown in Table 7.1. Table 7.2 gives the development length, l_d, for different reinforcing bars (when $f_y = 60$ ksi and $f'_c = 3$ ksi and 4 ksi) for both cases, when conditions *a* and *b* are met and for all other cases.

7.4 DEVELOPMENT LENGTH IN COMPRESSION

The development length of deformed bars in compression is generally smaller than that required for tension bars, due to the fact that compression bars do not have the cracks that develop in tension concrete members that cause a reduction in the bond between bars and the surrounding

Table 7.1 Values of l_d/d_b for Various Values of f'_c and f_y (Tension Bars), ($\lambda = 1.0$)

	$f_y = 40$ ksi				$f_y = 60$ ksi			
	No. 6 Bars		≥ No. 7 Bars		No. 6 Bars		≥ No. 7 Bars	
f'_c (ksi)	Conditions met	Other cases	Conditions met	Other cases	Conditions met	Other cases	Conditions met	Other cases
3	29.3	43.9	36.6	54.8	43.9	65.8	54.8	82.2
4	25.3	38.0	31.7	47.5	38.0	57.0	47.5	71.2
5	22.7	34.0	28.3	42.5	34.0	51.0	42.5	63.7
6	20.7	31.0	25.9	38.8	31.0	46.5	38.8	58.1

Table 7.2 Development Length l_d (in.) for Tension Bars and $f_y = 60$ ksi ($\psi_t = \psi_e = \lambda = 1.0$)

		Development Length l_d (in.) — Tension Bars			
		$f'_c = 3$ksi		$f'_c = 4$ ksi	
Bar number	Bar diameter (in.)	Conditions met	Other cases	Conditions met	Other cases
3	0.375	17	25	15	21
4	0.500	22	33	19	29
5	0.625	28	41	24	36
6	0.750	33	50	29	43
7	0.875	48	72	42	63
8	1.000	55	83	48	72
9	1.128	62	93	54	81
10	1.270	70	105	61	92
11	1.410	78	116	68	102

concrete. The ACI Code, Section 12.3.2, gives the basic development length in compression for all bars as follows:

$$l_{dc} = \frac{0.02 d_b f_y}{\lambda \sqrt{f'_c}} \geq 0.0003 d_b f_y \tag{7.14}$$

which must not be less than 8 in. The development length, l_{dc}, may be reduced by multiplying l_{dc} by $R_s = (A_s \text{ required})/(A_s \text{ provided})$. For spirally reinforced concrete compression members with spirals of not less than $\frac{1}{4}$ in. diameter and a spacing of 4 in. or less, the value of l_{dc} in Eq. 7.14 may be multiplied by $R_{sl} = 0.75$. In general, $l_d = l_{dc} \times (R_s \text{ or } R_{sl}, \text{ if applicable}) \geq 8$ in. Tables 7.3 and 7.4 give the values of l_{dc}/d_b when $f_y = 60$ ksi.

Table 7.3 Values of l_d/d_b for Various Values of f'_c and f_y (Compression Bars), $\lambda = 1.0$, Minimum $l_{dc} = 8$ in. $l_{dc}/d_b = 0.02 f_y/\lambda\sqrt{f'_c} \geq 0.0003 f_y$

f'_c(ksi)	3	4	5 or more
$f_y = 40$ ksi	15	13	12
$f_y = 60$ ksi	22	19	18

Table 7.4 Development Length, l_{dc} (in.), for Compression Bars ($f_y = 60$ ksi), $\lambda = 1.0$

Bar number	Bar diameter (in.)	Development Length, l_{dc} (in.) when f'_c =		
		3 (ksi)	4 (ksi)	5 (ksi) or more
3	0.375	9	8	8
4	0.500	11	10	9
5	0.625	14	12	12
6	0.750	17	15	14
7	0.875	20	17	16
8	1.000	22	19	18
9	1.128	25	22	21
10	1.270	28	25	23
11	1.410	31	27	26

7.5 SUMMARY FOR THE COMPUTATION OF l_d IN TENSION

Assuming normal construction practices, $(c_b + K_{tr})/d_b = 1.5$.

1. If one of the following two conditions is met:
   a. Clear spacing of bars $\geq d_b$, clear cover $\geq d_b$, and bars are confined with stirrups not less than the code minimum.
   b. Clear spacing of bars $\geq 2d_b$ and clear cover $\geq d_b$; then

$$\text{for no. 7 and larger bars, } \frac{l_d}{d_b} = \frac{\psi_t \psi_e f_y}{20\lambda\sqrt{f'_c}} \tag{7.8}$$

$$\text{for no. 6 or smaller bars, } \frac{l_d}{d_b} = \frac{\psi_t \psi_e f_y}{25\lambda\sqrt{f'_c}} \tag{7.9}$$

2. For all other cases, multiply these ratios by 1.5.
3. Note that $f'_c \leq 100$ psi and $\psi_t \psi_e \leq 1.7$; values of ψ_t, ψ_e, and λ are as explained earlier.
4. For bundled bars, either in tension or compression, l_d should be increased by 20% for three-bar bundles and by 33% for four-bar bundles. A unit of bundled bars is considered a single bar of a diameter and area equivalent to the total area of all bars in the bundle. This equivalent diameter is used to check spacings and concrete cover.

Example 7.1

Figure 7.5 shows the cross-section of a simply supported beam reinforced with four no. 8 bars that are confined with no. 3 stirrups spaced at 6 in. Determine the development length of the bars if the beam is made of normal-weight concrete, bars are not coated, $f'_c = 3$ ksi, and $f_y = 60$ ksi.

Solution

1. Check if conditions for spacing and concrete cover are met:
   a. For no. 8 bars, $d_b = 1.0$ in.
   b. Clear cover $= 2.5 - 0.5 = 2.0$ in. $> d_b$

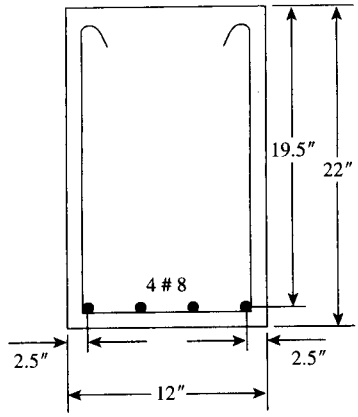


Figure 7.5 Example 7.1.

c. Clear spacing between bars $= \dfrac{12-5}{3} - 1.0 = 1.33$ in. $> d_b$

d. Bars are confined with no. 3 stirrup. The conditions are met. Then

$$\frac{l_d}{d_b} = \frac{\psi_t \psi_e f_y}{20\lambda\sqrt{f'_c}} \quad \text{(for bars > no. 7)} \tag{7.8}$$

2. Determine the multiplication factors: $\psi_t = 1.0$ (bottom bars), $\psi_e = 1.0$ (no coating), and $\lambda = 1.0$ (normal-weight concrete). Also check that $\sqrt{f'_c} = 54.8$ psi < 100 psi.

$$\frac{l_d}{d_b} = \frac{60,000}{(20 \times 1 \times \sqrt{3000})} = 54.8$$

So, $l_d = 54.8(1.0) = 54.8$ in., say, 55 in. These values can be obtained directly from Tables 7.1 and 7.2. Note that if the general formula for l_d/d_b (Eq. 7.7) is used, assuming $K_{tr} = 0$, then

$$\frac{l_d}{d_b} = \left(\frac{3}{40\lambda}\right)\left(\frac{f_y}{\sqrt{f'_c}}\right)\left(\frac{\psi_t \psi_e}{c_b/d_b}\right) \tag{7.7}$$

In this example, $\psi_t = \psi_e = \lambda = 1$.
Also, c_b = smaller of distance from center of bar to the nearest concrete surface (c_1) or one-half the center-to-center of bars spacing (c_2).

$$c_1 = 2.5 \text{ in.} \quad c_2 = \frac{0.5(12-5)}{3} = 1.17 \text{ in. (controls)}$$

$f_y/(c_b + K_{tr})/d_b = 1.17/1.0 = 1.17 < 1.5$, so use $(c_b + K_{tr})/d_b = 1.5$. Consequently, $l_d/d_b = 60,000(20\lambda\sqrt{f'_c})$ as in step 2, and $l_d = 55$ in.
Note : If the bars are not confined by stirrups, this value of l_d must be multiplied by 1.5 ($s = 1.33$ in. $< 2d_b = 2.0$ in.).

Example 7.2

Repeat Example 7.1 if the beam is made of lightweight aggregate concrete, the bars are epoxy coated, and A_s required from analysis is 2.79 in.^{2}

Solution

1. Determine the multiplication factors: $\psi_t = 1.0$ (bottom bars), $\psi_e = 1.5$ (epoxy coated), $\lambda = 0.75$ (lightweight aggregate concrete), and $R_s = (A_s \text{ required})/(A_s \text{ provided}) = 2.79/3.14 = 0.89$. The value of ψ_e is 1.5, because the concrete cover is less than $3d_b = 3$ in. Check that $\psi_t \psi_e = 1.0(1.5) = 1.5 < 1.7$.
2. $$\frac{l_d}{d_b} = \frac{R_s \psi_t \psi_e f_y}{20\lambda\sqrt{f'_c}} \text{ (for bars > no. 7)}$$
$$= \frac{0.89(1.0)(1.5)(60,000)}{((20)(0.75)\sqrt{3000})} = 73.1 \text{ in.}, \quad \text{say, 74 in.}$$
3. The development length l_d can be obtained from Table 7.2 ($l_d = 55$ in. for no. 8 bars) and then divided by the factor 0.75.

Example 7.3

A reinforced concrete column is reinforced with eight no. 10 bars, which should extend to the footing. Determine the development length needed for the bars to extend down in the footing. Use normal-weight concrete with $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

The development length in compression is

$$l_{dc} = \frac{0.02 d_b f_y}{\lambda\sqrt{f'_c}} \geq 0.0003 d_b f_y$$

$$l_{dc} = \frac{0.02(1.27)(60,000)}{(1)\sqrt{4000}} = 24.1 \text{ in.} \quad \text{(controls)}$$

The minimum l_{dc} is 0.0003(1.27)(60,000) = 22.86 in., but it cannot be less than 8 in. Because there are no other multiplication factors, then $l_d = 24.1$ in., or 25 in. (The same value is shown in Table 7.4.)

7.6 CRITICAL SECTIONS IN FLEXURAL MEMBERS

The critical sections for development of reinforcement in flexural members are

- At points of maximum stress
- At points where tension bars within the span are terminated or bent
- At the face of the support
- At points of inflection at which moment changes signs

The critical sections for a typical uniformly loaded continuous beam are shown in Fig. 7.6. The sections and the relative development lengths are explained as follows:

1. Three sections are critical for the negative moment reinforcement: Section 1 is at the face of the support, where the negative moment as well as stress are at maximum values. Two development lengths, x_1 and x_2, must be checked.

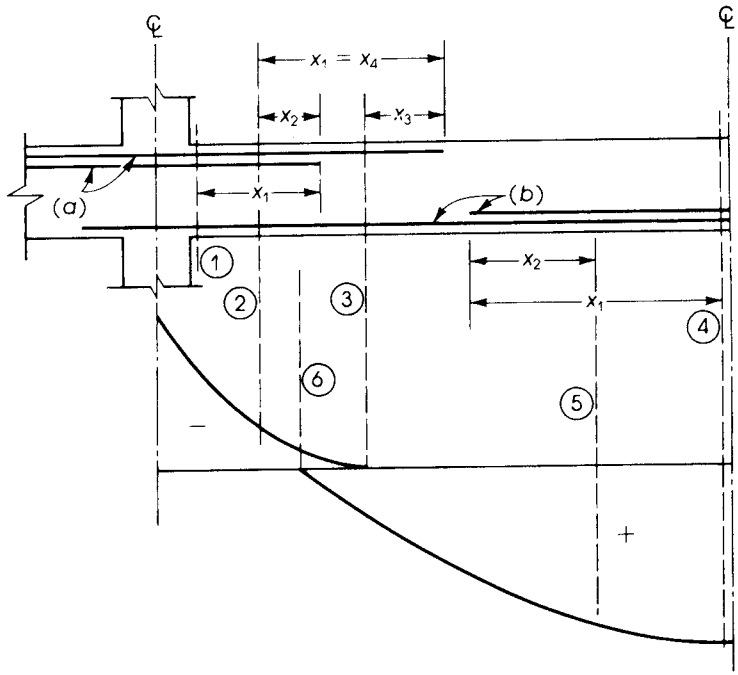


Figure 7.6 Critical sections (circled numbers) and development lengths ($X_1 - X_4$).

Section 2 is the section where part of the negative reinforcement bars can be terminated. To develop full tensile force, the bars should extend a distance x_2 before they can be terminated. Once part of the bars are terminated, the remaining bars develop maximum stress.

Section 3 is at the point of inflection. The bars shall extend a distance x_3 beyond section 3:x_3 must be equal to or greater than the effective depth, d, 12 bar diameters, or $\frac{1}{16}$ clear span, whichever is greater. At least one-third of the total reinforcement provided for negative moment at the support shall be extended a distance x_3 beyond the point of inflection, according to the ACI Code, Section 12.12.3.

2. Three sections are critical for positive moment reinforcement: Section 4 is that of maximum positive moment and maximum stresses. Two development lengths, x_1 and x_2, have to be checked. The length x_1 is the development length l_d specified by the ACI Code, Section 12.11, as mentioned later. The length x_2 is equal to or greater than d or 12 bar diameters.

Section 7.5 is where part of the positive reinforcement bars may be terminated. To develop full tensile force, the bars should extend a distance x_2. The remaining bars will have a maximum stress due to the termination of part of the bars. At the face of support, section 7.1, at least one-fourth of the positive moment reinforcement in continuous members shall extend along the same face of the member into the support, according to the ACI Code, Section 12.11.1. For simple members, at least one-third of the reinforcement shall extend into the support.

At points of inflection, section 7.6, limits are according to Section 12.11.3 of the ACI Code.

Example 7.4

A continuous beam has the bar details shown in Fig. 7.7. The bending moments for maximum positive and negative moments are also shown. We must check the development lengths at all critical sections. Given: $f'_c = 3$ ksi normal-weight concrete, $f_y = 40$ ksi, $b = 12$ in., $d = 18$ in., and span $L = 24$ ft.

Solution

The critical sections are (1) at the face of the support for tension and compression reinforcement (section 1), (7.2) at points where tension bars are terminated within the span (sections 2 and 5), (3) at point of inflection (sections 3 and 6), and (4) at midspan (section 4).

1. Development lengths for negative-moment reinforcement, from Fig. 7.7, are as follows: Three no. 9 bars are terminated at a distance $x_1 = 4.5$ ft from the face of the support, whereas the other three bars extend to a distance of 6 ft 0 in. (72 in.) from the face of the support.

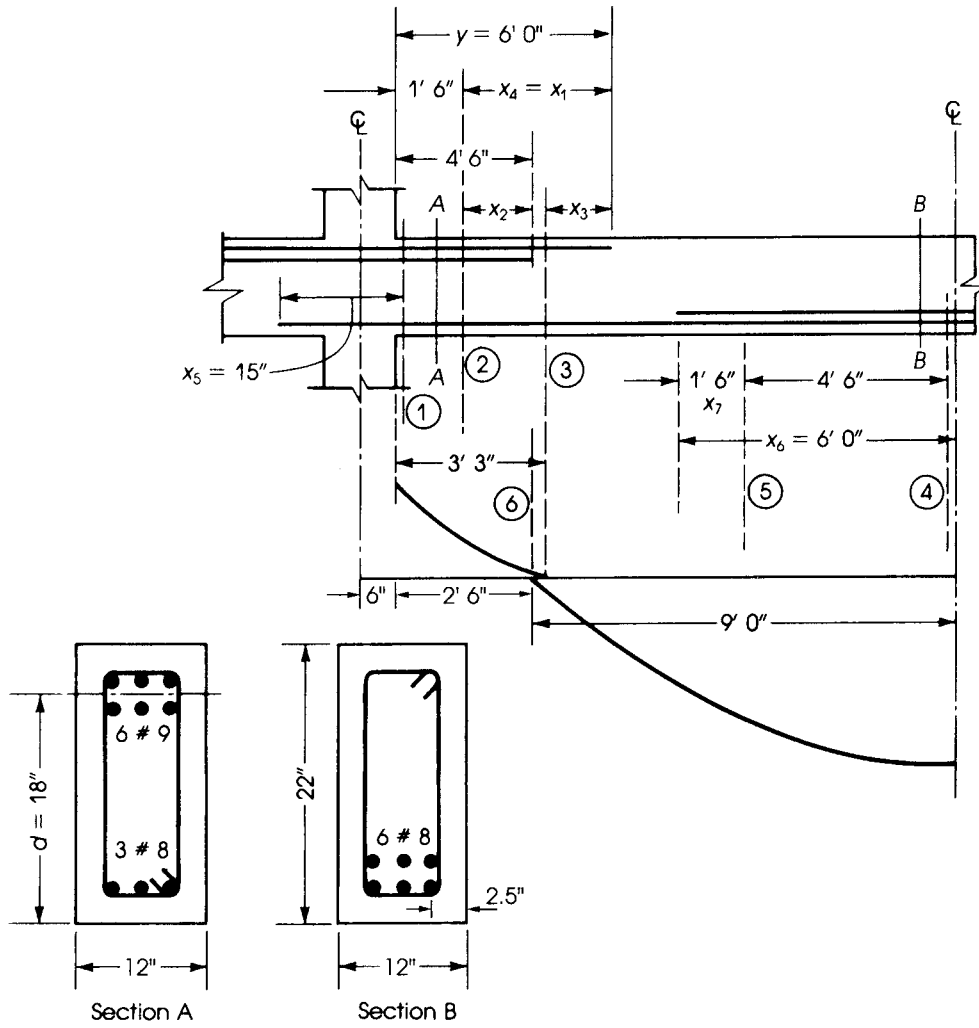


Figure 7.7 Example 7.4: Development length of a continuous beam.

a. The development length of no. 9 tension bars is $36.3d_b$ (Table 7.1) if conditions of spacing and cover are met.

$$\text{For no.9bars, } d_b = 1.128 \text{ in.}$$

$$\text{Cover} = 2.5 - \frac{1.128}{2} = 1.94 \text{ in.} > d_b$$

$$\text{Clear spacing} = \frac{12-5}{2} - 1.128 = 2.37 \text{ in.} > 2d_b$$

Then conditions are met, and $l_d = 36.6(1.128) = 41.3$ in. For top bars, $x_1 = l_d = 1.3(41.3) = 54$ in. $= 4.50$ ft $= x_1 > 12$ in. (minimum).

b. The development length x_2 shall extend beyond the point where three no. 9 bars are not needed, either $d = 18$ in. or $12d_b = 13.6$ in., whichever is greater. Thus, $x_2 = 18$ in. The required development length is $x_4 = 4.50$ ft, similar to x_1. Total length required is $y = x_1 + 1.5$ ft $= 6.0$ ft.

c. Beyond the point of inflection (section 3), three no. 9 bars extend a length $x_3 = y - 39 = 72 - 39 = 33$ in. The ACI Code requires that at least one-third of the bars should extend beyond the inflection point. Three no. 9 bars are provided, which are adequate. The required development length of x_3 is the greatest of $d = 18$ in., $12d_b = 13.6$ in., or $L/16 = 24 \times \frac{12}{16}$ in. $= 18$ in., which is less than x_3 provided.

2. Compressive reinforcement at the face of the support (section 7.1) (no. 8 bars): The development length x_5 is equal to

$$l_{dc} = \frac{0.02d_b f_y}{\lambda\sqrt{f'_c}} = \frac{0.02 \times 1 \times 40,000}{1 \times \sqrt{3000}} = 14.6 \text{ in.}$$

So, we can use 15 in.

$$\text{Minimum } l_{dc} = 0.0003d_b f_y = 0.0003 \times 1 \times 40,000 = 12 \text{ in.}$$

but it cannot be less than 8 in. The length 15 in. controls. For no. 8 bars, $d_b = 1$ in.; l_{dc} provided $= 15$ in., which is greater than that required.

3. Development length for positive moment reinforcement: Three no. 8 bars extend 6 ft beyond the centerline, and the other bars extend to the support. The development length x_6 from the centerline is $l_d = 36.6d_b = 37$ in. (Table 7.1), but it cannot be less than 12 in. That is, x_6 provided is 6 ft $= 72$ in. > 37 in.

The length x_7 is equal to d or $12d_b$, that is, 18 in. or $12 \times 1 = 12$ in. The provided value is 18 in., which is adequate.

The actual position of the termination of bars within the span can be determined by the moment-resistance diagram, as will be explained later.

7.7 STANDARD HOOKS (ACI CODE, SECTIONS 12.5 AND 7.1)

A *hook* is used at the end of a bar when its straight embedment length is less than the necessary development length, l_d. Thus the full capacity of the bar can be maintained in the shortest distance of embedment. The minimum diameter of bend, measured on the inside of the main bar of a standard hook D_b, is as follows (Fig. 7.8) [[9]]:

- For no. 3 to no. 8 bars (10–25 mm), $D_b = 6d_b$.
- For no. 9 to no. 11 bars (28, 32, and 36 mm), $D_b = 8d_b$.
- For no. 14 and no. 18 bars (43 and 58 mm), $D_b = 10d_b$.

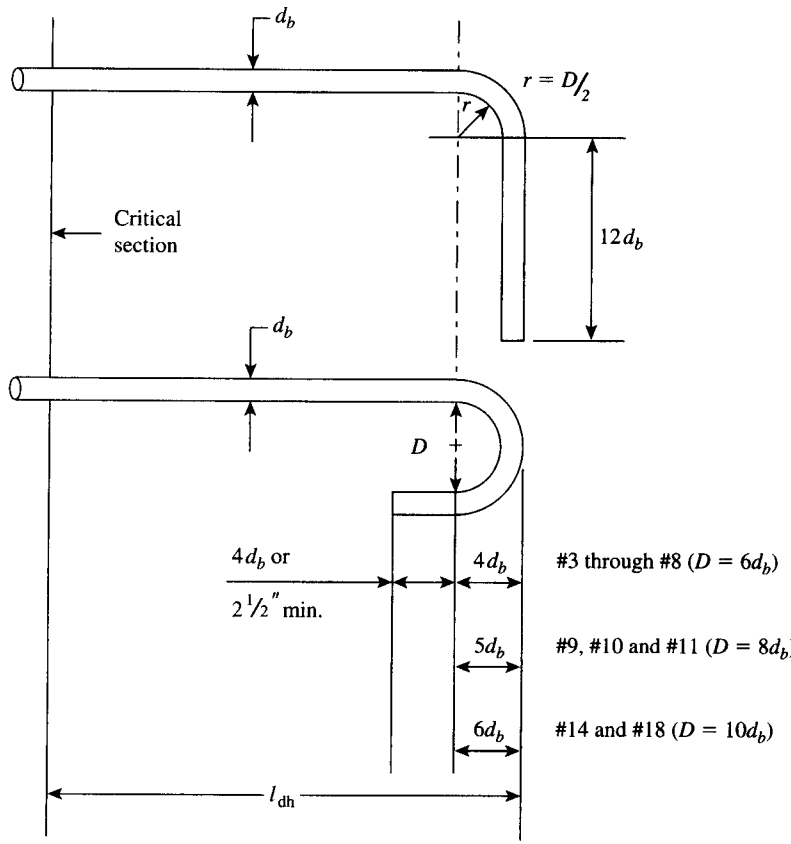


Figure 7.8 Hooked-bar details for the development of standard hooks [9]. Courtesy of ACI.

The ACI Code, Section 12.5.2, specifies a development length l_{dh} for hooked bar as follows:

$$l_{dh} = \left(\frac{0.02\psi_e f_y}{\lambda\sqrt{f'_c}}\right) (\text{Modification Factor})\, d_b \tag{7.15}$$

where

$\psi_e = 1.2$ for epoxy-coated bars

$\lambda = 0.75$ for lightweight aggregate concrete unless f_{ct} is specified then $\lambda = f_{ct}/(6.7(\sqrt{f'_c}) \leq 1$

ψ_e and $\lambda = 1.0$ for all other cases

For grade 60 hooked bar ($f_y = 60$ ksi) with $\psi_e = \lambda = 1$, l_{dh} becomes:

$$l_{dh} = \frac{1200 d_b}{\sqrt{f'_c}} (\text{Modification Factor})\, d_b \tag{7.15a}$$

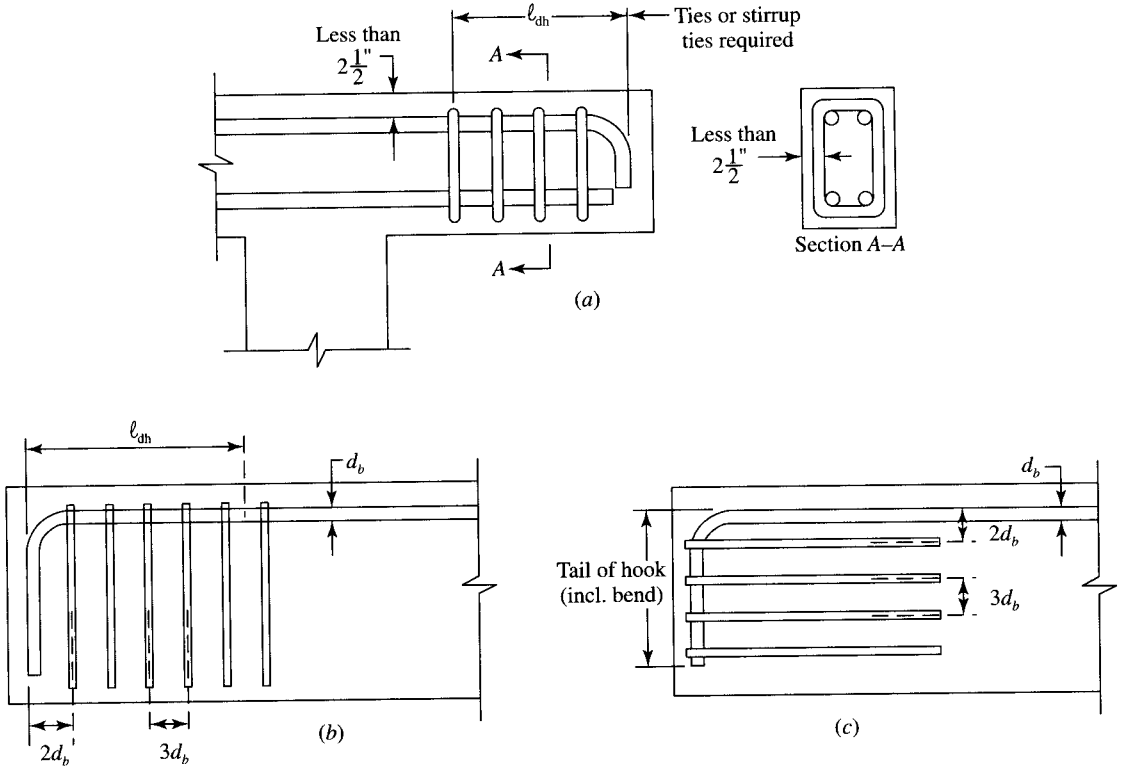


Figure 7.9 (*a*) Concrete cover limitations, (*b* and *c*) stirrups or ties placed perpendicular or parallel to the bar being developed [9]. Courtesy of ACI.

Based on different conditions, the development length, l_{dh}, must be multiplied by one of the following factors:

1. For 90° hooks of no. 11 or smaller bars are used and the hook is enclosed vertically or horizontally within stirrups or ties spaced not greater than three times the diameter of the hooked bar, the basic development length is multiplied by 0.8.
2. When no. 11 or smaller bars are used and the side concrete cover, normal to the plane of the hook, is not less than 2.5 in., the development length is multiplied by 0.7. The same factor applies for a 90° hook when the concrete cover on bar extension beyond the hook is not less than 2 in.
3. For 180° hooks of no. 11 or smaller bars that are enclosed with ties or stirrups perpendicular to the bar and spaced not greater than $3d_b$, the development length is multiplied by 0.8.
4. When a bar anchorage is not required, the basic development length for the reinforcement in excess of that required is multiplied by the ratio

$$\frac{A_s \text{ (required)}}{A_s \text{ (provided)}}$$

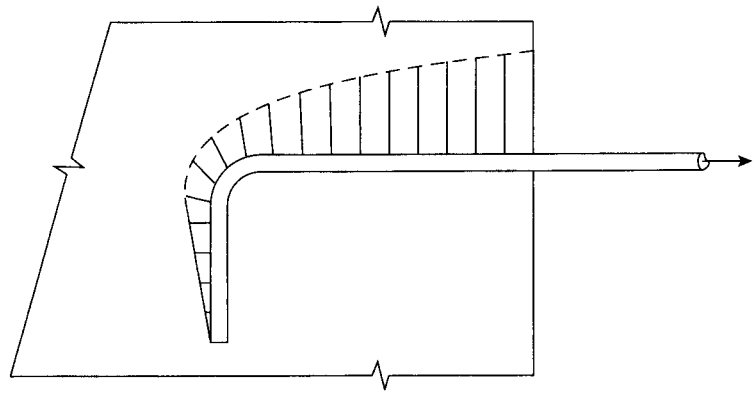


Figure 7.10 Stress distribution in 90° hooked bar.

5. When standard hooks with less than a 2.5-in. concrete cover on the side and top or bottom are used at a discontinuous end of a member, the hooks shall be enclosed by ties or stirrups spaced at no greater than $3d_b$. Moreover, the factor 0.8 given in item 1 shall not be used.

The development length, l_{dh}, of a standard hook for deformed bars in tension must not be less than $8d_b$ or 6 in., whichever is greater. Note that hooks are not effective for reinforcing bars in compression and may be *ignored* (ACI Code, Section 12.5).

Details of standard 90° and 180° hooks are shown in Fig. 7.8 [9]. The dimensions given are needed to protect members against splitting and spalling of concrete cover. Figure 7.9*a* shows details of hooks at a discontinuous end with a concrete cover less than 2.5 in. that may produce concrete spalling [9]. The use of closed stirrups is necessary for proper design. Figures 7.9*b* and *c* show placement of stirrups or ties perpendicular and parallel to the bar being developed, spaced along the development length. Figure 7.10 shows the stress distribution along a 90° hooked bar under a tension force p.

The development length required for deformed welded wire fabric is covered in Section 12.7 in the ACI Code. The basic development length (measured from the critical section) with at least one cross wire within the development length and not less than 2 in. shall be the greater of $(f_y - 35{,}000)/f_y$ (units in psi) or $5d_b/S_w$ but should not be taken greater than 1.0, where S_w = spacing of wire to be developed or spliced (in.).

Example 7.5

Compute the development length required for the top no. 8 bars of the cantilever beam shown in Fig. 7.11 that extend into the column support if the bars are

a. Straight
b. Have a 90° hook at the end
c. Have a 180° hook at the end

The bars are confined by no. 3 stirrups spaced at 6 in. and have a clear cover = 1.5 in. and clear spacings = 2.0 in. Use $f'_c = 4$ ksi normal-weight concrete and $f_y = 60$ ksi.

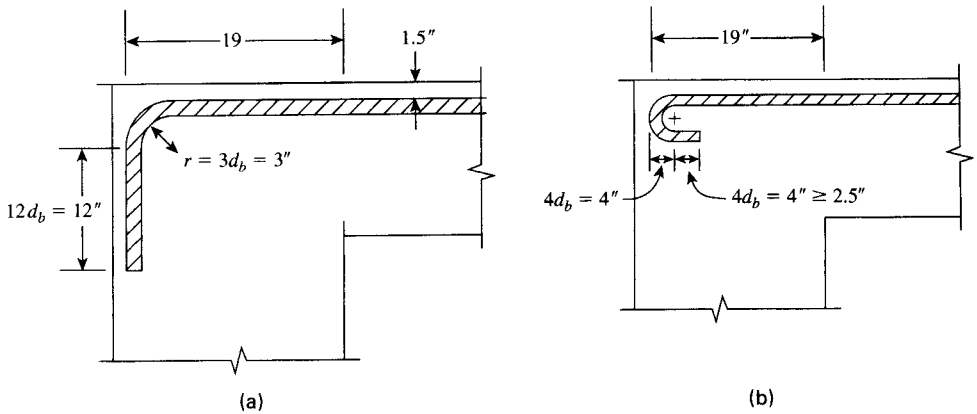


Figure 7.11 Example 7.5.

Solution

a. Straight bars: For no. 8 bars, $d_b = 1.0$ in. Because clear spacing $= 2d_b$ and clear cover is greater than d_b with bars confined by stirrups, then conditions *a* and *b* are met. Equation 7.10 can be used to calculate the basic l_d or you can get it directly from Table 7.2: $l_d = 48$ in. For top bars, $\psi_e = 1.3$ and final $l_d = 1.3(48) = 63$ in.

b. Bars with 90° hook: For no. 8 bars, $d_b = 1.0$ in. development length for $f_y = 60$ ksi $l_{dh} = 1200d_b/\sqrt{f'_c} = 1200(1.0)/\sqrt{4000} = 19$ in. Because no other modifications apply, then $l_{dh} = 19$ in. $> 8d_b = 8$ in. or 6 in. Other details are shown in Fig. 7.11. The factor $\psi_e = 1.3$ for top bars does not apply to hooks.

c. Bars with 180° hook: $l_{dh} = 19$ in., as calculated before. No other modifications apply; then $l_{dh} = 19$ in. $> 8d_b = 8$ in. Other details are shown in Fig. 7.11.

7.8 SPLICES OF REINFORCEMENT

7.8.1 General

Steel bars that are used as reinforcement in structural members are fabricated in lengths of 20, 40, and 60 ft (6, 12, and 18 m), depending on the bar diameter, transportation facilities, and other reasons. Bars are usually tailored according to the reinforcement details of the structural members. When some bars are short, it is necessary to splice them by lapping the bars a sufficient distance to transfer stress through the bond from one bar to the other.

Splices may be made by lapping or welding or with mechanical devices that provide positive connection between bars. Lap splices should not be used for bars larger than no. 11 (36 mm). For noncontact lap splices in flexural members, bars should not be spaced transversely farther apart than one-fifth the required length or 6 in. (150 mm). An approved welded splice is one in which the bars are butted and welded to develop in tension at least 125% of the specified yield strength of the bar. The ACI Code, Section 12.14, also specifies that full positive mechanical connections must develop in tension or compression at least 125% of the specified yield strength of the bar.

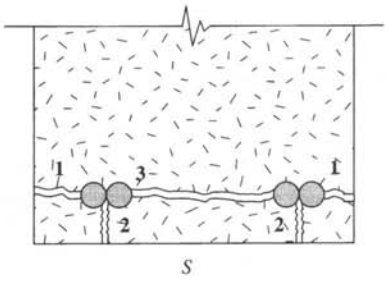


Figure 7.12 Lap splice failure due to the development of one or more cracks.

Splices should not be made at or near sections of maximum moments or stresses. Also, it is recommended that no bars should be spliced at the same location to avoid a weakness in the concrete section and to avoid the congestion of bars at the same location, which may cause difficulty in placing the concrete around the bars.

The stresses developed at the end of a typical lap splice are equal to 0, whereas the lap length, l_d, embedded in concrete is needed to develop the full stress in the bar, f_y. Therefore, a minimum lap splice of l_d is needed to develop a continuity in the spliced tension or compression bars. If adequate splice length is not provided, splitting and spalling occurs in the concrete shell (Fig. 7.12).

Splices in tension and compression are covered by Sections 12.15 and 12.16 of the ACI Code.

7.8.2 Lap Splices in Tension, l_{st}

Depending upon the percentage of bars spliced on the same location and the level of stress in the bars or deformed wires, the ACI Code introduces two classes of splices (with a minimum length of 12 in.):

1. Class A splices: These splices have a minimum length $l_{st} = l_d$ and are used when (a) one-half or less of the total reinforcement is spliced within the required lap length; and (b) the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice. The length l_d is the development length of the bar, as calculated earlier.
2. Class B splices: These splices have a minimum length $l_{st} = 1.3l_d$ and are used for all other cases that are different from the aforementioned conditions. For example, class B splices are required when all bars or deformed wires are spliced at the same location with any ratio of (A_s provided)/(A_s required). Splicing all the bars in one location should be avoided when possible.
3. l_d in class A and B splice is calculated without the 12 in. minimum requirement and without the modification factor of (A_s required)/(A_s provided).

7.8.3 Lap Splice in Compression, l_{sc}

The splice lap length of the reinforcing bars in compression, l_{sc}, should be equal to or greater than the development length of the bar in compression, l_d (including the modifiers), calculated earlier (Eq. 7.14).Moreover, the lap length shall satisfy the following (ACI Code, Section 12.16.1):

Table 7.5 Lap-Splice Length in Compression, l_{sc}(in.), ($f'_c \geq$ 3ksi and Minimum l_{sc} = 12 in.)

Bar number	Bar diameter (in.)	f_y (ksi)		
		40	60	80
3	0.375	12	12	18
4	0.500	12	15	24
5	0.625	13	19	30
6	0.750	15	23	36
7	0.875	18	27	42
8	1.000	20	30	48
9	1.128	23	34	55
10	1.270	26	39	61
11	1.410	29	43	68

$$l_{sc} \geq (0.0005 f_y d_b) \qquad (\text{for } f_y \leq 60{,}000 \text{ psi}) \tag{7.16}$$

$$l_{sc} = (0.0009 f_y - 24) d_b \quad (\text{for } f_y > 60{,}000 \text{ psi}) \tag{7.17}$$

For all cases, the lap length must not be less than 12 in. Table 7.5 gives the lap-splice length for various f_y values. If the concrete strength, f'_c, is less than 3000 psi, the lap length, l_{sc}, must be increased by one-third.

In spirally reinforced columns, lap-splice length within a spiral may be multiplied by 0.75 but may not be less than 12 in. In tied columns, with ties within the splice length having a minimum effective area of $0.0015hs$, lap splice may be multiplied by 0.83 but may not be less than 12 in., where h = overall thickness of column and s = spacing of ties (in.).

Example 7.6

Calculate the lap-splice length for six no. 8 tension bottom bars (in two rows) with clear spacing = 2.5 in. and clear cover = 1.5 in. for the following cases:

a. When three bars are spliced and (A_s provided)/(A_s required) > 2

b. When four bars are spliced and (A_s provided)/(A_s required) < 2

c. When all bars are spliced at the same location. Given: f'_c = 5 ksi and f_y = 60 ksi.

Solution

a. For no. 8 bars, d_b = 1.0 in., and $\psi_t = \psi_e = \lambda = 1.0$.: check first for $\sqrt{5000} = 70.7$ psi < 100 psi, and then calculate l_d from Equation 7.8 or Table 7.1, $l_d = 42.5 d_b$, conditions for clear spacings and cover are met. $l_d = 42.5(1.0) = 42.5$ in., or 43 in. For (A_s provided)/(A_s required) > 2, class A splice applies, $l_{st} = 1.0 l_d = 43$ in. > 12 in. (minimum). Bars spliced are less than half the total number.

b. l_d = 43 in., as calculated before. Because (A_s provided)/(A_s required) is less than 2, class B splice applies, $l_{st} = 1.3 l_d = 1.3(42.5) = 55.25$ in., say, 56 in., which is greater than 12 in.

c. Class B splice applies and l_{st} = 56 in. > 12 in.

Example 7.7

Calculate the lap-splice length for a no. 10 compression bar in a tied column when $f'_c = 5$ ksi and when (a) $f_y = 60$ ksi and (b) $f_y = 80$ ksi.

Solution

a. For no. 10 bars, $d_b = 1.27$ in., and the development length from Table 7.4 or 7.3 is 23 in. Because no modifiers apply, $l_{sc} = 23$ in. > 12 in. Check that $l_{sc} \geq 0.0005 d_b f_y = 0.0005(1.27)(60{,}000) = 38.1$ in. Therefore, $l_{sc} = 39$ in. controls.

b. The basic l_d is 23 in., as calculated before. Check that $l_{sc} \geq (0.0009 f_y - 24) d_b = [0.0009(80{,}000) - 24](1.27) = 61$ in. Therefore, $l_{sc} = 61$ in. controls.

7.9 MOMENT-RESISTANCE DIAGRAM (BAR CUTOFF POINTS)

The moment capacity of a beam is a function of its effective depth, d, width, b, and the steel area for given strengths of concrete and steel. For a given beam, with constant width and depth, the amount of reinforcement can be varied according to the variation of the bending moment along the span. It is a common practice to cut off the steel bars where they are no longer needed to resist the flexural stresses. In some other cases, as in continuous beams, positive-moment steel bars may be bent up, usually at 45°, to provide tensile reinforcement for the negative moments over the supports.

The factored moment capacity of an under-reinforced concrete beam at any section is

$$M_u = \phi A_s f_y \left(d - \frac{a}{2}\right) \tag{7.18}$$

The lever arm $(d - a/2)$ varies for sections along the span as the amount of reinforcement varies; however, the variation in the lever arm along the beam length is small and is never less than the value obtained at the section of maximum bending moment. Thus, it may be assumed that the moment capacity of any section is proportional to the tensile force or the area of the steel reinforcement, assuming proper anchorage lengths are provided.

To determine the position of the cutoff or bent points, the moment diagram due to external loading is drawn first. A moment-resistance diagram is also drawn on the same graph, indicating points where some of the steel bars are no longer required. The factored moment resistance of one bar, M_{ub}, is

$$M_{ub} = \phi A_{sb} f_y \left(d - \frac{a}{2}\right) \tag{7.19}$$

where

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

A_{sb} = area of one bar

The intersection of the moment-resistance lines with the external bending moment diagram indicates the theoretical points where each bar can be terminated. To illustrate this discussion, Fig. 7.13 shows a uniformly loaded simple beam, its cross-section, and the bending moment diagram. The bending moment curve is a parabola with a maximum moment at midspan of

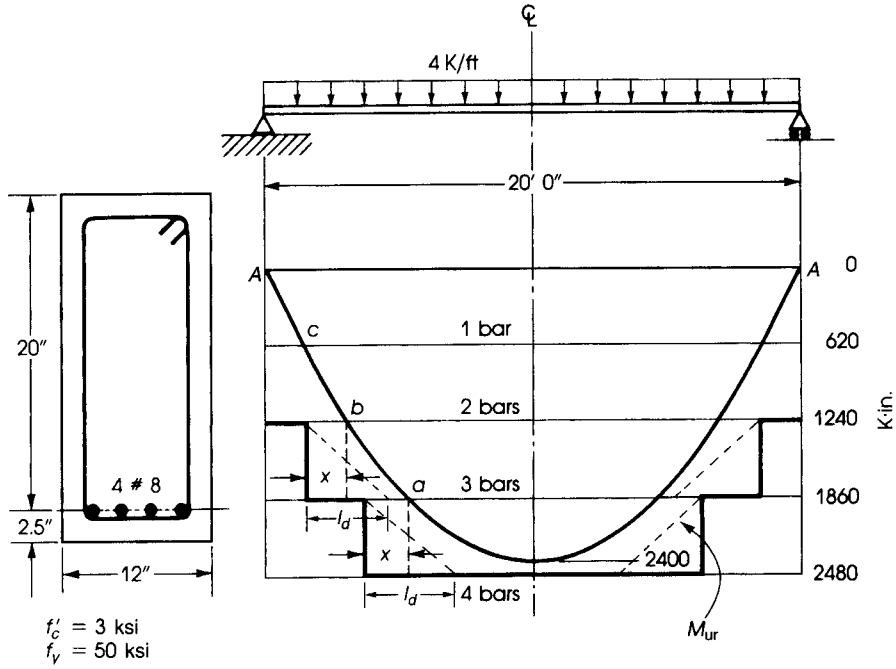


Figure 7.13 Moment–resistance diagram.

2400 K·in. Because the beam is reinforced with four no. 8 bars, the factored moment resistance of one bar is

$$M_{ub} = \phi A_{sb} f_y \left(d - \frac{a}{2}\right)$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4 \times 0.79 \times 50}{0.85 \times 3 \times 12} = 5.2 \text{ in.}$$

$$M_{ub} = 0.9 \times 0.79 \times 50 \left(20 - \frac{5.2}{2}\right) = 620 \text{ K·in.}$$

The factored moment resistance of four bars is thus 2480 K·in., which is greater than the external moment of 2400 K·in. If the moment diagram is drawn to scale on the base line *A*-*A*, it can be seen that one bar can be terminated at point *a*, a second bar at point *b*, the third bar at point *c*, and the fourth bar at the support end *a*. These points are the theoretical positions for the termination of the bars. However, it is necessary to develop part of the strength of the bar by bond, as explained earlier. The ACI Code specifies that every bar should be continued at least a distance equal to the effective depth, *d*, of the beam or 12 bar diameters, whichever is greater, beyond the theoretical points *a*, *b*, and *c*. The Code (Section 12.11.1) also specifies that at least one-third of the positive moment reinforcement must be continued to the support for simple beams. Therefore, for the example discussed here, two bars must extend into the support, and the moment-resistance diagram, M_{ur}, shown in Fig. 7.13, must enclose the external bending moment diagram at all points. Full load capacity of each bar is attained at a distance l_d from its end.

For continuous beams, the bars are bent at the required points and used to resist the negative moments at the supports. At least one-third of the total reinforcement provided for the negative moment at the support must be extended beyond the inflection points a distance not less than the effective depth, 12 bar diameters, or $\frac{1}{6}$ the clear span, whichever is greatest (ACI Code, Section 12.12.3).

Bent bars are also used to resist part of the shear stresses in beams. The moment–resistance diagram for a typical continuous beam is shown in Fig. 7.14.

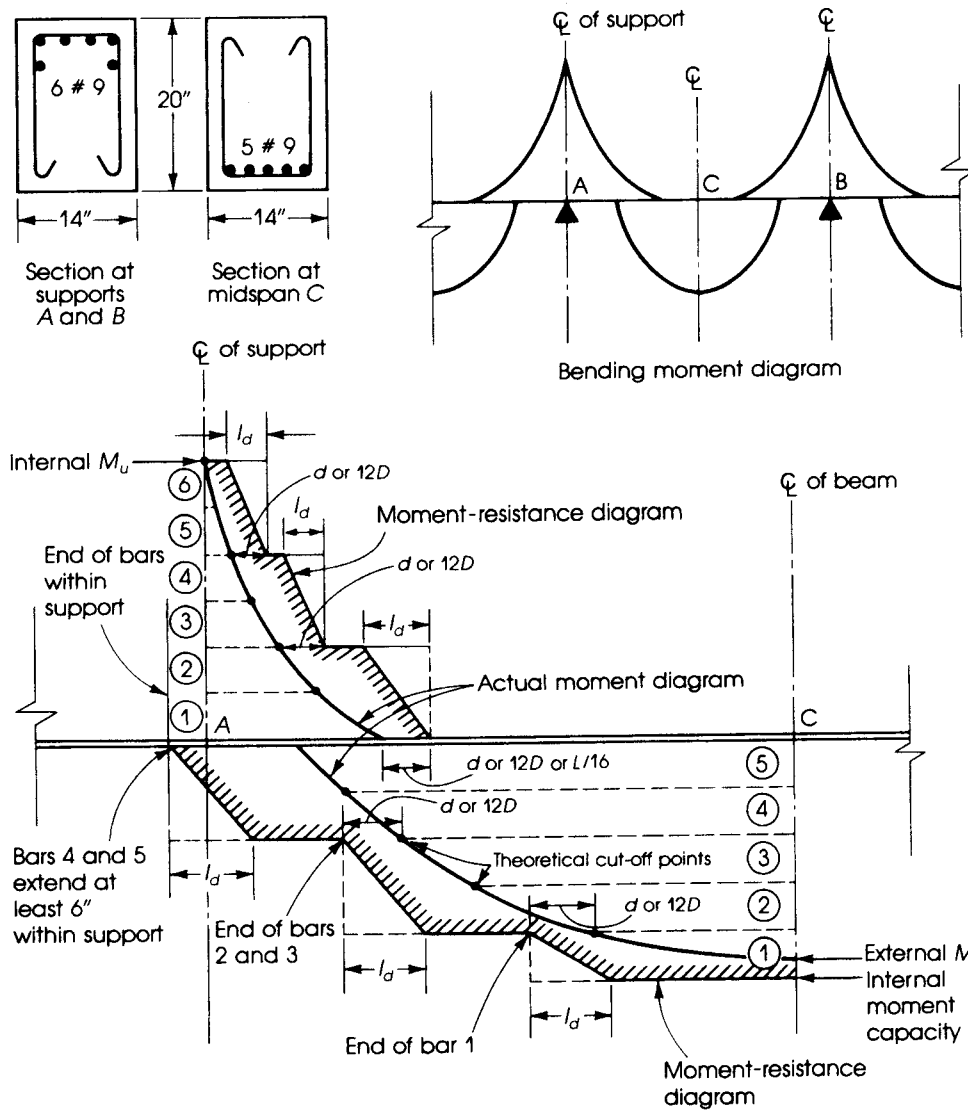


Figure 7.14 Sections and bending moment diagram (*top*) and moment–resistance diagram (*bottom*) of a continuous beam. Bar diameter is signified by *D*.

Example 7.8

For the simply supported beam shown in Fig. 7.15, design the beam for the given factored loads and draw the moment—resistance diagram. Also, show where the reinforcing bars can be terminated. Use $b = 10$ in., a steel ratio of 0.018, $f'_c = 3$ ksi, and $f_y = 40$ ksi.

Solution

For $\rho = 0.018$, $R_u = 556$ psi and $M_u = R_u bd^2$. $M_u = 132.5$ K-ft. Now $132.5(12) = 0.556(10)d^2$, so $d = 17$ in.; let $h = 20$ in. $A_s = 0.018(10)(17) = 3.06$ in.^{2}; use four no. 8 bars ($A_s = 3.14$ in.^{2}). Actual $d = 20 - 2.5 = 17.5$ in.

$$M_{ur} = \phi A_s f_y \left(d - \frac{a}{2}\right) \text{ and } a = \frac{3.14(40)}{0.85(3)(10)} = 4.93 \text{ in.}$$

$$M_{ur} \text{ (for one bar)} = 0.9(0.79)(40)\left(17.5 - \frac{4.93}{2}\right)$$

$$= 427.7 \text{K·in.} = 35.64 \text{ K·ft}$$

$$M_{ur} \text{ (for all four bars)} = 1710.8 \text{ K·in.} = 142.6 \text{ K·ft}$$

For the calculation of 'a', the four no. 8 bars were utilized rather than calculating the 'a' for the extended two bars. This assumption will slightly increase the length of the bars beyond the cutoff point.

Details of the moment–resistance diagram are shown in Fig. 7.15. Note that the bars can be bent or terminated at a distance of 17.5, say, 18 in. (or 12 bar diameters, whichever is greater), beyond the points where (theoretically) the bars are not needed. The development length, l_d, for no. 8 bars is $36.6d_b = 37$ in. (Table 7.1). The cutoff points of the first and second bars are at points A and B, but the actual points are at A' and B', 18 in. beyond A and B. From A', a length $l_d = 37$ in. backward is shown to establish the moment–resistance diagram (the dashed line). The end of the last two bars extending to the support will depend on how far they extend inside the support, say, at C'. Normally, bars are terminated within the span at A' and B' as bent bars are not commonly used to resist shear.

SUMMARY

Sections 7.1–7.2

Bond is influenced mainly by the roughness of the steel surface area, the concrete mix, shrinkage, and the cover of concrete. In general,

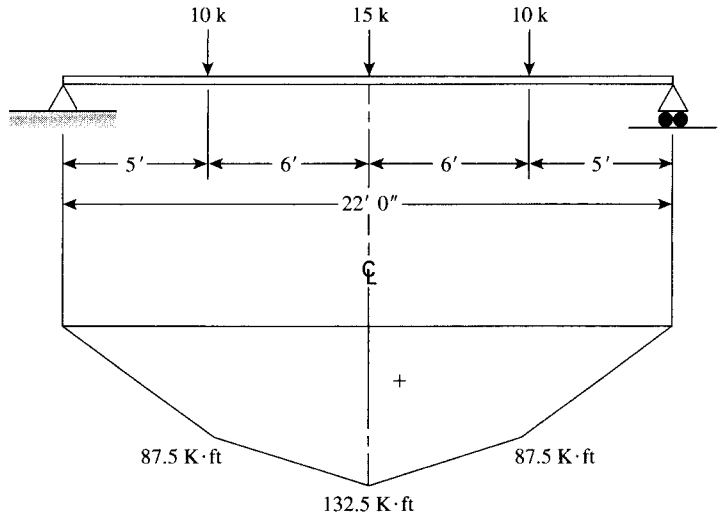
$$l_d = \frac{A_s f_y}{U_u \Sigma O} \tag{7.3}$$

Sections 7.3 and 7.5

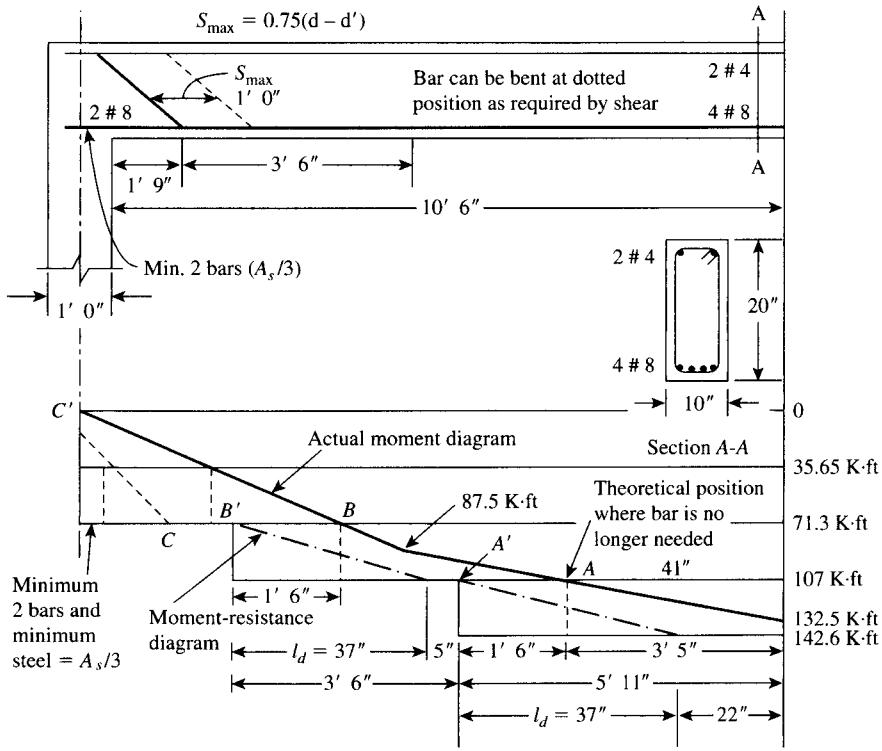
1. The general formula for the development length of deformed bars or wire shall be

$$\frac{l_d}{d_b} = \left(\frac{3}{40}\right)\left(\frac{f_y}{\lambda\sqrt{f'_c}}\right)\frac{\psi_t \psi_e \psi_s}{(c_b + k_{tr})/d_b} \tag{7.7}$$

As design simplification, K_{tr} may be assumed to be zero. Other values of l_d/d_b are given in Tables 7.1 and 7.2. ψ_t, ψ_e, ψ_s, and λ are multipliers defined in Section 7.3.1.



(a)



(b)

Figure 7.15 Example 7.8: Details of reinforcing bars and the moment–resistance diagram.

2. Simplified expressions are used when conditions for concrete cover and spacings are met. For no. 7 and larger bars,

$$\frac{l_d}{d_b} = \left(\frac{f_y}{\sqrt{f'_c}}\right)\left(\frac{\psi_t \psi_e}{20\lambda}\right) = Q \tag{7.8}$$

For no. 6 and smaller bars,

$$\frac{l_d}{d_b} = 0.8Q \tag{7.9}$$

3. For all other cases, multiply the previous Q by 1.5.
4. Minimum length is 12 in.

Section 7.4

Development length in compression for all bars is

$$l_d = \frac{0.02 d_b f_y}{\lambda\sqrt{f'_c}} \geq 0.0003 d_b f_y \geq 8 \text{ in.} \tag{7.14}$$

For specific values, refer to Tables 7.3 and 7.4.

Section 7.6

The critical sections for the development of reinforcement in flexural members are

- At points of maximum stress
- At points where tension bars are terminated within the span
- At the face of the support
- At points of inflection

Section 7.7

The minimum diameter of bends in standard hooks is

- For no. 3 to no. 8 bars, $6d_b$
- For no. 9 to no. 11 bars, $8d_b$

The development length l_{dh} of a standard hook is

$$l_{dh} = \left(\frac{0.02\psi f_y}{\lambda\sqrt{f'_c}}\right)(\text{Modification factor})\, d_b \tag{7.15}$$

Section 7.8

1. For splices in tension, the minimum lap-splice length is 12 in. If (a) one-half or less of the total reinforcement is spliced within the required lap length and (b) the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice, then $l_{st} = 1.0 l_d$ = class A splice.

2. For all other cases, class B has to be used when $l_{st} = 1.3l_d$.
3. For splices in compression, the lap length should be equal to or greater than l_{dc} in compression, but it also should satisfy the following: $l_{sc} \geq 0.0005\ f_y d_b$ (for $f_y \leq 60{,}000$ psi).

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PROBLEMS

7.1 For each assigned problem, calculate the development length required for the following tension bars. All bars are bottom bars in normal-weight concrete unless specified otherwise in the notes.

No.	Bar no.	f'_c (ksi)	f_y (ksi)	Clear cover (in.)	Clear spacing (in.)	Notes
a	5	3	60	2.0	2.25	
b	6	4	60	2.0	2.50	Lightweight aggregate concrete
c	7	5	60	2.0	2.13	Epoxy coated
d	8	3	40	2.5	2.30	Top bars, lightweight aggregate concrete
e	9	4	60	1.5	1.5	
f	10	5	60	2.0	2.5	No. 3 stirrups at 6 in.
g	11	5	60	3.0	3.0	
h	9	3	40	2.0	1.5	Epoxy coated
i	8	4	60	2.0	1.75	(A_s provided)/(A_s required) = 1.5
j	6	4	60	1.5	1.65	Top bars, epoxy coated and no. stirrup at 4 in.

7.2 For each assigned problem, calculate the development length required for the following bars in compression.

No.	Bar no.	f'_c (ksi)	f_y (ksi)	Notes
a	8	3	60	
b	9	4	60	
c	10	4	40	
d	11	5	60	(A_s required)/(A_s provided) = 0.8
e	7	6	60	(A_s required)/(A_s provided) = 0.9
f	9	5	60	Column with spiral no. 3 at 2 in.

7.3 Compute the development length required for the top no. 9 bars of a cantilever beam that extend into the column support if the bars are

a. Straight

b. Have a 90° hook at the end

c. Have a 180° hook at the end

The bars are confined with no. 3 stirrups spaced at 5 in. and have a clear cover of 2.0 in. Use $f'_c = 4$ ksi and $f_y = 60$ ksi. (Clear spacing = 2.5 in.)

7.4 Repeat Problem 7.3 when no. 7 bars are used.

7.5 Repeat Problem 7.3 when $f'_c = 3$ ksi and $f_y = 40$ ksi.

7.6 Repeat Problem 7.3 when no. 10 bars are used.

7.7 Calculate the lap-splice length for no. 9 tension bottom bars with clear spacing of 2.0 in. and clear cover of 2.0 in. for the following cases:

a. When 50% of the reinforcement is spliced and (A_s provided)/(A_s required) = 2

b. When 75% of the reinforcement is spliced and (A_s provided)/(A_s required) = 1.5

c. When all bars are spliced at one location and (A_s provided)/(A_s required) = 2

d. When all bars are spliced at one location and (A_s provided)/(A_s required) = 1.3. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

7.8 Repeat Problem 7.7 using $f'_c = 3$ ksi and $f_y = 60$ ksi.

7.9 Calculate the lap splice length for no. 9 bars in compression when $f'_c = 5$ ksi and $f_y = 60$ ksi.

7.10 Repeat Problem 7.9 when no. 11 bars are used.

7.11 Repeat Problem 7.9 when $f_y = 80$ ksi.

7.12 Repeat Problem 7.9 when $f'_c = 4$ ksi and $f_y = 60$ ksi.

7.13 A continuous beam has the typical steel reinforcement details shown in Fig. 7.16. The sections at midspan and at the face of the support of a typical interior span are also shown. Check the development lengths of the reinforcing bars at all critical sections. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

7.14 Design the beam shown in Fig. 7.17 using ρ_{max}. Draw the moment—resistance diagram and indicate where the reinforcing bars can be terminated. The beam carries a uniform dead load, including self-weight of 1.5 K/ft, and a live load of 2.2 K/ft. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, and $b = 12$ in.

7.15 Design the beam shown in Fig. 7.18 using a steel ratio $\rho = 1/2\ \rho_b$. Draw the moment—resistance diagram and indicate the cutoff points. Use $f'_c = 3$ ksi, $f_y = 60$ ksi, and $b = 12$ in.

7.16 Design the section at support B of the beam shown in Fig. 7.19, ρ_{max}. Adopting the same dimensions of the section at B for the entire beam ABC, determine the reinforcement required for part AB and draw the moment–resistance diagram for the beam ABC. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, and $b = 12$ in.

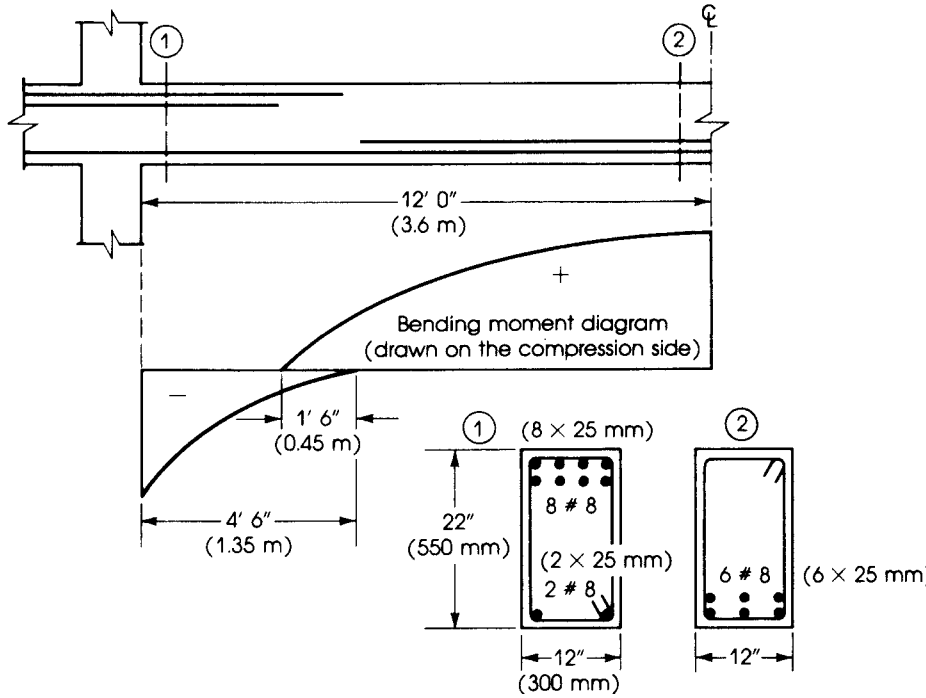


Figure 7.16 Problem 7.13.

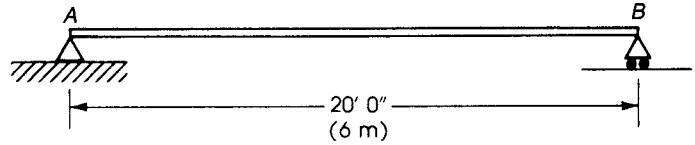


Figure 7.17 Problem 7.14: Dead load = 1.5 K/ft (22.5 kN/m), live load = 2.2 K/ft (33 kN/m).

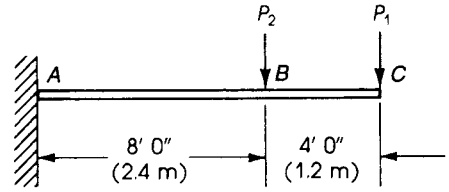


Figure 7.18 Problem 7.15: Dead load = 2 K/ft (30 kN/m), live load (concentrated loads only) is P_1 = 10 K (45 kN), P_2 = 16 K (72 kN).

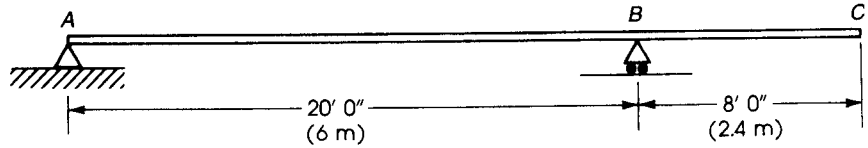


Figure 7.19 Problem 7.16: Dead load = 6 K/ft (90 kN/m), live load = 4 K/ft (60 kN/m).

CHAPTER 8

SHEAR AND DIAGONAL TENSION



Office building, Chicago, Illinois.

8.1 INTRODUCTION

When a simple beam is loaded as shown in Fig. 8.1, bending moments and shear forces develop along the beam. To carry the loads safely, the beam must be designed for both types of forces. Flexural design is considered first to establish the dimensions of the beam section and the main reinforcement needed, as explained in the previous chapters.

The beam is then designed for shear. If shear reinforcement is not provided, shear failure may occur. *Shear failure* is characterized by small deflections and lack of ductility, giving little or no warning before failure. On the other hand, flexural failure is characterized by a gradual increase in deflection and cracking, thus giving warning before total failure. This is due to the ACI Code limitation on flexural reinforcement. The design for shear must ensure that shear failure does not occur before flexural failure.

8.2 SHEAR STRESSES IN CONCRETE BEAMS

The general formula for the shear stress in a homogeneous beam is

$$v = \frac{VQ}{Ib} \tag{8.1}$$

where

V = total shear at the section considered

Q = statical moment about the neutral axis of that portion of cross-section lying between a line through the point in question parallel to the neutral axis and nearest face, upper or lower, of the beam

I = moment of inertia of cross-section about the neutral axis

b = width of beam at the given point

The distribution of bending and shear stresses according to elastic theory for a homogeneous rectangular beam is as shown in Fig. 8.2. The bending stresses are calculated from the flexural

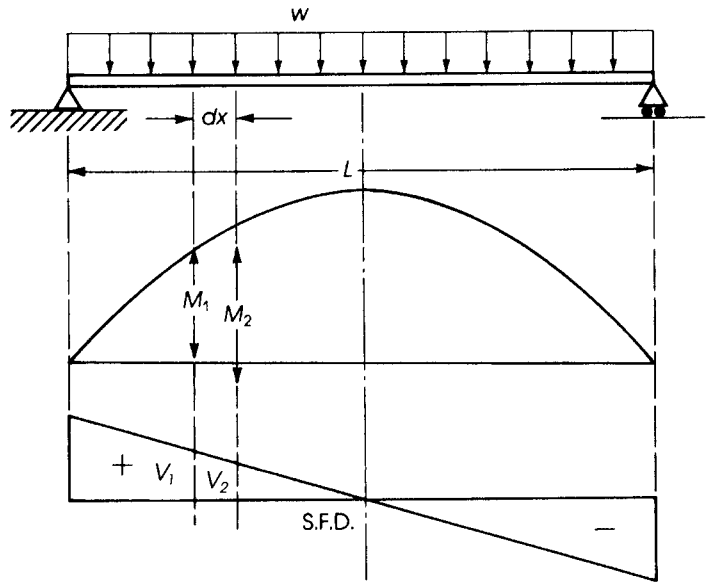


Figure 8.1 Bending moment and shearing force diagrams for a simple beam.

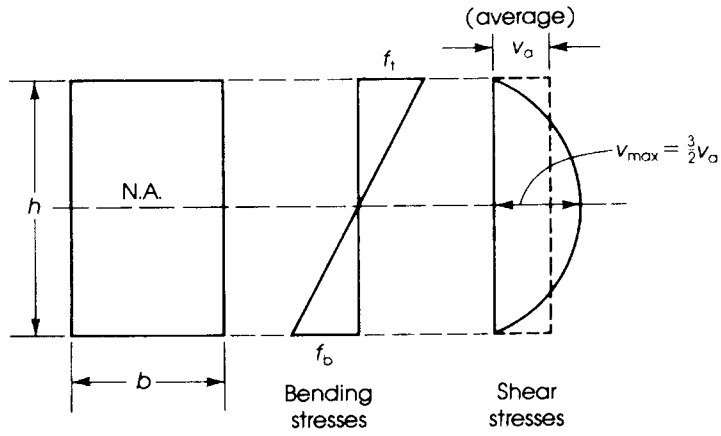


Figure 8.2 Bending and shear stresses in a homogeneous beam, according to elastic theory.

formula $f = Mc/I$, whereas the shear stress at any point is calculated by the shear formula of Eq. 8.1. The maximum shear stress is at the neutral axis and is equal to $1.5v_a$ (average shear), where $v_a = V/bh$. The shear stress curve is parabolic.

For a singly reinforced concrete beam, the distribution of shear stress above the neutral axis is a parabolic curve. Below the neutral axis, the maximum shear stress is maintained down to the level of the tension steel, because there is no change in the tensile force down to this point and the concrete in tension is neglected. The shear stress below the tension steel is 0 (Fig. 8.3). For doubly reinforced and T-sections, the distribution of shear stresses is as shown in Fig. 8.3.

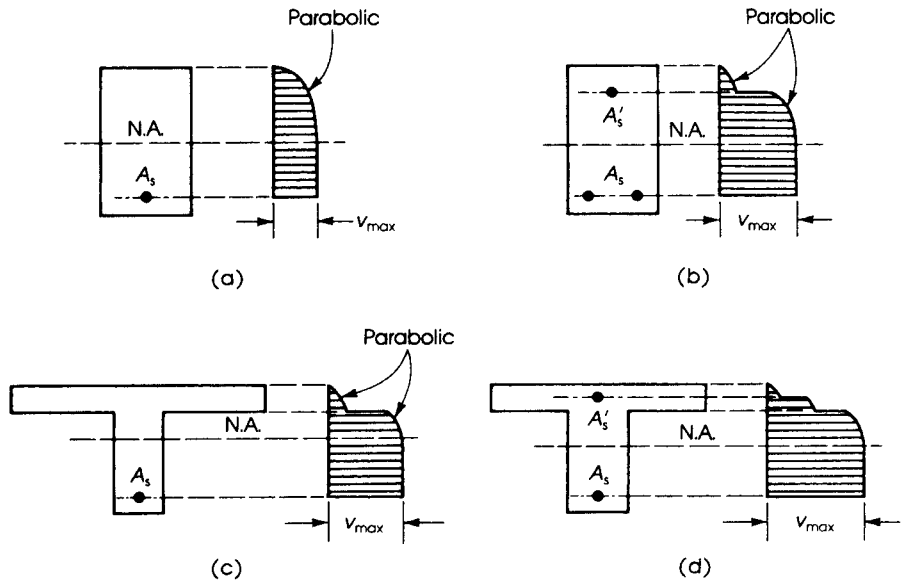


Figure 8.3 Distribution of shear stresses in reinforced concrete beams: (*a*) singly reinforced, (*b*) doubly reinforced, (*c*) T-section, (*d*) T-section with compression steel.

It can be observed that almost all the shear force is resisted by the web, whereas the flange resists a very small percentage; in most practical problems, the shear capacity of the flange is neglected.

Referring to Fig. 8.1 and taking any portion of the beam dx, the bending moments at both ends of the element, M_1 and M_2 are not equal. Because $M_2 > M_1$ and to maintain the equilibrium of the beam portion dx, the compression force C_2 must be greater than C_1 (Fig. 8.4). Consequently, a shear stress v develops along any horizontal section a-a_1 or b-b_1 (Fig. 8.4*a*). The normal and shear stresses on a small element at levels a-a_1 and b-b_1 are shown in Fig. 8.4*b*. Notice that the normal stress at the level of the neutral axis b-b_1 is 0, whereas the shear stress is maximum. The horizontal shear stress is equal to the vertical shear stress, as shown in Fig. 8.4*b*. When the normal stress f is 0 or low, a case of pure shear may occur. In this case, the maximum tensile stress f_t, acts at 45° (Fig. 8.4*c*).

The tensile stresses are equivalent to the principal stresses, as shown in Fig. 8.4*d*. Such principal stresses are traditionally called *diagonal tension stresses*. When the diagonal tension stresses reach the tensile strength of concrete, a diagonal crack develops. This brief analysis explains the concept of diagonal tension and diagonal cracking. The actual behavior is more complex, and it is affected by other factors, as explained later. For the combined action of shear and normal stresses at any point in a beam, the maximum and minimum diagonal tension (principal stresses) f_p are given by the equation

$$f_p = \frac{f}{2} \pm \sqrt{\left(\frac{f}{2}\right)^2 + v^2} \tag{8.2}$$

where

f = intensity of normal stress due to bending

v = shear stress

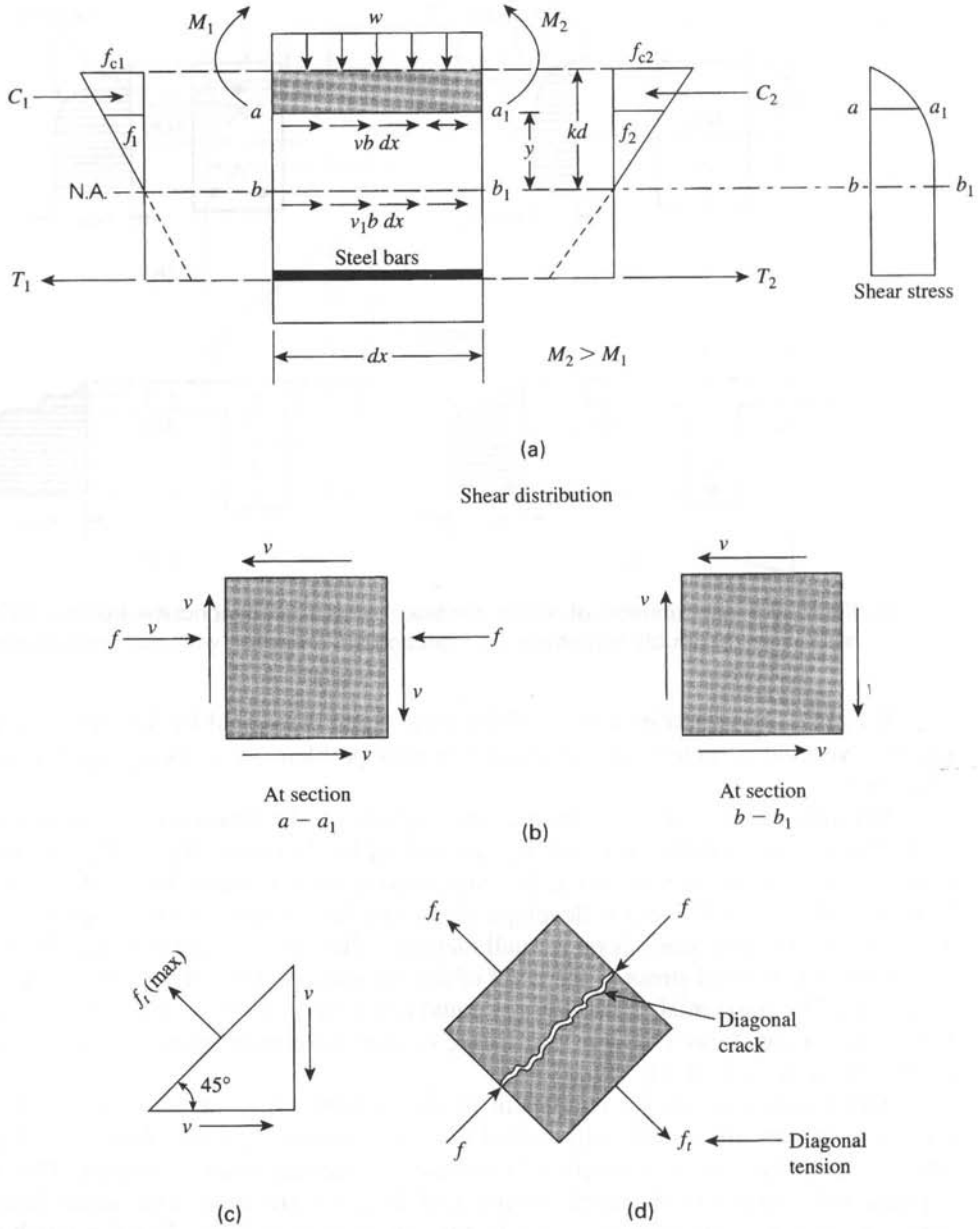


Figure 8.4 (*a*) Forces and stresses along the depth of the section, (*b*) normal and shear stresses, (*c*) pure shear, and (*d*) diagonal tension.

The shear failure in a concrete beam is most likely to occur where shear forces are maximum, generally near the supports of the member. The first evidence of impending failure is the formation of diagonal cracks.

8.3 BEHAVIOR OF BEAMS WITHOUT SHEAR REINFORCEMENT

Concrete is weak in tension, and the beam will collapse if proper reinforcement is not provided. The tensile stresses develop in beams due to axial tension, bending, shear, torsion, or a combination of these forces. The location of cracks in the concrete beam depends on the direction of principal stresses. For the combined action of normal stresses and shear stresses, maximum diagonal tension may occur at about a distance d from the face of the support.

The behavior of reinforced concrete beams with and without shear reinforcement tested under increasing load was discussed in Section 3.3. In the tested beams, vertical flexural cracks developed at the section of maximum bending moment when the tensile stresses in concrete exceeded the modulus of rupture of concrete, or $f_r = 7.5\lambda\sqrt{f'_c}$. Inclined cracks in the web developed at a later stage at a location very close to the support.

An inclined crack occurring in a beam that was previously uncracked is generally referred to as a *web-shear crack*. If the inclined crack starts at the top of an existing flexural crack and propagates into the beam, the crack is referred to as *flexural-shear crack* (Fig. 8.5). Web-shear cracks occur in beams with thin webs in regions with high shear and low moment. They are relatively uncommon cracks and may occur near the inflection points of continuous beams or adjacent to the supports of simple beams.

Flexural-shear cracks are the most common type found in reinforced concrete beams. A flexural crack extends vertically into the beam; then the inclined crack forms, starting from the top of the beam when shear stresses develop in that region. In regions of high shear stresses, beams must be reinforced by stirrups or by bent bars to produce ductile beams that do not rupture at a failure. To avoid a shear failure before a bending failure, a greater factor of safety must be provided against a shear failure. The ACI Code specifies a capacity reduction factor, ϕ, of 0.75 for shear.

Shear resistance in reinforced concrete members is developed by a combination of the following mechanisms [2] (Fig. 8.5):

- Shear resistance of the uncracked concrete, V_z [3]
- Interface shear transfer, V_a, due to aggregate interlock tangentially along the rough surfaces of the crack [3]
- Arch action [4]
- Dowel action, V_d, due to the resistance of the longitudinal bars to the transverse shearing force [5]

In addition to these forces, shear reinforcement increases the shear resistance V_s, by which depends on the diameter and spacings of stirrups used in the concrete member. If shear reinforcement is not provided in a rectangular beam, the proportions of the shear resisted by the various mechanisms are 20% to 40% by V_z 35% to 50% by V_a and 15% to 25% by V_d [6].

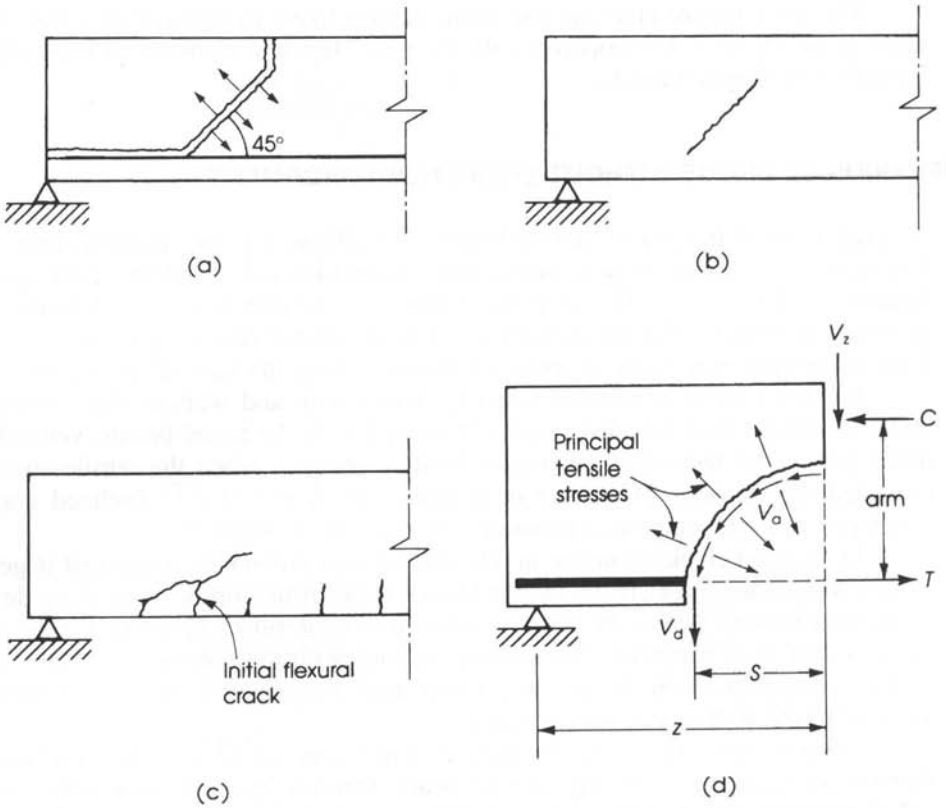


Figure 8.5 Shear failure: (*a*) general form, (*b*) web-shear crack, (*c*) flexural-shear crack, (*d*) analysis of forces involved in shear. V_a is interface shear, V_z is shear resistance, and V_d is dowel force.

8.4 MOMENT EFFECT ON SHEAR STRENGTH

In simply supported beams under uniformly distributed load, the midspan section is subjected to a large bending moment and zero or small shear, whereas sections near the ends are subjected to large shear and small bending moments (Fig. 8.1). The shear and moment values are both high near the intermediate supports of a continuous beam. At a location of large shear force and small bending moment, there will be little flexural cracking, and an average stress v is equal to V/bd. The diagonal tensile stresses are inclined at about 45° (Fig. 8.4*c*). Diagonal cracks can be expected when the diagonal tensile stress in the vicinity of the neutral axis reaches or exceeds the tensile strength of concrete. In general, the factored shear strength varies between $3.5\sqrt{f'_c}$ and $5\sqrt{f'_c}$. After completing a large number of beam tests on shear and diagonal tension [1], it was found that in regions with large shear and small moment, diagonal tension cracks were formed at an average shear force of

$$V_c = 3.5\sqrt{f'_c}b_w d \tag{8.3}$$

where b_w is the width of the web in a T-section or the width of a rectangular section and d is the effective depth of the beam.

In locations where shear forces and bending moments are high, flexural cracks are formed first. At a later stage, some cracks bend in a diagonal direction when the diagonal tension stress at the upper end of such cracks exceeds the tensile strength of concrete. Given the presence of large moments on a beam, for which adequate reinforcement is provided, the nominal shear force at which diagonal tension cracks develop is given by

$$V_c = 1.9\lambda\sqrt{f'_c}b_w d \tag{8.4}$$

This value is a little more than half the value in Eq. 8.3 when bending moment is very small. This means that large bending moments reduce the value of shear stress for which cracking occurs. The following equation has been suggested to predict the nominal shear stress at which a diagonal crack is expected [1]:

$$v_c = \frac{V}{b_w d} = \left(1.9\lambda\sqrt{f'_c} + 2500\rho\frac{Vd}{M}\right) \le 3.5\lambda\sqrt{f'_c} \tag{8.5}$$

1. ACI Code, Section 11.2.2.1, adopted this equation for the nominal shear force to be resisted by concrete for members subjected to shear and flexure only by:

$$V_c = \left[1.9\lambda\sqrt{f'_c} + 2500\rho_w\frac{V_u d}{M_u}\right]b_w d \le 3.5\lambda\sqrt{f'_c}b_w d \tag{8.6}$$

where $\rho_w = A_s/b_w$, d and b_w are the web width in a T-section or the width of a rectangular section, and V_u and M_u are the factored shearing force and bending moment occurring simultaneously on the considered section.

The value of $V_u d/M_u$ must not exceed 1.0 in Eq. 8.6. If M_u is large in Eq. 8.6, the second term becomes small and v_c approaches $1.9\lambda\sqrt{f'_c}$. If M_u is small, the second term becomes large and the upper limit of $3.5\lambda\sqrt{f'_c}$ controls. As an alternative to Eq. 8.6, the ACI Code, Section 11.2.1.1, permits evaluating the shear strength of concrete as follows:

$$V_c = 2\lambda\sqrt{f'_c}b_w d \tag{8.7}$$

$$V_c = 0.17\lambda\sqrt{f'_c}b_w d \qquad \text{(SI)}$$

2. For members subjected to significant axial compression force N_u,

$$V_c = \left(1.9\lambda\sqrt{f'_c} + 2500\rho_w\frac{V_u d}{M_m}\right)b_w d \tag{8.8}$$

$$M_m = M_u - N_u\left(\frac{4h-d}{8}\right)$$

where

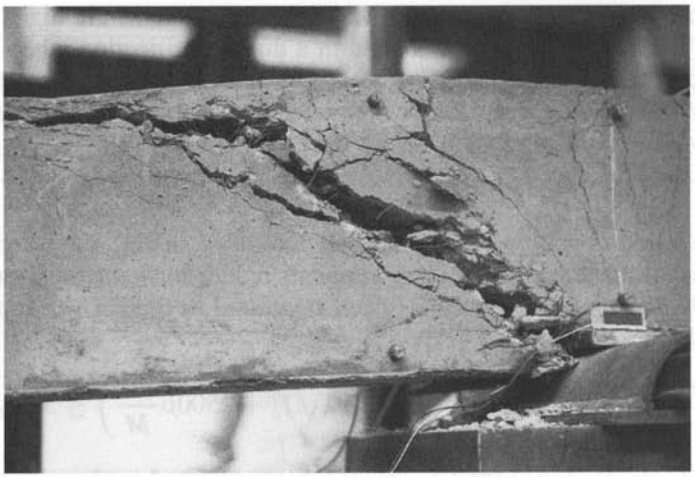
$$\rho_w = \frac{A_s}{b_w d}$$

h = overall depth

$V_u d/M_u$ may be greater than 1.0, but V_c must not exceed

$$V_c = 3.5\lambda\sqrt{f'_c}b_w d\sqrt{1 + \frac{N_u}{500A_g}} \tag{8.9}$$

where A_g is the gross area in.^{2}



Shear failure near a middle support.

Alternatively, V_c may be computed by

$$V_c = b_w d \left(2 + 0.001 \frac{N_u}{A_g}\right) \lambda \sqrt{f'_c} \tag{8.10}$$

3. In the case of members subjected to significant axial tensile force N_u,

$$V_c = b_w d \left(2 + 0.004 \frac{N_u}{A_g}\right) \lambda \sqrt{f'_c} \tag{8.11}$$

where N_u is to be taken as negative for tension and N_u/A_g is in psi. If V_c is negative, V_c should be taken equal to zero.

8.5 BEAMS WITH SHEAR REINFORCEMENT

Different types of shear reinforcement may be used:

1. Stirrups, which can be placed either perpendicular to the longitudinal reinforcement or inclined, usually making a 45° angle and welded to the main longitudinal reinforcement. Vertical stirrups, using no. 3 or no. 4 U-shaped bars, are the most commonly used shear reinforcement in beams (Fig. 8.6*a*).
2. Bent bars, which are part of the longitudinal reinforcement, bent up (where they are no longer needed) at an angle of 30° to 60°, usually at 45°.
3. Combinations of stirrups and bent bars.
4. Welded wire fabric with wires perpendicular to the axis of the member.
5. Spirals, circular ties, or hoops in circular sections, as columns.

The shear strength of a reinforced concrete beam is increased by the use of shear reinforcement. Prior to the formation of diagonal tension cracks, shear reinforcement contributes very little to the shear resistance. After diagonal cracks have developed, shear reinforcement augments the shear resistance of a beam, and a redistribution of internal forces occurs at the cracked section.

When the amount of shear reinforcement provided is small, failure due to yielding of web steel may be expected, but if the amount of shear reinforcement is too high, a shear-compression failure may be expected, which should be avoided.

Concrete, stirrups, and bent bars act together to resist transverse shear. The concrete, by virtue of its high compressive strength, acts as the diagonal compression member of a lattice girder system, where the stirrups act as vertical tension members. The diagonal compression force is such that its vertical component is equal to the tension force in the stirrup. Bent-up reinforcement acts also as tension members in a truss, as shown in Fig. 8.6.

In general, the contribution of shear reinforcement to the shear strength of a reinforced concrete beam can be described as follows [2]:

1. It resists part of the shear, V_s.
2. It increases the magnitude of the interface shear, V_a (Fig. 8.5), by resisting the growth of the inclined crack.

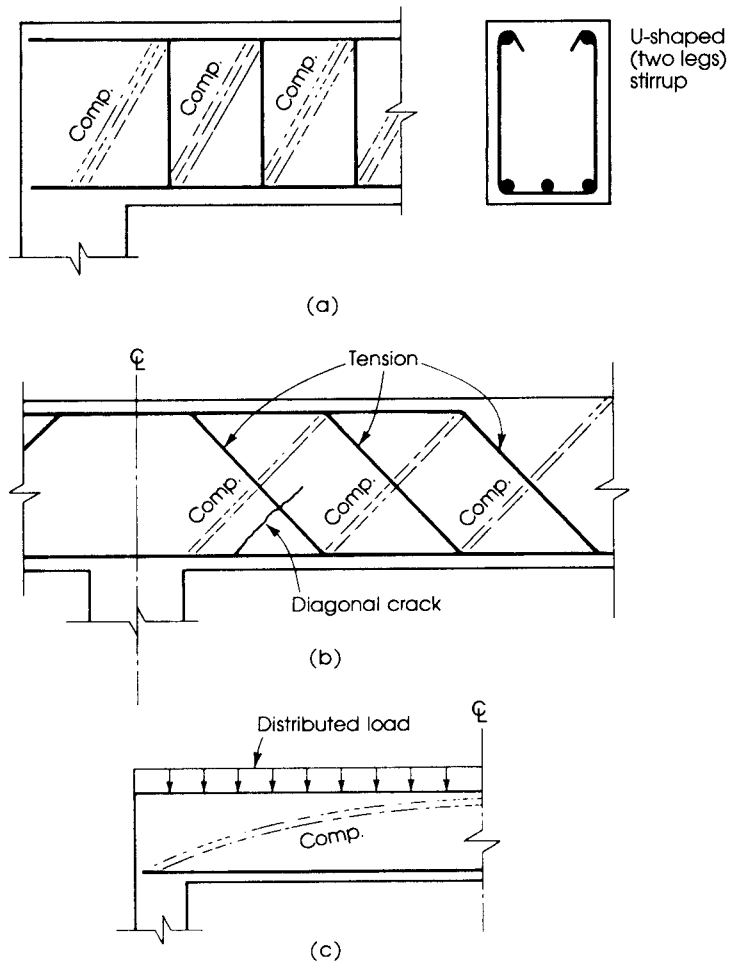


Figure 8.6 Truss action of web reinforcement with (*a*) stirrups, (*b*) bent bars, and (*c*) tension steel.

3. It increases the dowel force, V_d (Fig. 8.5), in the longitudinal bars.
4. The confining action of the stirrups on the compression concrete may increase its strength.
5. The confining action of stirrups on the concrete increases the rotation capacity of plastic hinges that develop in indeterminate structures at ultimate load and increases the length over which yielding takes place [7].

The total nominal shear strength of beams with shear reinforcement V_n is due partly to the shear strength attributed to the concrete V_c and partly to the shear strength contributed by the shear reinforcement V_s:

$$V_n = V_c + V_s \tag{8.12}$$

The shear force V_u produced by factored loads must be less than or equal to the total nominal shear strength V_n or

$$V_u \leq \phi V_n = \phi(V_c + V_s) \tag{8.13}$$

where $V_u = 1.2V_D + 1.6V_L$ and $\phi = 0.75$.

An expression for V_s may be developed from the truss analogy (Fig. 8.7). For a 45° crack and a series of inclined stirrups or bent bars, the vertical shear force V_s resisted by shear reinforcement is equal to the sum of the vertical components of the tensile forces developed in the inclined bars. Therefore,

$$V_s = nA_v f_{yt} \sin\alpha \tag{8.14}$$

where A_v is the area of shear reinforcement with a spacing s, and f_{yt} is the yield strength of shear reinforcement, ns is defined as the distance aa_1a_2:

$$d = a_1a_4 = aa_1 \tan 45^\circ \qquad \text{(from triangle } aa_1a_4\text{)}$$

$$d = a_1a_4 = a_1a_2 \tan\alpha \qquad \text{(from triangle } a_1a_2a_4\text{)}$$

$$ns = aa_1 + a_1a_2$$

$$= d(\cot 45^\circ + \cot\alpha) = d(1 + \cot\alpha)$$

$$n = \frac{d}{S}(1 + \cot\alpha)$$

Substituting this value in Eq. 8.14 gives

$$V_s = \frac{A_v f_{yt} d}{S} \sin\alpha(1 + \cot\alpha) = \frac{A_v f_{yt} d}{S}(\sin\alpha + \cos\alpha) \tag{8.15}$$

For the case of vertical stirrups, $\alpha = 90^\circ$ and

$$V_s = \frac{A_v f_{yt} d}{S} \quad \text{or} \quad s = \frac{A_v f_{yt} d}{V_s} \tag{8.16}$$

In the case of T-sections, b is replaced by the width of web b_w in all shear equations. When $\alpha = 45^\circ$, Eq. 8.15 becomes

$$V_s = 1.4\left(\frac{A_v f_{yt} d}{S}\right) \text{ or } s = \frac{1.4\ A_v f_{yt} d}{V_s} \tag{8.17}$$

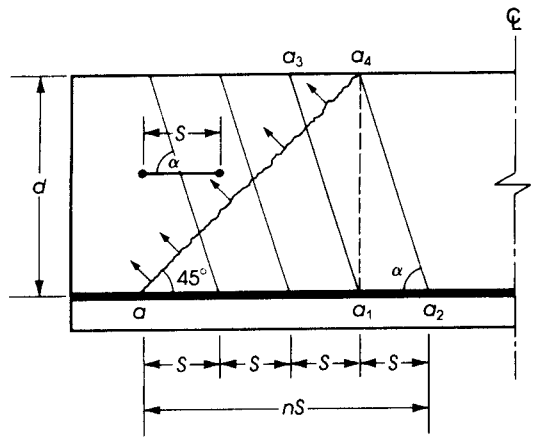


Figure 8.7 Factors in inclined shear reinforcement.

For a single bent bar or group of parallel bars in one position, the shearing force resisted by steel is

$$V_s = A_v f_{yt} \sin\alpha \text{ or } A_v = \frac{V_s}{f_{yt} \sin\alpha} \tag{8.18}$$

For $\alpha = 45°$,

$$A_v = 1.4\left(\frac{V_s}{f_{yt}}\right) \tag{8.19}$$

For circular sections, mainly in columns, V_s shall be computed from Eq. 8.16 using $d = 0.8 \times$ diameter and A_v = two times the area of the bar in a circular tie, hoop, or spiral.

8.6 ACI CODE SHEAR DESIGN REQUIREMENTS

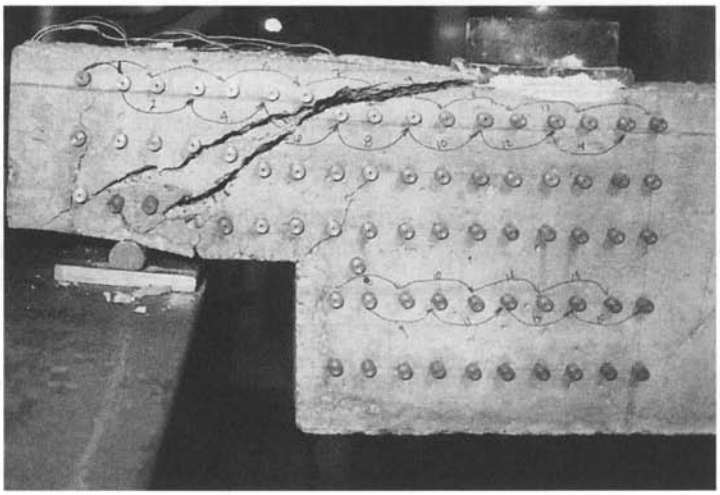
8.6.1 Critical Section for Nominal Shear Strength Calculation

The ACI Code, Section 11.1.3, permits taking the critical section for nominal shear strength calculation at a distance d from the face of the support. This recommendation is based on the fact that the first inclined crack is likely to form within the shear span of the beam at some distance d away from the support. The distance d is also based on experimental work and appeared in the testing of the beams discussed in Chapter 3. This critical section is permitted on the condition that the support reaction introduces compression into the end region, loads are applied at or near the top of the member, and no concentrated load occurs between the face of the support and the location of the critical section.

The Code also specifies that shear reinforcement must be provided between the face of the support and the distance d, using the same reinforcement adopted for the critical section.

8.6.2 Minimum Area of Shear Reinforcement

The presence of shear reinforcement in a concrete beam restrains the growth of inclined cracking. Moreover, ductility is increased, and a warning of failure is provided. If shear reinforcement is



Shear failure in dapped-end beam.

not provided, brittle failure will occur without warning. Accordingly, a minimum area of shear reinforcement is specified by the Code. The ACI Code, Section 11.4.6 requires all stirrups to have a minimum shear reinforcement area, A_v, equal to

$$A_v = 0.75\sqrt{f'_c}\left(\frac{b_w s}{f_{yt}}\right) \geq \frac{50 b_w s}{f_{yt}} \tag{8.20}$$

where b_w is the width of the web and s is the spacing of the stirrups. The minimum amount of shear reinforcement is required whenever V_u exceeds $\phi V_c/2$, except in

- Slabs and footings
- Concrete floor joist construction
- Beams where the total depth does not exceed 10 in., 2.5 times the flange thickness for T-shaped flanged sections, or one-half the web width, whichever is greatest.

If $0.75\sqrt{f'_c} = 50$ then $f'_c = 4444$ psi. This means that when $f'_c < 4500$ psi, the minimum $A_v = 50 b_w s/f_{yt}$ controls and when $f'_c \geq 4500$ psi, then the minimum $A_v = 0.75\sqrt{f'_c}(b_w s/f_{yt})$ controls. This increase in the minimum area of shear reinforcement for $f'_c \geq 4500$ psi is to prevent sudden shear failure when inclined cracking occurs.

It is a common practice to increase the depth of a slab, footing, or shallow beam to increase its shear capacity. Stirrups may not be effective in shallow members, because their compression zones have relatively small depths and may not satisfy the anchorage requirements of stirrups. For beams that are not shallow, reinforcement is not required when V_u is less than $\phi V_c/2$.

The minimum shear reinforcement area can be achieved by using no. 3 stirrups placed at maximum spacing, S_{max}. If $f_y = 60$ ksi and U-shaped (two legs) no. 3 stirrups are used, then Eq. 8.20 becomes

$$S_{max} = \frac{A_v f_{yt}}{(0.75\sqrt{f'_c})b_w} \leq \frac{A_v f_{yt}}{50 b_w} \tag{8.21}$$

$$\begin{aligned}
&\text{For } f'_c < 4500 \text{ psi}, \quad S_{\max}(\text{in.}) = 0.22(60{,}000)/50b_w = 264/b_w.\\
&\text{For } f'_c = 4500 \text{ psi}, \quad S_{\max}(\text{in.}) = 262/b_w.\\
&\text{For } f'_c = 5000 \text{ psi}, \quad S_{\max}(\text{in.}) = 249/b_w.\\
&\text{For } f'_c = 6000 \text{ psi}, \quad S_{\max}(\text{in.}) = 227/b_w.
\end{aligned} \tag{8.22}$$

If U-shaped no. 4 stirrups are used, then for $f'_c < 4500$ psi,

$$S_{\max}(\text{in.}) = \frac{0.4(60{,}000)}{50b_w} = \frac{480}{b_w}$$

$$\begin{aligned}
&\text{For } f'_c = 4500 \text{ psi}, \quad S_{\max}(\text{in.}) = 476/b_w.\\
&\text{For } f'_c = 5000 \text{ psi}, \quad S_{\max}(\text{in.}) = 453/b_w.\\
&\text{For } f'_c = 6000 \text{ psi}, \quad S_{\max}(\text{in.}) = 413/b_w.
\end{aligned} \tag{8.23}$$

Note that $S_{\max}$ shall not exceed 24 in., nor $d/2$.

Table 8.1 gives $S_{\max}$ based on Eqs. 8.22 and 8.23. Final spacings should be rounded to the lower inch. For example, $S = 20.3$ in. becomes 20 in.

8.6.3 Maximum Shear Carried by Web Reinforcement V_s

To prevent a shear-compression failure, where the concrete may crush due to high shear and compressive stresses in the critical region on top of a diagonal crack, the ACI Code, Section 11.4.7.9, requires that V_s shall not exceed $(8\sqrt{f'_c})b_w d$. If V_s exceeds this value, the section should be increased. Based on this limitation,

$$\text{If } f'_c = 3 \text{ ksi, then } V_s \le 0.438 b_w d \text{ (kips)} \quad \text{or} \quad V_s/b_w d \le 438 \text{ psi.}$$

$$\text{If } f'_c = 4 \text{ ksi, then } V_s \le 0.506 b_w d \text{ (kips)} \quad \text{or} \quad V_s/b_w d \le 506 \text{ psi.}$$

$$\text{If } f'_c = 5 \text{ ksi, then } V_s \le 0.565 b_w d \text{ (kips)} \quad \text{or} \quad V_s/b_w d \le 565 \text{ psi.}$$

8.6.4 Maximum Spacing of Stirrups

To ensure that a diagonal crack will always be intersected by at least one stirrup, the ACI Code, Section 11.4.5, requires that the spacings between stirrups shall not exceed $d/2$, nor 24 in., provided that $V_s \le (4\sqrt{f'_c})b_w d$. This is based on the assumption that a diagonal crack develops at 45° and extends a horizontal distance of about d. In regions of high shear, where V_s exceeds $(4\sqrt{f'_c})b_w d$, the maximum spacing between stirrups must not exceed $d/4$. This limitation is necessary to ensure that the diagonal crack will be intersected by at least three stirrups. When V_s exceeds the maximum value of $8\sqrt{f'_c}b_w d$, this limitation of maximum stirrup spacing does not apply, and the dimensions of the concrete cross-section should be increased.

Table 8.1 Values of $S_{\max} = (A_v f_y/50b_w) = 24$ in. when $f_{yt} = 60$ ksi and $f'_c < 4500$ psi

b_w (in.)	10	11	12	13	14	15	16	18	20	22	24	b_w
$S_{\max}$ (in.) no. 3 stirrups	24	24	22	20.3	18.9	17.6	16.5	14.7	13.2	12	11	$\frac{264}{b_w}$
S_{max} (in.) no. 4 stirrups	24	24	24	24	24	24	24	24	24	21.8	20	$\frac{480}{b_w}$

A second limitation for the maximum spacing of stirrups may also be obtained from the condition of minimum area of shear reinforcement. A minimum A_v is obtained when the spacing s is maximum (Eq. 8.21).

A third limitation for maximum spacing is 24 in. $V_s \leq (4\sqrt{f'_c})b_w d$ and 12 in. when V_s is greater than $(4\sqrt{f'_c})b_w d$ but less than or equal to $(8\sqrt{f'_c})b_w d$. The least value of all maximum spacings must be adopted. The ACI Code maximum spacing requirement ensures closely spaced stirrups that hold the longitudinal tension steel in place within the beam, thereby increasing their dowel capacity, V_d (Fig. 8.5).

8.6.5 Yield Strength of Shear Reinforcement

The ACI Code, Section 11.4.2, requires that the design yield strength of shear reinforcement shall not exceed 60 ksi (420 MPa). The reason behind this decision is to limit the crack width caused by the diagonal tension and to ensure that the sides of the crack remain in close contact to improve the interface shear transfer, V_a (Fig. 8.5). For welded deformed wire fabric, the design yield strength shall not exceed 80 ksi (560 MPa).

8.6.6 Anchorage of Stirrups

The ACI Code, Section 12.13.1, requires that shear reinforcement be carried as close as possible to the compression and tension extreme fibers, within the Code requirements for concrete cover, because near ultimate load the flexural tension cracks penetrate deep into the beam. Also, for stirrups to achieve their full yield strength, they must be well anchored. Near ultimate load, the stress in a stirrup reaches its yield stress at the point where a diagonal crack intercepts that stirrup. The ACI Code requirements for stirrup anchorage, Section 12.13, are as follows:

1. Each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar (ACI Code, Section 12.13.3). See Fig. 8.8*a*.
2. The code allows the use of a standard hook of 90°, 135°, or 180° around longitudinal bars for no. 5 bars or D31 wire stirrups. If no. 6, 7, or 8 stirrups with $f_{yt} > 40$ ksi are used, the Code (Section 12.13.2) requires a standard hook plus an embedment length of $0.014 d_b f_{yt}/(\lambda\sqrt{f'_c})$ between midheight of the member and the outside of the hook. If the bars are bent at 90°, extensions shall not be less than $12d_b$. For no. 5 bars or smaller stirrups, the extension is $6d_b$ (ACI Code, Section 7.1.3). See Fig. 8.8*b*.
3. If spliced double U-stirrups are used to form closed stirrups, the lap length shall not be less than $1.3l_d$ (ACI Code, Section 12.13.5). See Fig. 8.8*c*.
4. Welded wire fabric is used for shear reinforcement in the precast industry. Anchorage details are given in the ACI Code, Section 12.13.2.3, and in its commentary.
5. Closed stirrups are required for beams subjected to torsion or stress reversals (ACI Code, Section 7.11).
6. Beams at the perimeter of the structure should contain closed stirrups to maintain the structural integrity of the member (ACI Code, Section 7.13.2.2).

8.6.7 Stirrups Adjacent to the Support

The ACI Code, Section 11.1.3, specifies that shear reinforcement provided between the face of the support and the critical section at a distance d from it may be designed for the same shear V_u at the critical section. It is common practice to place the first stirrup at a distance $S/2$ from the face of the support, where s is the spacing calculated by Eq. 8.16 for V_u at the critical section.

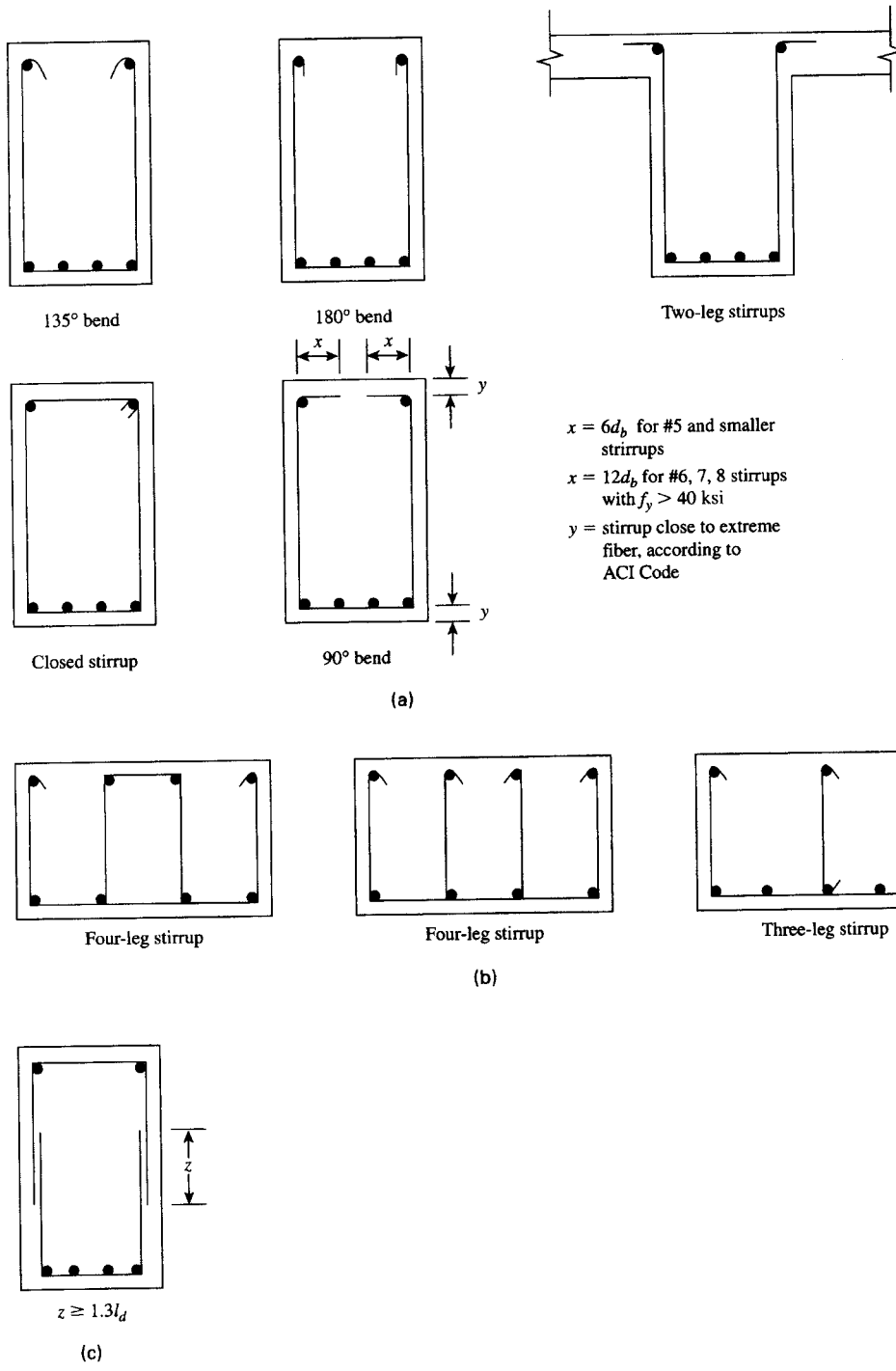


Figure 8.8 Stirrup types: (*a*) U-stirrups enclosing longitudinal bars, anchorage lengths, and closed stirrups, (*b*) multileg stirrups, and (*c*) spliced stirrups.

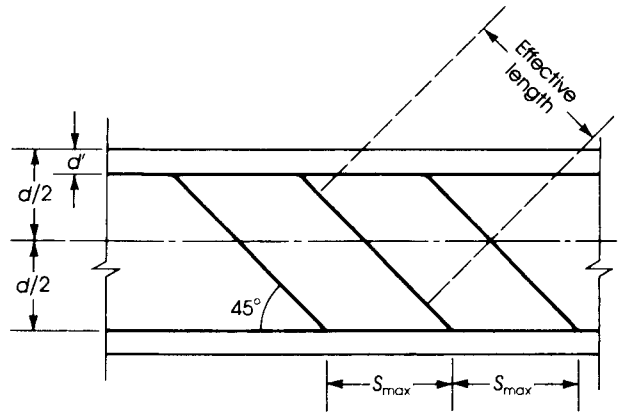


Figure 8.9 Effective length and spacing of bent bars.

8.6.8 Effective Length of Bent Bars

Only the center three-fourths of the inclined portion of any longitudinal bar shall be considered effective for shear reinforcement. This means that the maximum spacing of bent bars is $0.75(d - d')$. From Fig. 8.9, the effective length of the bent bar is $0.75(d - d')/(\sin 45°) = 0.75(1.414)(d - d') = 1.06(d - d')$. The maximum spacing S is equal to the horizontal projection of the effective length of the bent bar. Thus $S_{max} = 1.06(d - d')\cos 45°$, or $S_{max} = 0.707[1.06(d - d')] = 0.75(d - d')$.

8.7 DESIGN OF VERTICAL STIRRUPS

Stirrups are needed when $V_u > \frac{1}{2}\phi V_c$. Minimum stirrups are used when V_u is greater than $\frac{1}{2}\phi V_c$ but less than ϕV_c. This is achieved by using no. 3 stirrups placed at maximum spacing. When V_u is greater than ϕV_c stirrups must be provided. The spacing of stirrups may be less than the maximum spacing and can be calculated using Eq. 8.16: $S = A_v f_{yt} d / V_s$.

The stirrups that are commonly used in concrete sections are made of two-leg no. 3 or no. 4 U-stirrups with $f_{yt} = 60$ ksi. If no. 3 stirrups are used, then Eq. 8.16 becomes

$$\frac{S}{d} = \frac{A_v f_y}{V_s} = \frac{0.22(60)}{V_s} = \frac{13.2}{V_s} \tag{8.24}$$

If no. 4 stirrups are used, then

$$\frac{S}{d} = \frac{A_v f_y}{V_s} = \frac{0.4(60)}{V_s} = \frac{24}{V_s} \tag{8.25}$$

The ratio of stirrup spacings relative to the effective depth of the beam, d, depends on V_s. The values of S/d for different values of V_s when $f_y = 60$ ksi are given in Tables 8.2 and 8.3 for no. 3 and no. 4 U-stirrups, respectively. The same values are plotted in Figs. 8.10 and 8.11. The following observations can be made:

1. If no. 3 stirrups are used, $S = d/2$ when $V_s \leq 26.4$ K. When V_s increases, S/d decreases in a nonlinear curve to reach 0.132 at $V_s = 100$ K. If the minimum spacing is limited to

Table 8.2 S/d Ratio for Different Values of V_s ($f_{yt} = 60$ ksi, $S/d = 13.2/V_s$), No. 3 Stirrups

V_s (K)	26.4	30	40	50	52.8	60	70	80	90	100	125
S/d	0.5	0.44	0.33	0.264	0.25	0.22	0.19	0.165	0.15	0.132	0.106

Table 8.3 S/d Ratio for Different Values of V_s($f_{yt} = 60$ ksi, $S/d = 24/V_s$), No. 4 Stirrups

V_s(K)	48	50	60	70	80	90	96	100	110	120	150	175
S/d	0.50	0.48	0.40	0.34	0.3	0.27	0.25	0.24	0.22	0.20	0.16	0.137

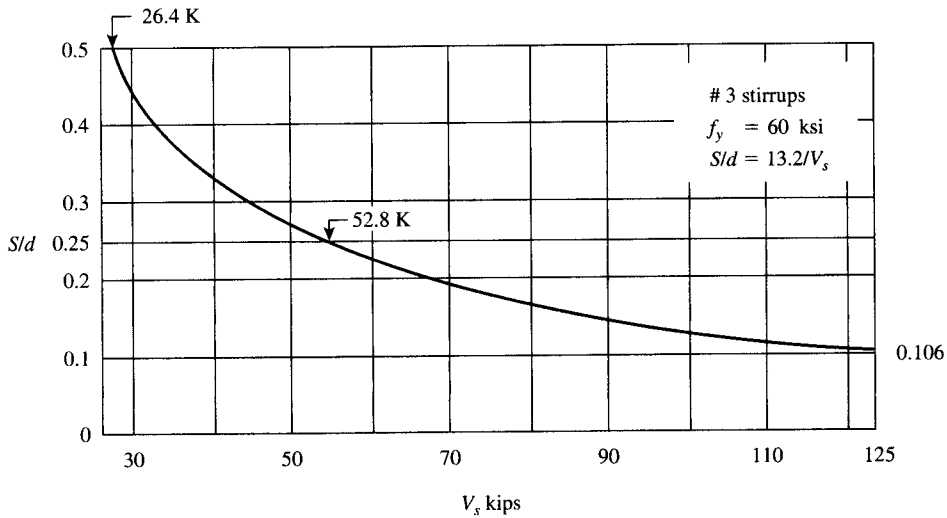


Figure 8.10 V_s versus S/d for no. 3 stirrups and $f_{yt} = 60$ ksi.

3 in., then d must be equal to or greater than 22.7 in. to maintain that 3-in. spacing. When V_s is equal to or greater than 52.8 K, then $S \leq d/4$.

2. If no. 4 U-stirrups are used, $S = d/2$ when $V_s \leq 48$ K. When V_s increases, S/d decreases to reach 0.16 at $V_s = 150$ K. If the minimum spacing is limited to 3 in., then $d \geq 18.75$ in. to maintain the 3-in. spacing. When V_s is equal to or greater than 96 K, then $S \leq d/4$.
3. If grade 40 U-stirrups are used ($f_{yt} = 40$ ksi), multiply the S/d values by $\frac{2}{3}$ or, in general, $f_{yt}/60$.

8.8 DESIGN SUMMARY

The design procedure for shear using vertical stirrups according to the ACI Code can be summarized as follows:

1. Calculate the factored shearing force, V_u, from the applied forces acting on the structural member. The critical design shear value is at a section located at a distance d from the face of the support.

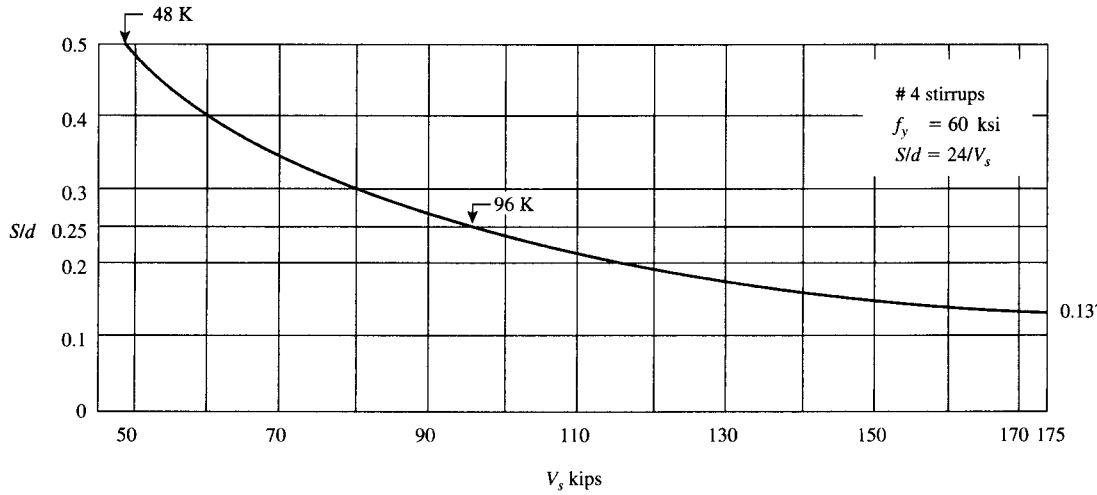


Figure 8.11 V_s versus S/d for no. 4 stirrups and f_{yt} = 60 ksi.

2. Calculate $\phi V_c = \phi 2\lambda\sqrt{f'_c}b_w d$, or

$$\phi V_c = \phi\left[1.9\lambda\sqrt{f'_c} + 2500\rho_w\frac{V_u d}{M_u}\right]b_w d \le \phi 3.5\lambda\sqrt{f'_c}b_w d$$

Then calculate $\frac{1}{2}\phi V_c$

3. **a.** If $V_u < \frac{1}{2}\phi V_c$, no shear reinforcement is needed.

   b. If $\frac{1}{2}\phi V_c < V_u \le \phi V_c$, minimum shear reinforcement is required. Use no. 3 U-stirrups spaced at maximum spacings, as explained in step 7.

   c. If $V_u > \phi V_c$, shear reinforcement must be provided according to steps 4 through 8.

4. If $V_u > \phi V_c$, calculate the shear to be carried by shear reinforcement:

$$V_u = \phi V_c + \phi V_s \text{ or } V_s = \frac{V_u - \phi V_c}{\phi}$$

5. Calculate $V_{c_1} = (4\sqrt{f'_c})b_w d$ and $V_{c_2} = (8\sqrt{f'_c})b_w d = 2V_{c_1}$. Compare the calculated V_s with the maximum permissible value of $V_{c_2} = (8\sqrt{f'_c})b_w d$. If V_s is less than V_{c_2}, proceed in the design; if not, increase the dimensions of the section.

6. Calculate the stirrup spacings based on the calculated $S_1 = A_v f_{yt} d/V_s$ or use Figs. 8.10 and 8.11 or Tables 8.2 and 8.3.

7. Determine the maximum spacing allowed by the ACI Code. The maximum spacing is the least of S_2 and S_3:

   a. $S_2 = d/2 \le 24$ in. if $V_s <= V_{c_1} = (4\sqrt{f'_c})b_w d$.
   $S_2 = d/4 \le 12$ in. if $V_{c_1} < V_s \le V_{c_2}$.

   b. $S_3 = A_v f_{yt}/50b_w \ge A_v f_{yt}/(0.75\sqrt{f'_c}b_w)$
   $S_{\max}$ is the smaller of S_2 and S_3. Values of S_3 are shown in Table 8.1.

8. If S_1 calculated in step 6 is less than $S_{\max}$ (the smaller of S_2 and S_3), then use S_1 to the nearest smaller $\frac{1}{2}$ in. If $S_1 > S_{\max}$, then use $S_{\max}$ as the adopted S.

9. The ACI Code did not specify a minimum spacing. Under normal conditions, a practical minimum S may be assumed to be equal to 3 in. for $d \leq 20$ in. and 4 in. for deeper beams. If S is considered small, either increase the stirrup bar number or use multiple-leg stirrups (Fig. 8.8).
10. For circular sections, the area used to compute V_c = diameter times the effective depth d, where $d = 0.8$ the diameter, ACI Code, Section 11.2.3.

Example 8.1

A simply supported beam has a rectangular section $b = 12$ in., $d = 21.5$ in., and $h = 24$ in. and is reinforced with four no. 8 bars. Check if the section is adequate for each of the following factored shear forces. If it is not adequate, design the necessary shear reinforcement in the form of U-stirrups. Use $f'_c = 4$ ksi and $f_{yt} = 60$ ksi. Assume normal-weight concrete.

a. $V_u = 12$ K **(b)** $V_u = 24$ K **(c)** $V_u = 54$ K **(d)** $V_u = 77$ K **(e)** $V_u = 128$ K

Solution

In general, $b_w = b = 12$ in., $d = 21.5$ in., and

$$\phi V_c = \phi(2\lambda\sqrt{f'_c})bd = 0.75(2)(1)(\sqrt{4000})(12)(21.5) = 24.5 \text{ K}$$

$$\tfrac{1}{2}\phi V_c = 12.25 \text{ K}$$

$$V_{c_1} = (4\sqrt{f'_c})bd = (4\sqrt{4000})(12)(21.5)/1000 = 65.3 \text{ K}$$

$$V_{c_2} = (8\sqrt{f'_c})bd = 130.6 \text{ K}$$

a. $V_u = 12 \text{ K} < \frac{1}{2}\phi V_c = 12.25$ K, section is adequate, and shear reinforcement is not required.

b. $V_u = 24 \text{ K} > \frac{1}{2}\phi V_c$, but it is less than $\phi V_c = 24.5$ K. Therefore, $V_s = 0$ and minimum shear reinforcement is required. Choose no. 3 U-stirrup (two legs) at maximum spacing. $A_v = 2(0.11) = 0.22$ in^2. Maximum spacing is the least of

$S_2 = d/2 = 21.5/2 = 10.75$ in., say, 10.5 in. (controls).

$S_3 = A_v f_{yt}/50b_w = 0.22(60{,}000)/50(12) = 22$ in. (or use Table 8.1)

$S_4 = 24$ in. Use no. 3 U-stirrups spaced at 10.5 in.

c. $V_u = 54 \text{ K} > \phi V_c$. Shear reinforcement is needed. Calculation may be organized in steps:
Calculate $V_s = (V_u - \phi V_c)/\phi = (54 - 24.5)/0.75 = 39.3$ K.
Check if $V_s \leq V_{c_1} = (4\sqrt{f'_c})b_w d = 65.3$ K. Because $V_s < 65.3$ K, then $S_{\max} = d/2$, and the $d/4$ condition does not apply.
Choose no. 3 U-stirrups and calculate the required spacings based on V_s.

$$S_1 = \frac{A_v f_{yt} d}{V_s} = \frac{0.22(60)(21.5)}{39.3} = 7.26 \text{ in.} \qquad \text{say, 7 in.}$$

Calculate maximum spacings: $S_2 = 10.5$ in., $S_3 = 22$ in., and $S_4 = 24$ in. and maximum $S = 10.5$ in. (calculated in (b)).
Because $S = 7$ in. $< S_{\max} = 10.5$ in., then use no. 3 U-stirrups spaced at 7 in.

d. $V_u = 77 \text{ K} > \phi V_c$, so stirrups must be provided.
Calculate $V_s = (V_u - \phi V_c)/\phi = (77 - 24.5)/0.75 = 70$ K.
Check if $V_s \leq V_{c_1} = 4\sqrt{f'_c}b_w d = 65.3$ K. Because $V_s > 65.3$ K, then $S_{\max} = d/4 = 12$ in. must be used.
Check if $V_s \leq V_{c_2} = 8\sqrt{f'_c}b_w d = 130.6$ K. Because $V_{c_1} < V_s < V_{c_2}$, then stirrups can be used without increasing the section.

Choose no. 3 U-stirrups and calculate S_1 based on V_s:

$$S_1 = \frac{A_v f_{yt} d}{V_s} = \frac{0.22(60)(21.5)}{70} = 4.1 \text{ in.}, \qquad \text{say, 4 in.}$$

Calculate maximum spacings: $S_2 = d/4 = 21.5/4 = 5.3$ in., say, 5.0 in.; $S_3 = 22$ in.; and $S_4 = 12$ in. Hence $S_{max} = 5$-in. controls.
Because $S = 4$ in. $< S_{max} = 5$ in., then use no. 3 stirrups spaced at 4 in.

e. $V_u = 128$ K $> \phi V_c$, so shear reinforcement is required.
Calculate $V_s = (V_u - \phi V_c)/\phi = (128 - 24.5)/0.75 = 138$ K.
Because $V_s > V_{c_2} = 130.2$ K, the section is not adequate. Increase one or both dimensions of the beam section.

Notes : Table 8.2 and Fig. 8.10 can be used to calculate the spacing S for (c) and (d).

1. For (c), $V_s = 39.3$ K, from Fig. 8.10 (or Table 8.2 for no. 3 U-stirrups), $S/d = 0.34$ and $S_1 = 7.3$ in., which is less than $d/2 = 10.5$ in. Note that S_{max} based on V_s is $d/2$ and not $d/4$. Also, from Table 8.1, $S_3 = A_v f_{yt}/50 b_w = 22$ in.
2. For (d), $V_s = 70$ K, $S/d = 0.19$ and $S_1 = 4.1$ in. $V_s = 70$ is greater than 52.8 K, and $S_{max} = d/4$ is required.

Example 8.2

A 17-ft-span simply supported beam has a clear span of 16 ft and carries uniformly distributed dead and live loads of 4.5 K/ft and 3.75 K/ft, respectively. The dimensions of the beam section and steel reinforcement are shown in Fig. 8.12. Check the section for shear and design the necessary shear reinforcement. Given $f'_c = 3$ ksi normal-weight concrete and $f_{yt} = 60$ ksi.

Solution

Given b_w (web) $= 14$ in., $d = 22.5$ in.

1. Calculate factored shear from external loading:

   factored uniform load $= 1.2(4.5) + 1.6(3.75) = 11.4$K/ft

   $$V_u(\text{at face of support}) = \frac{11.4(16)}{2} = 91.2\text{K}$$

   Design V_u (at d distance from the face of support) $= 91.2 - 22.5(11.4)/12 = 69.83$ K.
2. Calculate ϕV_c:

   $$\phi V_c = \phi(2\lambda\sqrt{f'_c}) b_w d = \frac{0.75(2)(1)(\sqrt{3000})(14)(22.5)}{1000} = 25.88 \text{ K}$$

   $$\tfrac{1}{2}\phi V_c = 12.94 \text{ K}$$

   Calculate $V_{c_1} = (4\sqrt{f'_c}) b_w d = (4\sqrt{3000})(14)(22.5)/1000 = 69$ K. Calculate $V_{c_2} = (8\sqrt{f'_c}) b_w d = 138$ K.
3. Design $V_u = 69.83$ K $> \phi V_c = 25.88$ K; therefore, shear reinforcement must be provided. The distance x' at which no shear reinforcement is needed (at $\frac{1}{2}\phi V_c$) is

   $$x' = \left(\frac{91.2 - 12.94}{91.2}\right)(8) = 6.86 \text{ ft} = 82 \text{ in.}$$

   (from the triangles of shear diagram, Fig. 8.12).

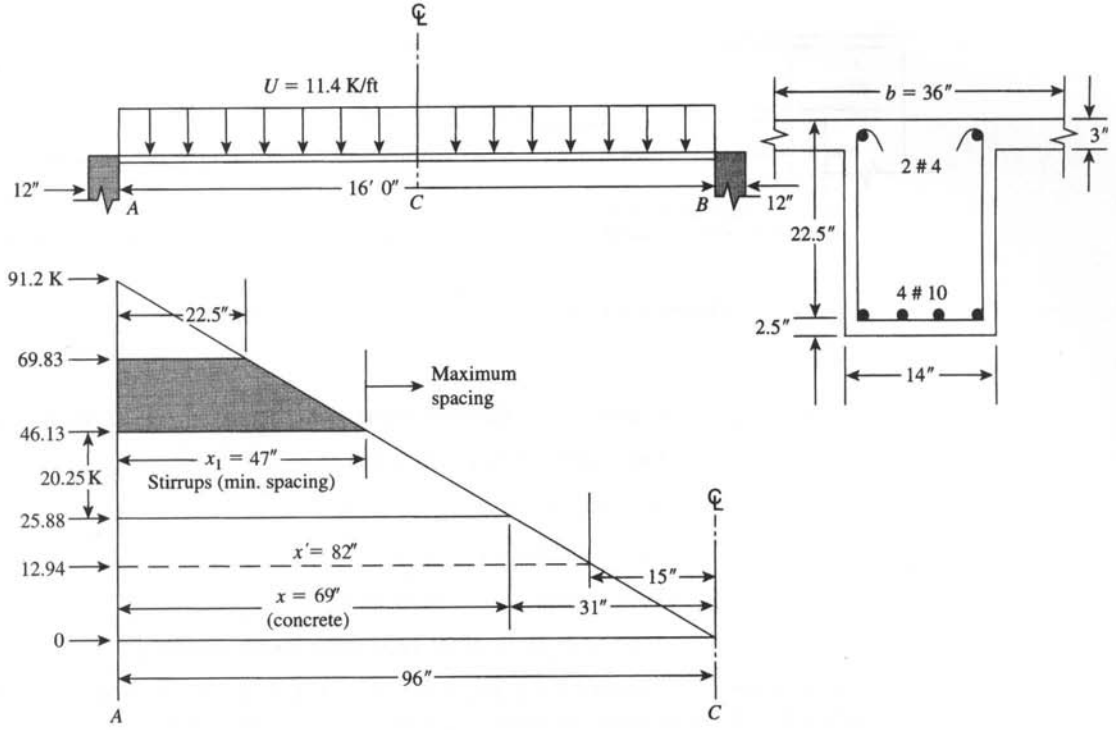


Figure 8.12 Example 8.2.

4. Calculate $V_s = (V_u - \phi V_c)/\phi = (69.83 - 25.88)/0.75 = 58.6$ K. Because V_s is less than $V_{c_1} = (4\sqrt{f'_c})b_w d$, then $S_{\max} = d/2$ must be considered (or refer to Fig. 8.10 or Table 8.2: $V_s < 52.8$ K).
5. Design of stirrups: Choose no. 3 U-stirrups, $A_v = 2(0.11) = 0.22\,\text{in.}^2$ Calculate S_1 based on $V_s = 58.6$ K, $S_1 = A_v f_y d/V_s = 13.2\,d/V_s = 5.07$ in., say, 5 in. (or get $s/d = 0.225$ from Table 8.2 or Fig. 8.10).
6. Calculate maximum spacings: $S_2 = d/2 = 22.5/2 = 11.25$ in., say, 11.0 in.; $S_3 = A_v f_{yt}/50b_w = 0.22(60{,}000)/50(14) = 18.9$ in. (or use Table 8.1); $S_4 = 24$ in.; $S_{\max} = 11$ in. controls.
7. Because $S_1 = 5$ in. $< S_{\max} = 11$ in., use no. 3 U-stirrups spaced at 5 in.
8. Calculate V_s for maximum spacings of 11 in.:

$$V_s = \frac{A_v f_{yt} d}{S} = \frac{0.22(60)(22.5)}{11} = 27 \text{ K}$$

$$\phi V_s = 20.25 \text{ K}$$

$$\phi V_c + \phi V_s = 25.88 + 20.25 = 46.13 \text{ K}$$

The distance x_1 at which $S = 11$ can be used is

$$\left(\frac{91.2 - 46.13}{91.2}\right)(96) = 47 \text{ in.}$$

Because x_1 is relatively small, use $S = 5$ in. for a distance greater than or equal to 47 and then use $S = 11$ for the rest of the beam. *Note* : If x_1 is long, then an intermediate spacing between 5 in. and 11 in. may be added.

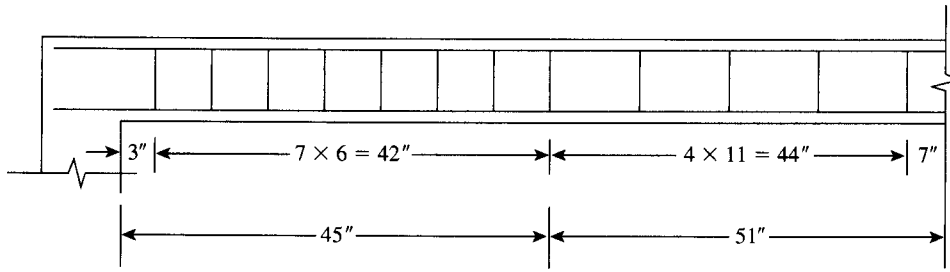


Figure 8.13 Example 8.2: distribution of stirrups.

9. Distribute stirrups as follows: Place the first stirrup at $S/2$ from the face of the support.

First stirrup at $S/2 = 5/2 = 2$ in.

Nine stirrups at $S = 5 = 45$ in.

Total $= 45 + 2$ in. $= 47$ in.

Four stirrups at $S = 11 = 44$ in.

Total $= 91$ in. > 82 in. (minimum length required)

The total number of stirrups for the beam is $2(1 + 9 + 4) = 28$. Distribution of stirrups is shown in Fig. 8.13, whereas calculated shear forces are shown in Fig. 8.12.

10. Place two no. 4 bars at the top of beam section to act as stirrup hangers.

8.9 SHEAR FORCE DUE TO LIVE LOADS

In Example 8.2, it was assumed that the dead and live loads are uniformly distributed along the full span, producing zero shear at midspan. Actually, the dead load does exist along the full span, but the live load may be applied to the full span or part of the span, as needed to develop the maximum shear at midspan or at any specific section. Figure 8.14*a* shows a simply supported beam with a uniform load acting on the full span. The shear force varies linearly along the beam, with maximum shear acting at support A.

In the case of live load, $W_2 = 1.6W_L$, the maximum shear force acts at support A when W_2 is applied on the full span, Fig. 8.14*a*. The maximum shear at midspan develops if the live load is placed on half the beam, BC (Fig. 8.14*b*), producing V_u at midspan equal to $W_2L/8$. Consequently, the design shear force is produced by adding the maximum shear force due to live load (placed at different lengths of the span) to the dead load shear force (Fig. 8.14*c*) to give the shear distribution shown in Fig. 8.14*d*. It is a common practice to consider the maximum shear at support A to be $W_uL/2 = (1.2W_D + 1.6W_L)L/2$, whereas V_u at midspan is $W_2L/8 = (1.6W_L)L/8$ with a straight-line variation along AC and CB, as shown in Fig. 8.14*d*. The design for shear in this case will follow the same procedure explained in Example 8.2. If the approach is applied to the beam in Example 8.2, then V_u (at A) $= 91.2$ K and V_u (at midspan) $= (1.6 \times 3)(16/8) = 10$ K.

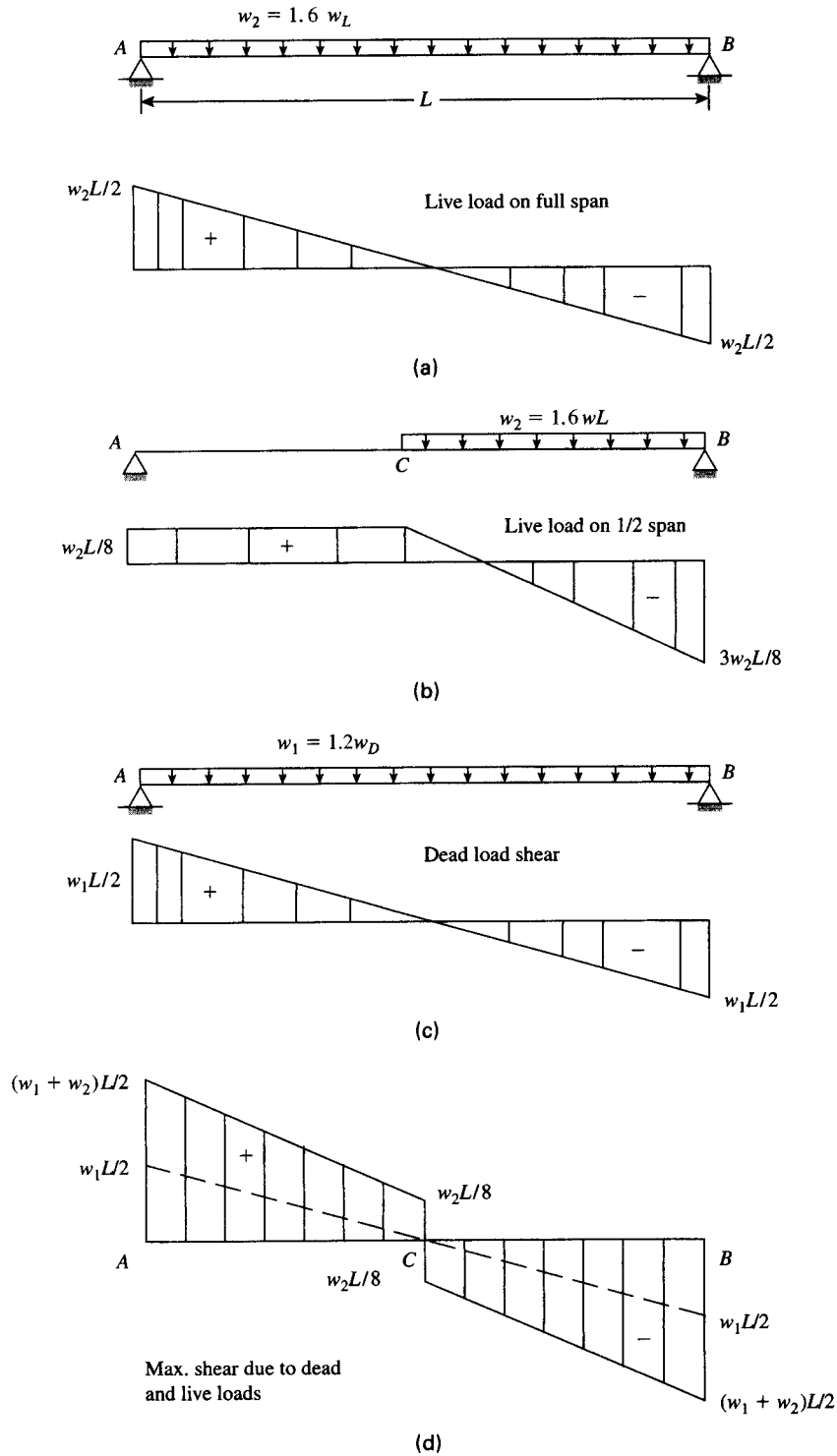


Figure 8.14 Effect of live load application on part of the span.

Example 8.3

A 10-ft-span cantilever beam has a rectangular section and carries uniform and concentrated factored loads (self-weight is included), as shown in Fig. 8.15. Using $f'_c = 4$ ksi normal-weight concrete and $f_y = 60$ ksi, design the shear reinforcement required for the entire length of the beam according to the ACI Code.

Solution

1. Calculate the shear force along the beam due to external loads.

$$V_u \text{ (at support)} = 5.5(10) + 20 + 8 = 83 \text{ K}$$

$$V_{ud} \text{ (at } d \text{ distance)} = 83 - 5.5\left(\frac{20.5}{12}\right) = 73.6 \text{ K}$$

$$V_u \text{ (at 4 ft left)} = 83 - 4(5.5) = 61 \text{ K}$$

$$V_u \text{ (at 4 ft right)} = 61 - 20 = 41 \text{ K}$$

$$V_u \text{ (at free end)} = 8 \text{ K}$$

The shear diagram is shown in Fig. 8.15.

2. Calculate ϕV_c:

$$\phi V_c = 2\lambda\sqrt{f'_c}bd = 2(0.75)(1)\sqrt{4000}(12)(20.5) = 23.34 \text{ K}$$

$$\frac{1}{2}\phi V_c = 11.67 \text{ K}$$

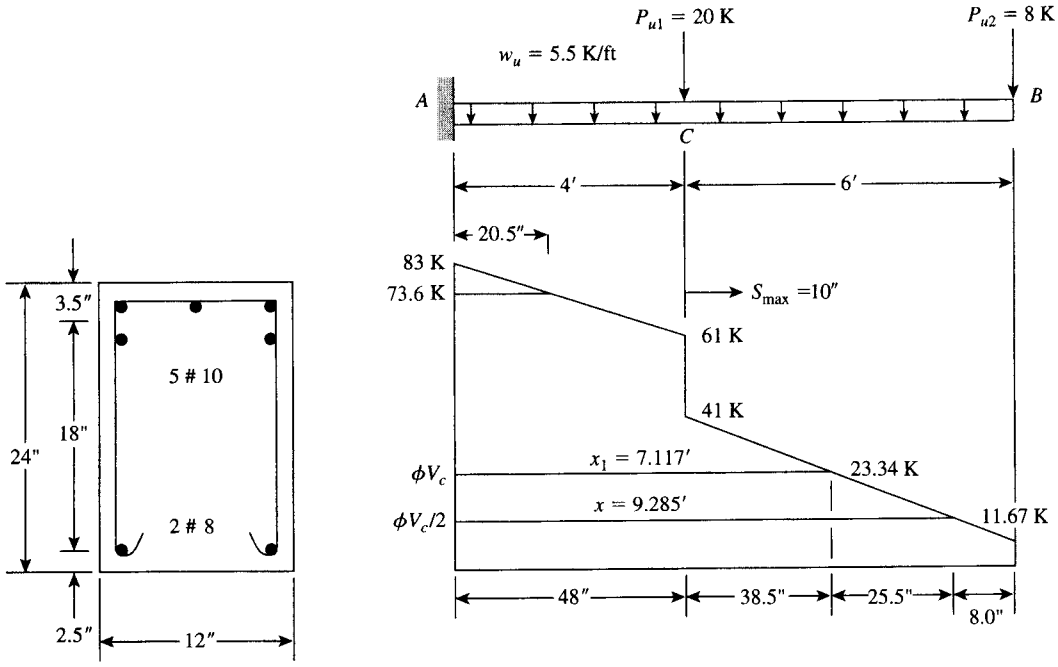


Figure 8.15 Example 8.3.

Because $V_{ud} > \phi V_c$ shear reinforcement is required. Calculate

$$V_{c_1} = 4\sqrt{f'_c}bd = 4\sqrt{4000}(12)(20.5) = 62.2 \text{ K}$$

$$V_{c_2} = 8\sqrt{f'_c}bd = 2V_1 = 124.4 \text{ K}$$

The distance x at which no shear reinforcement is needed (at $\frac{1}{2}\phi V_c = 11.67$ K), measured from support A:

$$x = 4 + \left(\frac{41 - 11.67}{41 - 8}\right)6 = 9.33 \text{ ft} = 112 \text{ in.}$$

(8.0 in. from free end). Similarly, x_1 for ϕV_c is 7.21 ft from A (33.5 in. from the free end).

3. Part AC: Design shear $V_u = V_{ud} = 73.6$ K. Calculate $V_s = (V_u - \phi V_c)/\phi = (73.6 - 23.34)/0.75 = 67$ K. Because $V_{c_1} < V_s < V_{c_2}$, $S_{max} \leq d/4$ must be considered (or check Fig. 8.10).
4. Design stirrups: Choose no. 3 U-stirrups, $A_v = 0.22 \text{ in.}^2$ Calculate S_1 (based on V_s):

$$S_1 = \frac{A_v f_{yt} d}{V_s} = \frac{13.2d}{V_s} = \frac{13.2(20.5)}{67} = 4.0 \text{ in.}$$

Use 4.0 in. (or get $s/d = 0.22$ from Fig. 8.10).

5. Calculate maximum spacings: $S_2 = d/4 = 20.5/4 = 5.12$ in., so use 5.0 in.

$$S_3 = \frac{A_v f_{yt}}{50 b_w} = 22 \text{ in.} \quad \text{(from Table 8.1 for } b = 12 \text{ in.)}$$

$$S_4 = 12 \text{ in.}$$

Then $S_{max} = 5.0$ in.

6. Because $S = 4$ in. $< S_{max} = 5.1$ in., use no. 3 stirrups spaced at 4 in.
7. At C, design shear $V_u = 61$ K $> \phi V_c$. Then $V_s = (61 - 23.34)/0.75 = 50.2$ K, $S_1 = A_v f_{yt} d/V_s = 5.4$ in.

$$V_s = 50.2 \text{ K} < V_{c_1} = 62.2 \text{ K} \quad S_2 = \frac{d}{2} = \frac{20.5}{2} = 10.25 \text{ in.} \quad \text{(or 10 in.)}$$

$S_1 = 5.4$ in. $< S_2$; then $S_1 = 5.4$ or 5.0 in. controls.

8. Because spacings of 5.5 in. and 4.0 in. are close, use no. 3 U-stirrups spaced at 4 in. for part AC.
9. Part BC:

   a. $V_u = 41 \text{ K} > \phi V_c$

   $$V_s = (V_u - \phi V_c)/\phi = (41 - 23.34)/0.75 = 23.55 \text{ K} < V_{c_1} = 62.2 \text{ K}$$

   b. $S_1 = A_v f_{yt} d/V_s = (13.2)(20.5)/23.55 = 11.5$ in.

   c. $S_2 = d/2 = 20.5/2 = 10.25$ in. (or less than $S_3 = 22$ in. or $S_4 = 24$ in.). Let $S_{max} = 10$ in. Choose no. 3 stirrups spaced at 10 in. for part BC.

10. Distribution of stirrups measured from support A: Place the first stirrup at

$$\frac{S}{2} = \frac{4}{2} = 2 \text{ in.}$$

$$12 \times 4 \text{ in.} = \underline{48 \text{ in.}}$$

$$50 \text{ in.}$$

$$6 \times 10 \text{ in.} + 1 \times 8 \text{ in.} = \underline{68 \text{ in.}}$$

$$\text{Total} \quad 118 \text{ in.}$$

Distance left to the free end is 7 in., which is less than 8.0 in., where no stirrups are needed. Distribution of stirrups is shown in Fig. 8.16. Total number of stirrups is 20.

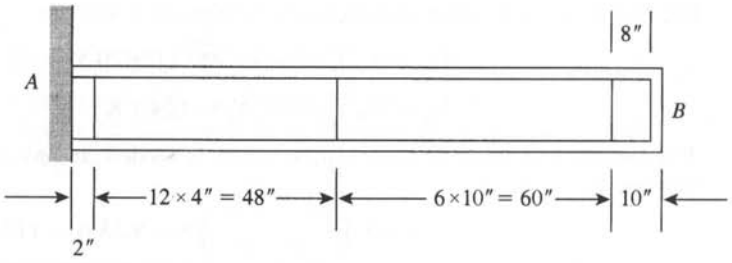


Figure 8.16 Example 8.3: distribution of stirrups.

8.10 SHEAR STRESSES IN MEMBERS OF VARIABLE DEPTH

The shear stress, v, is a function of the effective depth, d; therefore, shear stresses vary along a reinforced concrete beam with variable depth [10]. In such a beam (Fig. 8.17), consider a small element dx. The compression force C at any section is equal to the moment divided by its arm, or $C = M/y$. The first derivative of C is

$$dC = \frac{ydM - Mdy}{y^2}$$

If C_1 is greater than C_2, then $C_1 - C_2 = dC = vb\,dx$

$$vbdx = \frac{ydM - Mdy}{y^2} = \frac{dM}{y} - \frac{M}{y^2}dy$$

$$v = \frac{1}{yb}\left(\frac{dM}{dx}\right) - \frac{M}{by^2}\left(\frac{dy}{dx}\right)$$

Because $y = jd$, dM/dx is equal to the shearing force V and $d(jd)/dx$ is the slope,

$$v = \frac{V}{bjd} - \frac{M}{b(jd)^2}\left[\frac{d}{dx}(jd)\right] \quad \text{and} \quad v = \frac{V}{bjd} \pm \frac{M}{b(jd)^2}(\tan\alpha) \tag{8.26}$$

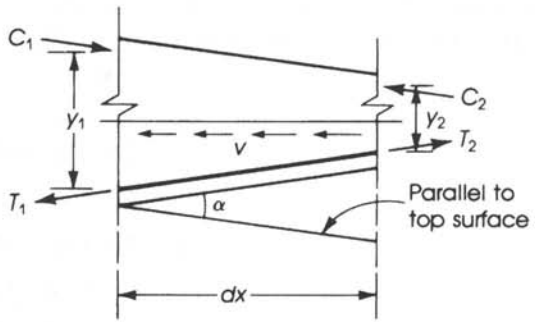


Figure 8.17 Shear stress in a beam with variable depth.

where V and M are the external shear and moment, respectively, and α is the slope angle of one face of the beam relative to the other face. The plus sign is used when the beam depth decreases as the moment increases, whereas the minus sign is used when the depth increases as the moment increases. This formula is used for small slopes, where the angle α is less than or equal to 30°.

A simple form of Eq. 8.26 can be formed by eliminating the j value:

$$v = \frac{V}{bd} \pm \frac{M}{bd^2} (\tan \alpha) \tag{8.27}$$

For the strength design method, the following equation may be used:

$$v_u = \frac{V_u}{\phi bd} \pm \frac{M_u}{\phi bd^2} (\tan \alpha) \tag{8.28}$$

For the shearing force,

$$\phi V_n = V_u \pm \frac{M_u}{d} (\tan \alpha) \tag{8.29}$$

Figure 8.18 shows a cantilever beam with a concentrated load P at the free end. The moment and the depth d increase toward the support. In this case a negative sign is used in Eqs. 8.27, 8.28, and 8.29. Similarly, a negative sign is used for section t in the simply supported beam shown, and a positive sign is used for section Z, where moment increases as the depth decreases.

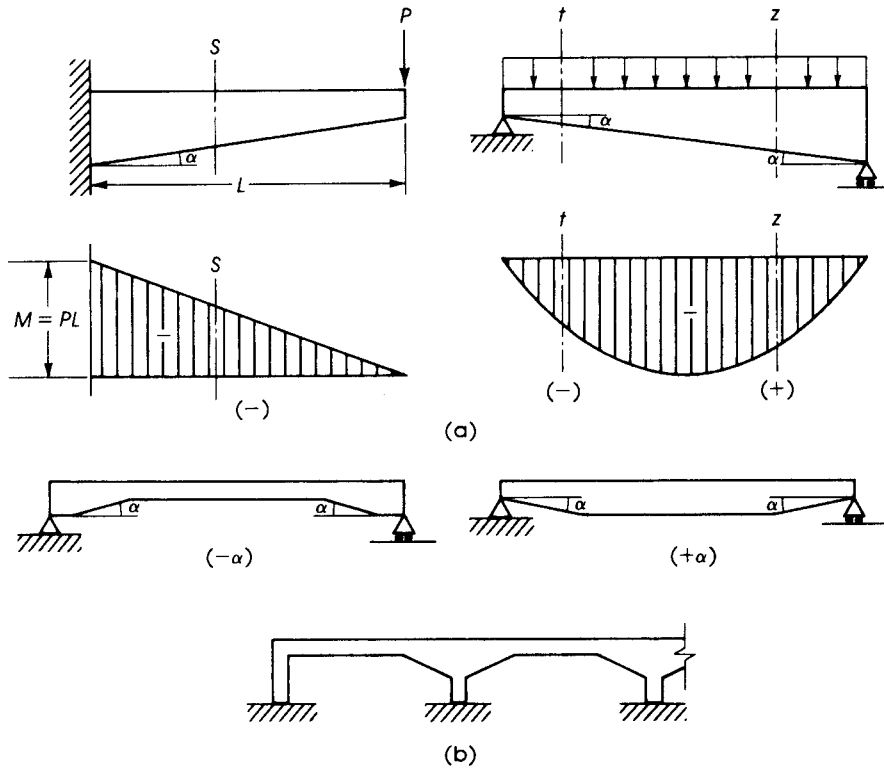


Figure 8.18 Beams with variable depth: (*a*) moment diagrams and (*b*) typical forms.

In many cases, the variation in the depth of beams occurs on parts of the beams near their supports (Fig. 8.18).

Tests [11] on beams with variable depth indicate that beams with greater depth at the support fail mainly by shear compression. Beams with smaller depth at the support fail generally by an instability type of failure, caused by the propagation of the major crack in the beam upward and then horizontally to the beam's top section. Tests also indicate that for beams with variable depth (Fig. 8.18) with an inclination α of about 10° and subjected to shear and flexure, the concrete shear strength, V_{cv}, may be computed by

$$V_{cv} = V_c(1 + \tan\alpha) \tag{8.30}$$

where

V_{cv} = shear strength of beam with variable depth

V_c = ACI Code Eq. 11.5

$$= \left(1.9\lambda\sqrt{f'_c} + 2500\rho_w \frac{V_u d_s}{M_u}\right) b_w d_s \leq 3.5\lambda\sqrt{f'_c} b_w d_s$$

α = angle defining the orientation of reinforcement, considered positive for beams of small depth at the support and negative for beams with greater depth at the support (Fig. 8.18)

d_s = effective depth of the beam at the support

The simplified ACI Code, Eq. 11.3,can also be used to compute V_c:

$$V_c = (2\lambda\sqrt{f'_c}) b_w d_s \tag{8.31}$$

Example 8.4

Design the cantilever beam shown in Fig. 8.19 under the factored loads applied if the total depth at the free end is 12 in., and it increases toward the support. Use a steel percentage $\rho = 1.5\%$, $f'_c = 4$ ksi normal-weight concrete, $f_y = 60$ ksi, and $b = 10$ in.

Solution

1. M_u (support) $= (2.5/2)(8)^2(12) + (14)(8)(12) = 2304$ K·in.
2. For $\rho = 1.5\%$, $R_u = 703$ psi (from Table 4.1).

$$d = \sqrt{\frac{M}{R_u b}} = \sqrt{\frac{2304}{0.703 \times 10}} = 18.1 \text{ in.}$$

$A_s = 0.015 \times 10 \times 18.1 = 2.72 \text{ in.}^2$ (use three no. 9 bars); let actual $d = 19.5$ in., $h = 22$ in.

3. Design for shear: Maximum shear at the support is $14 + 20 = 34$ K. Because the beam section is variable, moment effect shall be considered; because the beam depth increases as the moment increases, a minus sign is used in Eq.8.28.

$$v_u = \frac{V_u}{\phi b d} - \frac{M_u}{\phi b d^2}(\tan\alpha)$$

To find $\tan\alpha$, let d at the free end be 9.5 in., and d at the support be 19.5 in.:

$$\tan\alpha = \frac{19.5 - 9.5}{8 \times 12} = 0.1042$$

$$v_u \text{ (at the support)} = \frac{34{,}000}{(0.75 \times 10 \times 19.5)} - \frac{2304 \times 1000 \times 0.1042}{[0.75 \times 10 \times (19.5)^2]}$$

$$= 148 \text{ psi}$$

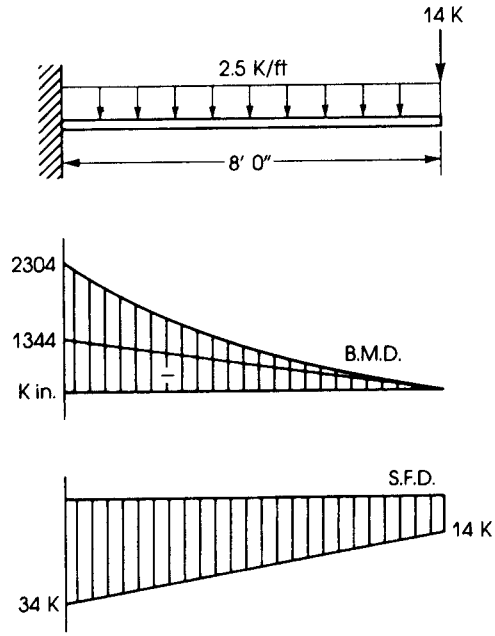


Figure 8.19 Example 8.4 with bending moment diagram (*middle*) and shear force diagram (*bottom*).

4. Shear stress at the free end is $V_u/\phi bd (M_u = 0)$.

$$v_u = \frac{14{,}000}{0.75 \times 10 \times 9.5} = 196 \text{ psi}$$

5. At a distance $d = 18$ in. from the face of the support, the effective depth is 17.6 in. (from geometry),

$$V_u = 34 - 2.5 \times \frac{18}{12} = 30.25 \text{ K}$$

$$M_u \text{ (at 18 in. from support)} = 14 \times 78 + \frac{2.5}{12} \times \frac{(78)^2}{2}$$

$$= 1726 \text{ K·in.}$$

$$v_u = \frac{30.25}{0.75 \times 10 \times 18} - \frac{1726 \times 1000 \times 0.1042}{0.75 \times 10 \times (18)^2}$$

$$= 150 \text{ psi}$$

6. At midspan (48 in. from the support),

$$d = 14.5 \text{ in.}$$

$$V_u = 14 + 10 = 24 \text{ K}$$

$$M_u = 14 \times 48 + \frac{2.5}{12} \times \frac{(48)^2}{2} = 912 \text{ K·in.}$$

$$v_u = \frac{24{,}000}{0.75 \times 10 \times 14.5} - \frac{912 \times 1000 \times 0.1042}{0.75 \times 10 \times (14.5)^2} = 160 \text{ psi}$$

Similarly, at 6 ft from the support (2 ft from the free end),

$$d = 12 \text{ in.} \quad V_u = 19 \text{ K} \quad M_u = 396 \text{ K·in.}$$
$$v_u = 173 \text{ psi}$$

At 1 ft from the free end,

$$d = 10.75 \text{ in.} \quad V_u = 16.5 \text{ K} \quad M_u = 183 \text{ K·in.}$$
$$v_u = 182 \text{ psi}$$

These values are shown in Fig. 8.20.

7. Shear stress resisted by concrete is

$$2\lambda\sqrt{f'_c} = (2)(1)\sqrt{4000} = 126.5 \text{ psi}$$

Minimum shear stress to be resisted by shear reinforcement

$$v_{us} = 196 - 126.6 = 69.5 \text{ psi}$$

(V_u and consequently v_{us} have already been increased by the ratio $1/\phi$ in Eq. 8.28).

8. Choose no. 3 stirrups with two legs.

$$A_v = 2 \times 0.11 = 0.22 \text{ in.}^2$$
$$S \text{ (required)} = \frac{A_v f_{yt}}{v_s b_w} = \frac{0.22 \times 60{,}000}{69.5 \times 10} = 19 \text{ in.}$$
$$S_{max} \left(\text{for } \frac{d}{2}\right) = 9.5 \text{ in. to } 4.5 \text{ in. at the free end}$$
$$S_{max} \text{ (for minimum} A_v) = \frac{A_v f_{yt}}{50 b_w} = \frac{0.22 \times 60{,}000}{50 \times 10} = 26.4 \text{ in.}$$

9. Check for maximum spacing ($d/4$): $v_{us} \leq 4\sqrt{f'_c}$.

$$4\sqrt{f'_c} = (4)\sqrt{4000} = 253 > 69.5 \text{ in.}$$

10. Distribution of stirrups (distances from the free end):

1 stirrup at 2 in. = 42 in.
10 stirrups at 4.5 in. = 45 in.
3 stirrups at 7 in. = 21 in.
3 stirrups at 8 in. = 24 in.
Total = 92 in.

There is 4 in. left to the face of the support.

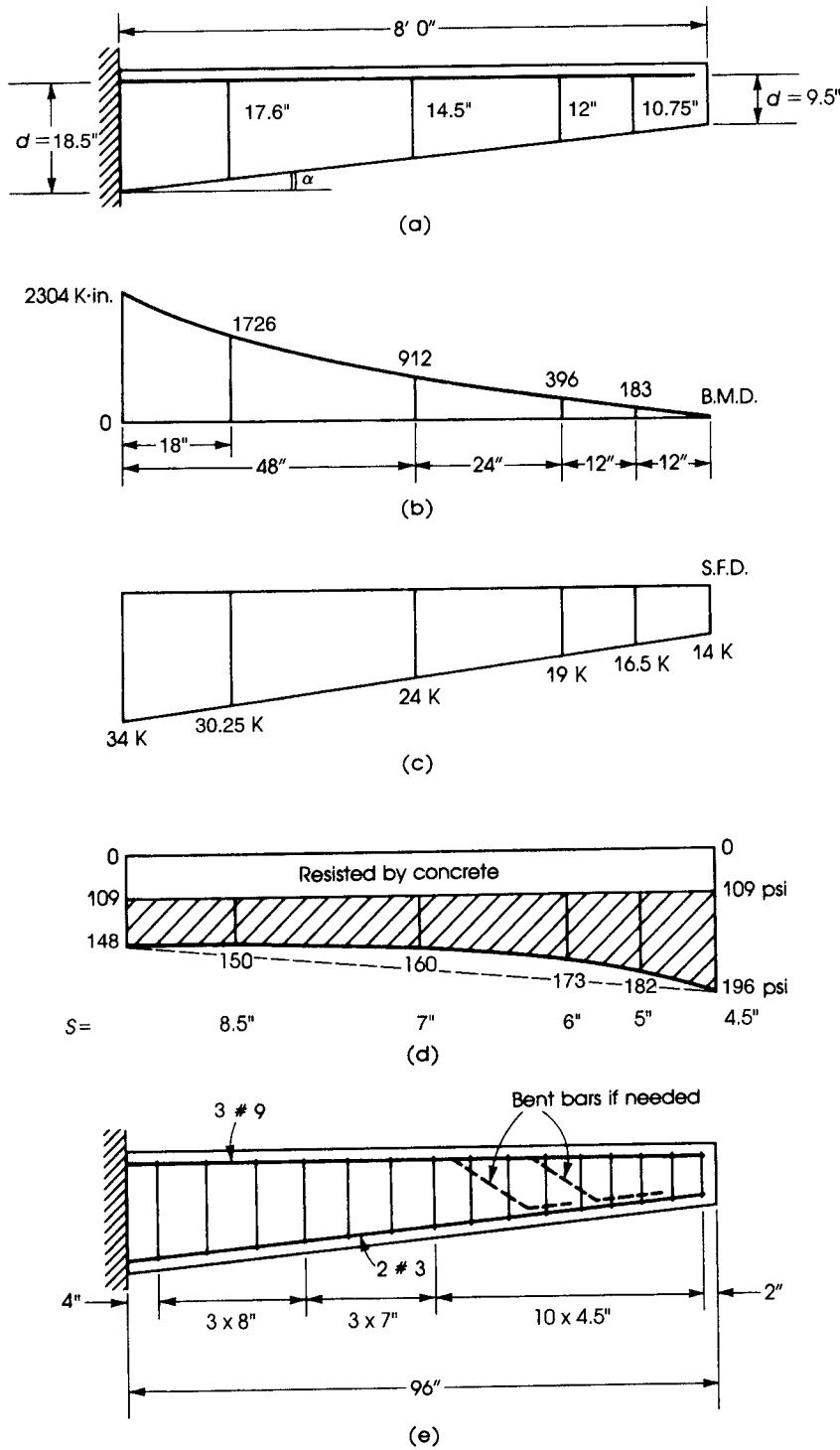


Figure 8.20 Example 8.4: web reinforcement for a beam of variable depth.

8.11 DEEP FLEXURAL MEMBERS

Flexural members should be designed as deep beams if the ratio of the clear span, l_n (measured from face to face of the supports; Fig. 8.21), to the overall depth, h, is less than 4 (ACI Code, Section 11.8). The members should be loaded on one face and supported on the opposite face so that compression struts can develop between the loads and supports (Fig. 8.22). If the loads are applied through the bottom or sides of the deep beam, shear design equations for ordinary beams given earlier should be used. Examples of deep beams are short-span beams supporting heavy loads, vertical walls under gravity loads, shear walls, and floor slabs subjected to horizontal loads.

The definition of deep flexural members is also presented in the ACI Code, Section 10.7.1. It indicates that flexural members where the ratio of the clear span, l_n, to the overall depth, h (Fig. 8.21), is less than 4 and regions loaded with concentrated loads within twice the member depth from the face of the support are considered deep flexural members. Such beams should be designed taking into account nonlinear distribuion of stress and lateral buckling (Fig. 8.22*a*).

Figure 8.22*a* shows the elastic stress distribution at the midspan section of a deep beam, and Fig. 8.22*b* shows the principal trajectories in top-loaded deep beams. Solid lines indicate tensile stresses, whereas dashed lines indicate compressive stress distribution. Under heavy loads, inclined vertical cracks develop in the concrete in a direction perpendicular to the principal tensile stresses and almost parallel to the dashed trajectories (Fig. 8.22*c*). Hence, both horizontal and vertical reinforcement is needed to resist principal stresses. Moreover, tensile flexural reinforcement is needed within about the bottom one-fifth of the beam along the tensile stress trajectories (Fig. 8.22*b*). In general, the analysis of deep beams is complex and can be performed using truss models or more accurately using a finite-element approach or similar methods. A simplified provision for the shear design of deep beams can be presented in steps as follows:

1. Critical section: If the critical section for shear design in deep beams supporting top vertical loads is located at a distance X from the face of the support, then the distance X can be determined as follows (Fig. 8.23):
   - **a.** For deep beams supporting uniformly distributed loads, $X = 0.15l_n$, where l_n = clear span.
   - **b.** For concentrated loads, $X_1 = 0.5a_1$ (left support) or $X_2 = 0.5a_2$ (right support) (Fig. 8.23), where a_1 and a_2 equal the shear span near each support. The shear span is the distance between the concentrated load and the face of the support.

   In all cases, the distances X, X_1, and X_2 must not exceed the effective depth, d.

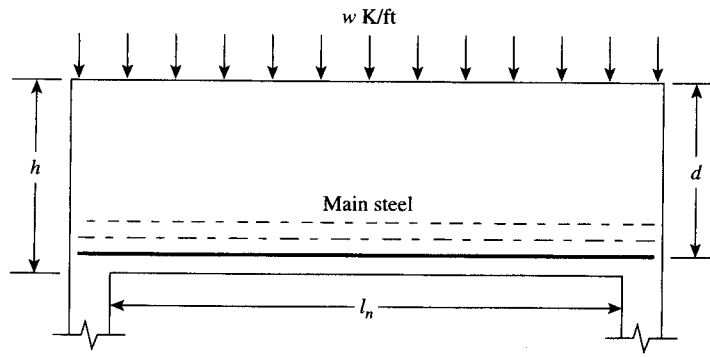


Figure 8.21 Single-span deep beam ($l_n/d < 5$).

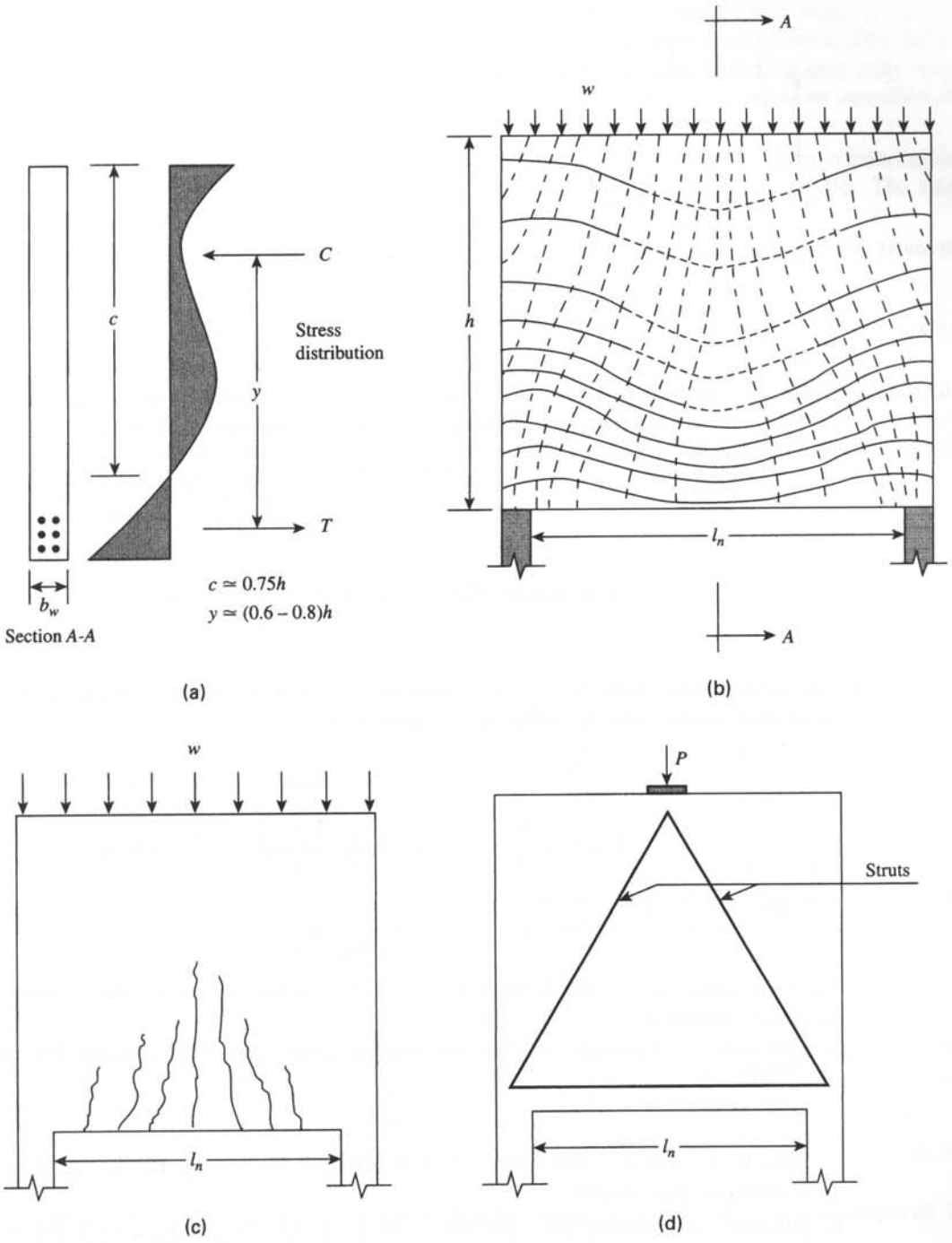


Figure 8.22 Stress distribution and cracking: (*a*) elastic stress distribution, (*b*) stress trajectories (tension, solid lines, and compression, dashed lines), (*c*) cracks pattern, and (*d*) truss model for a concentrated load applied at the wall upper surface.

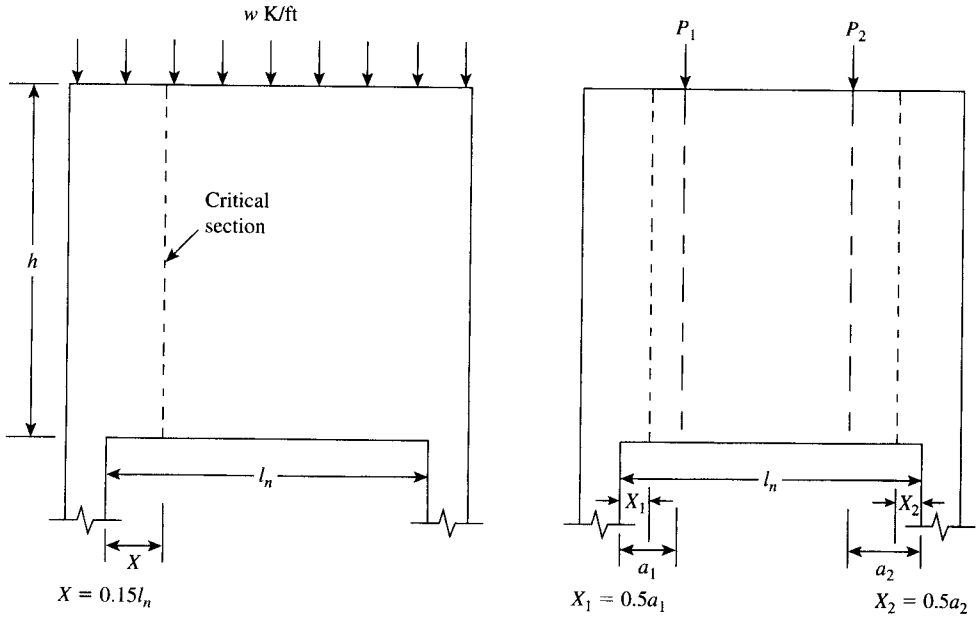


Figure 8.23 Critical sections for shear design.

2. Maximum shear strength ϕV_n: The maximum shear strength, ϕV_n, for deep flexural members shall not exceed the following values ($\phi = 0.75$):

$$\text{For } \frac{l_n}{d} < 2, \quad \phi V_n = \phi 8\sqrt{f'_c} b_w d \tag{8.32a}$$

$$\text{For } 2 \le \frac{l_n}{d} \le 5, \quad \phi V_n = \phi \frac{2}{3}\left(10 + \frac{l_n}{d}\right)\sqrt{f'_c} b_w d \tag{8.32b}$$

or let

$$\phi V_n = \phi 10\sqrt{f'_c} b_w d \tag{8.33}$$

for both cases, ACI Code, Section 11.7.3. If V_u exceeds ϕV_n, then the section dimensions must be increased.

3. a. Concrete shear strength, V_c: The nominal shear strength, V_c, of concrete can be estimated as follows:

$$V_c = 2\lambda\sqrt{f'_c} b_w d \tag{8.34}$$

This V_c is similar to the concrete shear strength for regular beams, as in the previous sections of this chapter.

b. Alternatively, another expression may be used that takes into account the effect of the factored moment and shear at the critical section:

$$V_c = \left(3.5 - \frac{2.5M_u}{V_u d}\right)\left(1.9\lambda\sqrt{f'_c} + \frac{2500\rho_w V_u d}{M_u}\right) b_w d \tag{8.35}$$

but V_c should not exceed $6\sqrt{f'_c} b_w d$.

The value of $(3.5 - 2.5\ M_u/V_u d)$ may not be greater than 2.5 and must not be less than 1.0. The values of M_u and V_u are taken at the critical design section. This higher shear strength of Eq. 8.35 is used with the idea that minor unsightly cracking may occur in the deep beam and can be tolerated. Cracks may start to develop at about one-third the factored load.

4. Shear reinforcement: When the factored shear force, V_u, exceeds ϕV_c, shear reinforcement must be provided, considering that $V_u = \phi(V_c + V_s)$, or $V_s = (V_u - \phi V_c)/\phi$. The steps are as follows:
   a. Determine V_s: The force resisted by shear reinforcement V_s is determined from the following expression:

$$V_s = \left[\frac{A_v}{S_v}\left(\frac{1 + l_n/d}{12}\right) + \frac{A_{vh}}{S_h}\left(\frac{11 - l_n/d}{12}\right)\right] f_y d \tag{8.36}$$

   where A_v = total area of vertical shear reinforcement spaced at S_v and perpendicular to the main flexural tensile reinforcement on both faces of the beam and A_{vh} = total area of horizontal shear reinforcement spaced at S_h parallel to the main flexural tensile reinforcement on both faces of the beam.

   b. Spacing of shear reinforcement is

$$\text{Maximum vertical spacings} \quad S_v \le \frac{d}{5} \le 12 \text{ in.}$$

$$\text{Maximum horizontal spacings} \quad S_h \le \frac{d}{5} \le 12 \text{ in.}$$

   c. Minimum shear reinforcement: The area of vertical shear reinforcement is $A_v = 0.0025 b_w S_v$. The area of horizontal shear reinforcement is $A_{vh} = 0.0015\ b_w S_h$.

   d. The shear reinforcement required at the critical section should be extended throughout the length and depth of the deep beam.

   e. For continuous deep beams, the same shear reinforcement may be used in all spans if the spans are almost equal with similar loading.

5. Flexural reinforcement of deep beams: The flexural behavior of deep beams is complex and requires nonlinear analysis of stresses and strains along the depth of the beam. For a preliminary design, the following simplified approach may be used:

$$\phi M_n = \phi A_s f_y y$$

   where y = moment arm = $(d - a/2)$. Because the value of $(d - a/2)$ is not easy to calculate, the moment arm y may be taken approximately equal to $0.6h$ for $l_n/h = 1.0$ and equal to $0.8h$ for $l_n/h = 2.0$. Linear interpolation may be used to estimate y when l_n/h varies between 1.0 and 2.0. Therefore,

$$A_s = \frac{M_u}{\phi y f_y} \tag{8.37}$$

   The value of A_s may not be less than the minimum flexural reinforcement required for regular beams given next, assuming $d = 0.9h$:

$$\text{Minimum } A_s = \left(\frac{3\sqrt{f'_c}}{f_y}\right) b_w d \ge \left(\frac{200}{f_y}\right) b_w d \tag{8.38}$$

   The second term controls when $f'_c < 4500$ psi. Note that, and f_y are in psi.

The flexural tension reinforcement shoul d be placed within $h/4$ to $h/5$ of the beam and should be adequately spaced along the bottom tension zone. Tension bars should be well anchored to the supports.

For more accurate analysis and design and for continuous deep beams, a rigorous non-linear approach should be used to determine the proper amount and distribution of the tension reinforcement.

Example 8.5

A simply supported deep beam has a span = 14 ft, a clear span of $l_n = 12$ ft, a total height of $h = 8$ ft, and width of $b = 16$ in. The deep beam supports a uniform service dead load of 41 K/ft (including self-weight) and a live load of 22 K/ft on top of the beam. Design the beam for moment and shear using $f'_c = 4$ ksi normal-weight concrete, and $f_y = 60$ ksi. Refer to Fig. 8.24.

Solution

1. Design for moment:

$$W_u = 1.2W_D + 1.6W_L = 1.2(41) + 1.6(22) = 84.4 \text{ K/ft}$$

$$M_u = \frac{W_u L^2}{8} = \frac{84.4(14)^2}{8} = 2067.8 \text{ K·ft}$$

$$\frac{l_n}{h} = \frac{12}{8} = 1.5$$

Determine the moment arm, y. For $l_n/h = 1.0$, $y = 0.6d$, and for $l_n/h = 2.0$, $y = 0.8d$; hence for $l_n/h = 1.5$, $y = 0.7d$ (by interpolation) $= 0.7(8 \times 12) = 67.2$ in.

$$A_s = \frac{M_u}{\phi y f_y} = \frac{2067.8 \times 12}{0.9(67.2)(60)} = 6.84 \text{ in.}^2$$

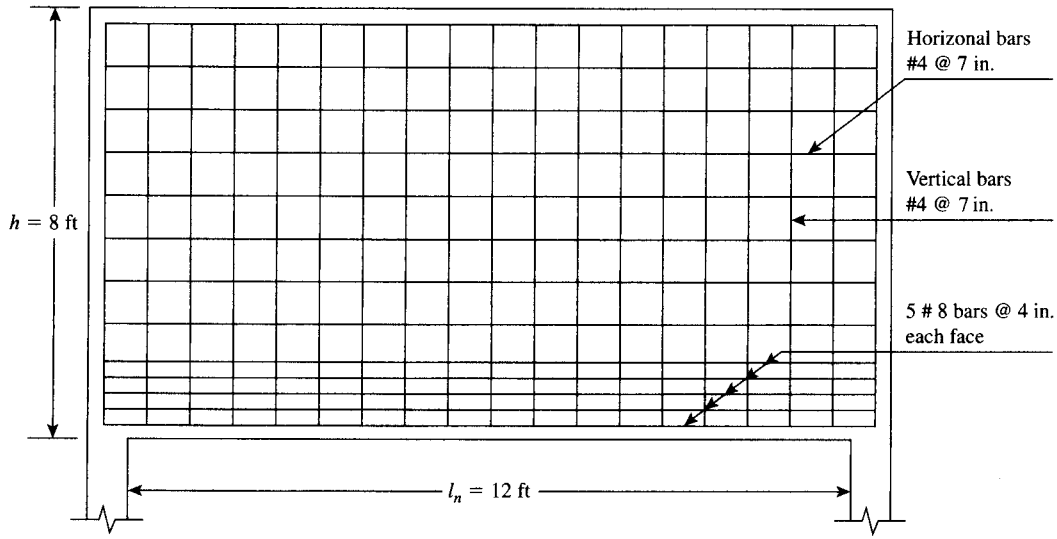


Figure 8.24 Example 8.5.

Assume $d = 0.9h = 0.9(8 \times 12) = 86.4$ in. Because $f'_c < 4500$ psi,

$$A_s \text{ (minimum)} = \left(\frac{200}{f_y}\right) b_w d = \frac{200(16)(86.4)}{60{,}000} = 4.6 \text{ in.}^2$$

Therefore, $A_s = 6.84$ in.^{2} controls. Choose 10 no. 8 bars (7.85 in.^{2}), five on each face, distributed within $h/5 = 8(12)/5 = 19.2$ in. of the tension zone of the beam. Spacing of bars $= 19.2/5 = 3.84$ in., or 4 in. Bars should be well anchored into the supports.

2. Design for shear:

a. Calculate V_u and M_u at the distance $x = 0.15l_n = d$ from the face of the support.

$$0.15l_n = 0.15(12 \times 12) = 21.6 \text{ in.} = 1.8 \text{ ft} < d = 86.4 \text{ in.}$$

$$\text{Design } V_u = 84.4(12/2) - 84.4(1.8) = 354.5 \text{ K}$$

$$M_u = 84.4(6)(1.8) - \frac{84.4(1.8)^2}{2} = 774.8 \text{ K·ft}$$

$$\frac{M_u}{V_u d} = \frac{774.8(12)}{354.5(86.4)} = 0.304$$

b. Calculate V_c:

$$3.5 - 2.5\frac{M_u}{V_u d} = 3.5 - 2.5(0.304) = 2.74 > 2.5$$

So, use 2.5 In this case, determine $M_u/V_u d$ to be used to calculate V_c: $2.5 = 3.5 - 2.5\ M_u/(V_u d)$, and $M_u/(V_u d) = 0.4$.

$$\frac{V_u d}{M_u} = \frac{1}{0.4} = 2.5$$

$$\rho_w = \frac{A_s}{b_w d} = \frac{7.85}{16 \times 86.4} = 0.00496$$

$$V_c = 2.5\left(1.9\lambda\sqrt{f'_c} + \frac{2500\rho_w V_u d}{M_u}\right) b_w d$$

$$= 2.5[(1.9)(1)\sqrt{4000} + (2500)(0.00496)(2.5)](16)(86.4)$$

$$= 522.4 \text{ K}$$

$$V_c \le 6\sqrt{f'_c} b_w d = 6\sqrt{4000}(16)(86.4) = 524.6 \text{ K}$$

Hence, $V_c = 522.4$ controls and $\phi V_c = 392$ K.

c. Calculate $V_s = (V_u - \phi V_c)/\phi$. Because $\phi V_c = 392 \text{ K} > V_u = 354.5$ K, then $V_s = 0$, and only minimum shear reinforcement is required.

d. Calculate shear reinforcement: Assume no. 4 bars placed on both faces in the horizontal and vertical directions; then $A_v = A_{vh} = 2(0.2) = 0.4$ in.^{2} Maximum allowable spacing of vertical bars is $S_v = d/5 = 18$ in. $S_v = 86.4/5 = 17.3$ in. > 12 in. use $S_u = 12$ in. Maximum allowable spacing of horizontal bars is $S_h = d/5 = 18$ in. $S_h = 86.4/5 = 17.3$ in > 12 in.; use $S_h = 12$ in. Minimum A_v (vertical) $= 0.0025 b_w S_v = 0.0025(16)(72) = 0.48$ in.$^2 > 0.4$ in.^{2} Minimum A_{vh} (horizontal) $= 0.0015 b_w S_h = 0.0015(16)(12) = 0.288$ in.$^2 < 0.4$ in.^{2} Reduce spacing to $S_v = 0.4/(0.0025 \times 16) = 10$ in. Therefore, use no. 4 vertical bars spaced at 10 in., and use no. 4 horizontal bars spaced at 12 in.

3. If $V_c = 2\lambda\sqrt{f'_c} b_w d$ is used, then $V_c = (2)(1)\sqrt{4{,}000}(16)(86.4) = 174.9$ K and $\phi V_c = 0.75 V_c = 131$ K, which is less than $V_u = 354.5$ K. Hence, shear reinforcement is required.

$$V_s = \frac{V_u - \phi V_c}{\phi} = \frac{354.1 - 131}{0.75} = 297.5 \text{ K}$$

Assuming no. 4 bars placed on both faces in the vertical and horizontal directions, then $A_v = A_{vh} = 2(0.2) = 0.4\,\text{in.}^2$ Assuming that the spacings of bars in both directions are equal, $S_v = S_h = S$, and $l_n/d = 12 \times 12/86.4 = 1.67$, then

$$V_s = \left[\frac{A_v}{S_v}\left(\frac{1+l_n/d}{12}\right) + \frac{A_{vh}}{S_h}\left(\frac{11-l_n/d}{12}\right)\right] f_y d$$
$$297.5 = \left[\frac{0.4}{S}\left(\frac{1+1.67}{12}\right) + \frac{0.4}{S}\left(\frac{11-1.67}{12}\right)\right](60)(86.4) \tag{8.36}$$

$S = 7\,\text{in.}$, which is less than the maximums $S_v = 17.3\,\text{in.}$ and $S_h = 18\,\text{in.}$ Use $S = 7\,\text{in.}$ for both vertical and horizontal spacing.

$$\text{Minimum } A_v \text{ (vertical)} = 0.0025(16)(7) = 0.28\ \text{in.}^2 < 0.4\ \text{in.}^2$$

$$\text{Minimum } A_{vh} \text{ (horizontal)} = 0.0015(16)(7) = 0.168\ \text{in.}^2 < 0.4\ \text{in.}^2$$

Then use no. 4 bars spaced at 7 in. on both faces in the horizontal and vertical directions. A welded wire fabric mesh may be adopted to replace the preceding bar arrangements. It can be seen that this solution is more conservative than that given in step 2. Reinforcement details are shown in Fig. 8.24 on page 281.

Example 8.6: Strut and Tie Deep Beam

A simply supported deep beam has a clear span = 12 ft, a total height = 6 ft, and a width = 18 in. The beam supports an 18-in. square column at midspan carrying a dead load = 300 K, and a live load = 240 K. Design the beam using the strut and tie model, using $f'_c = 4$ ksi and $f_y = 60$ ksi. (Refer to Fig. 8.25).

Solution

1. Calculate the factored loads (Fig. 8.25):

$$\text{Weight of the beam} = 15 \times 6 \times 1.5 \times 0.150 = 20\text{K}$$

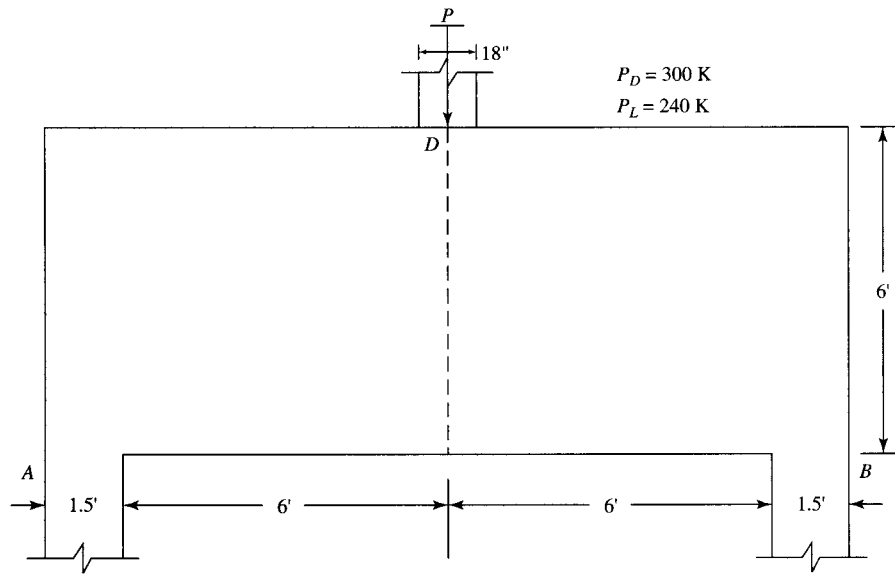


Figure 8.25 Example 8.6

Since the weight of the beam is small relative to the concentrated loads at midspan, add it to the concentrated load at midspan.

$$P_u = 1.2D + 1.6L = 1.2(300 + 20) + 1.6(240) = 768 \text{ K}$$

$$R_A = R_B = 768/2 = 384 \text{ K}$$

2. Check if the beam is deep according to to the ACI Code, Section 11.8: Clear span, $l_n = 12$ ft, $h = 6$ ft, and $l_n/h = 2 < 4$, a deep beam.
3. Calculate the maximum shear strength of the beam cross-section: Let V_u at $A = R_A = 384$ K, and assume $d = 0.9h = 0.9 \times 72 = 64$ in.

$$V_n = 10\sqrt{f'_c}b_w d = 10 \times \sqrt{4000}(18 \times 64) = 728.6 \text{ K}$$

$$\phi V_n = 0.75(728.6) = 546 \text{ K} > V_u \qquad \text{(o.k.)}$$

4. Select a truss model.
A triangular truss model is chosen. Assume that the nodes act at the centerline of the supports and at 6.0 in from the lower or upper edge of the beam (Fig. 8.26). The strut and tie model consists of a tie AB and two struts AD and BD. Also, the reactions at A and B and the load P_u at D represent vertical struts.

$$\text{Length of the diagonal strut } AD = \sqrt{(60)^2 + (80)^2} = 100.8 \text{ in.}$$

Let the angle between the strut and the tie $= \theta$, $\tan\theta = 60/81 = 0.7407$, and $\theta = 36.5$ degrees >26 degrees, which is o.k.

5. Calculate the forces in the truss members: The compression force in strut $AD = F_{AD} = F_{BD}$ = 384 (100.8/60) = 645 K. The tension force in the tie $AB = F_{AB} = 384$ (100.8/81) = 478 K.
6. Calculate the effective strength, f_{cu}. Assume that confining reinforcement is provided to resist the splitting forces. Struts AD and BD represent the bottle-shape compression members, and therefore, $\beta_s = 0.75$.

$$f_{ce} = 0.85\beta_s f'_c = 0.85 \times 0.75 \times 4 = 2.55 \text{ ksi}$$

The vertical struts at A, B, and D have uniform sections, and therefore $\beta_s = 1.0$.

$$f_{ce} = 0.85 \times 1.0 \times 4 = 3.4 \text{ ksi}$$

The nodal zone D has a C-C-C force and therefore, $\beta_s = 1.0$. The effective strength at nodal zone D is:

$$f_{ce} = 0.85 \times 1.0 \times 4 = 3.4 \text{ ksi}$$

Since the struts AD and BD connect to the other nodes, $f_{ce} = 2.55$ ksi controls to all nodal zones.

7. Design of nodal zones:
   a. Design of nodal zone at A : Assume that the faces of the nodal zone have the same stress of 2.55 ksi and the faces are perpendicular to their respective forces.

$$\phi F_n \geq F_u \quad \text{or} \quad \phi f_{ce} A_{cs} \geq F_u$$

   where $\phi = 0.75$ for struts, ties, and nodes. The length of the horizontal face ab (Fig. 8.27a) is equal to $F_u/(\phi f_{ce} b) = 384/(0.75 \times 2.55 \times 18) = 11.2$ in. From geometry, the length ac = 11.2 (478/384) = 13.94 in., say 14 in. Similarly, the length bc = 11.2 (645/384) = 18.8 in. The center of the nodal zone is located at 14/2 = 7 in. from the bottom of the beam, which is close to 6.0 in., assumed earlier.

   b. Design of nodal zone at D (Fig. 8.27b): The length of the horizontal face de = 768/(0.75 × 2.55 × 18) = 22.3 in. The length of $df = ef$ = 22.3(645/768) = 18.7 in. The length of fg = 15.0 in., and the center of the nodal zone is located at 15/3 = 5.0 in from the top, which is close to the assumed 6.0 in.

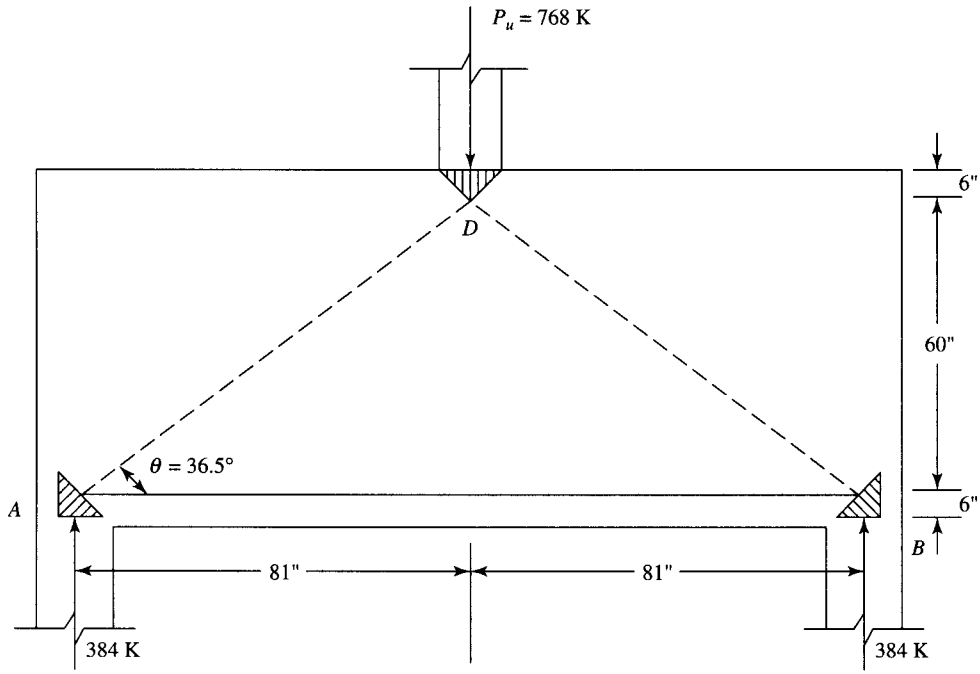


Figure 8.26 Example 8.6: idealized beam.

8. Design of vertical and horizontal reinforcement:
   a. Vertical bars: The angle between the vertical bars and strut = 53.5°, from Fig. 8.27*a*. Use No. 5 bars spaced at 12 in., two branches, $A_s = 2\ (0.31) = 0.62$ in$_2$. Sin 53.5° = 0.804.

$$(A_{si}/b_s s) \sin \gamma_i = (0.62/18 \times 12)(0.804) = 0.0023$$

   b. Horizontal bars: The angle between the horizontal bars and strut = 36.5°, (Fig. 8.27*a*). Use No. 5 bars spaced at 12 in., two branches, $A_s = 0.62$ in.^{2}. Sin 36.5° = 0.595.

$$(A_{si}/b_s s) \sin \gamma_i = (0.62/18 \times 12)(0.595) = 0.0017$$

   c. Total $(A_{si}/b_s s) \sin \gamma_i = 0.0023 + 0.0017 = 0.004 > 0.003$, which is o.k.
9. Design of the horizontal tie AB :
   a. Calculate A_s:

$$F_u = \phi A_s f_y \quad A_s = 478/(0.75 \times 60) = 10.6 \text{ in.}^2$$

   Use 12 no. 9 bars, $A_s = 12$ in.^{2} in three rows as shown in Fig. 8.27*c*.
   b. Calculate anchorage length: Anchorage length is measured from the point beyond the extended nodal zone, Fig. 8.28. Tan 36.5 = 7/x. Then x = 9.5 in. Available anchorage length = 9.5 + 5.6 + 9 − 1.5 in. (cover) = 22.6 in. Development length of no. 9 bars required = 47.5 in. (Table 7.1), which is greater than 22.6 in. Use a standard 90° hook enclosed within the column reinforcement.

$$l_{dh} = 0.02\psi_e f_y d_b/\lambda\sqrt{f'_c} \tag{7.15}$$

$$\psi_e = \lambda = 1.0 \qquad d_b = 1.128 \text{ in.}$$

$$l_{dh} = 0.02(1)(60{,}000)(1.128)/\left((1)\left(\sqrt{4000}\right)\right) = 21.4 \text{ in} < 22.6 \text{ in.}$$

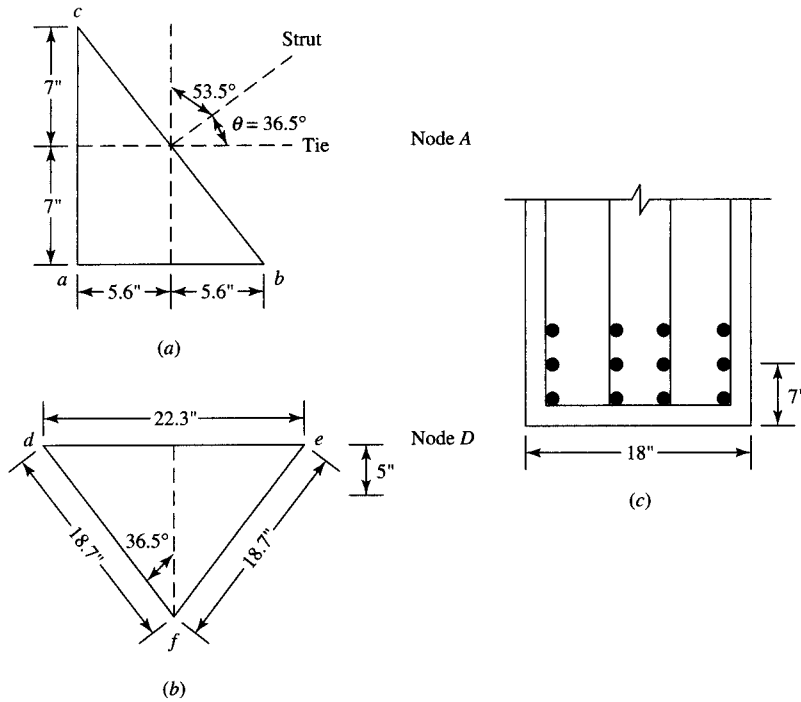


Figure 8.27 Example 8.6: nodal zones, (*a*) at node *A*, (*b*) at node *D*, and (*c*) reinforcement details.

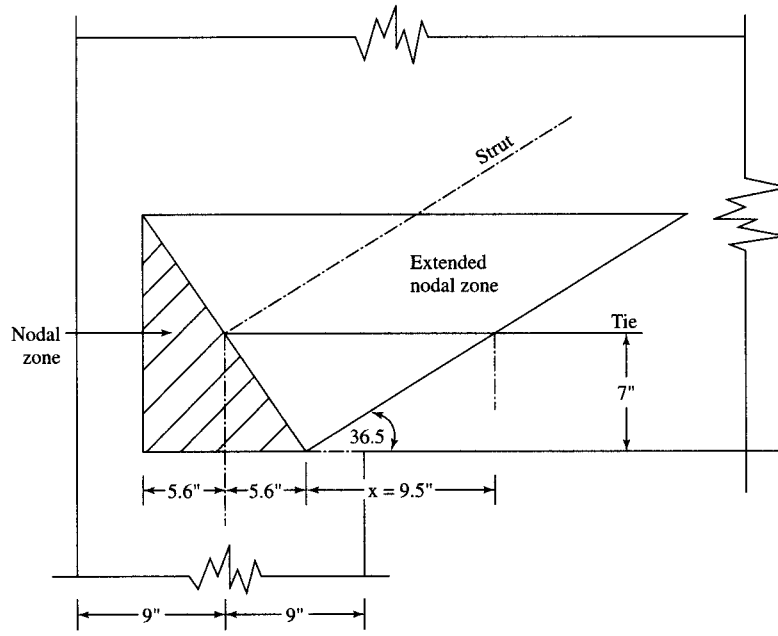


Figure 8.28 Example 8.6: development of tie reinforcement.

Table 8.4 Shear Reinforcement Formulas

U.S. Customary Units	SI Units
V_u = design shear	V_u = design shear
(Maximum design V_u is at a distance d from the face of the support.)	
$V_c = \left(2.0\lambda\sqrt{f'_c}\right) b_w d$	$V_c = \left(0.17\lambda\sqrt{f'_c}\right) b_w d$
$V_c = \left[1.9\lambda\sqrt{f'_c} + \left(2500\rho_w \frac{V_u d}{M_u}\right)\right] b_w d$	$V_c = \left[0.16\lambda\sqrt{f'_c} + \left(17.2\rho_w \frac{V_u d}{M_u}\right)\right] b_w d$
$\rho_w = \frac{A_s}{b_w d} \quad \frac{V_u d}{M_u} \le 1.0$	$\rho_w = \frac{A_s}{b_w d} \quad \frac{V_u d}{M_u} \le 1.0$
$V_c \le \left(3.5\lambda\sqrt{f'_c}\right) b_w d$	$V_c \le \left(0.29\lambda\sqrt{f'_c}\right) b_w d$
$V_u = \phi V_c + \phi V_s$	$V_u = \phi V_c + \phi V_s$
Vertical stirrups	
$\phi V_s = V_u - \phi V_c$	$\phi V_s = V_u - \phi V_c$
$S = \frac{A_v f_{yt} d}{V_s}$	$S = \frac{A_v f_{yt} d}{V_s}$
Minimum $A_v = \frac{50 b_w S}{f_{yt}} \le 0.75\sqrt{f'_c}\left(\frac{b_w S}{f_{yt}}\right)$	Minimum $A_v = \frac{0.35 b_w S}{f_{yt}} \le 0.0062\sqrt{f'_c}\left(\frac{b_w S}{f_y}\right)$
Maximum $S = \frac{A_v f_{yt}}{50 b_w} \ge \frac{A_v f_{yt}}{0.75\sqrt{f'_c} b_w}$	Maximum $S = \frac{A_v f_{yt}}{0.35 b_w} \ge \frac{A_v f_y}{0.062\sqrt{f'_c} b_w}$
For vertical web reinforcement	
Maximum $S = \frac{d}{2} \le 24$in.	Maximum $S = \frac{d}{2} \le 600$ mm
if $V_s \le 4.0\sqrt{f'_c}(b_w d)$	if $V_s \le 0.33\sqrt{f'_c}(b_w d)$
Maximum $S = d/4 = 12$ in.	Maximum $S = d/4 = 300$ mm
if $V_s > 4.0\sqrt{f'_c}(b_w d)$	if $V_s > 0.33\sqrt{f'_c}(b_w d)$
$V_s \le 8\sqrt{f'_c}(b_w d)$	$V_s \le 0.67\sqrt{f'_c}(b_w d)$
Otherwise increase the dimensions of the section.	
Series of bent bars or inclined stirrups	
$A_v = \frac{V_s S}{f_{yt}\, d(\sin\alpha + \cos\alpha)}$	$A_v = \frac{V_s S}{f_{yt}\, d(\sin\alpha + \cos\alpha)}$
For $\alpha = 45°$, $S = \frac{1.4 A_v f_y d}{V_s}$	For $\alpha = 45°$, $S = \frac{1.4 A_v f_y d}{V_s}$
For a single bent bar or group of bars, parallel and bent in one position	
$A_v = \frac{V_s}{f_{yt} \sin\alpha}$	$A_v = \frac{V_s}{f_{yt} \sin\alpha}$
For $\alpha = 45°$, $A_y = \frac{1.4 V_s}{f_{yt}}$	For $\alpha = 45°$, $A_v = \frac{1.4 V_s}{f_{yt}}$
$V_s \le \left(3\sqrt{f'_c}\right) b_w d$	$V_s \le \left(0.25\sqrt{f'_c}\right) b_w d$

8.12 EXAMPLES USING SI UNITS

The general design requirements for shear reinforcement according to the ACI Code are summarized in Table 8.4, which gives the necessary design equations in both U.S. customary and SI units. The following example shows the design of shear reinforcement using SI units.

Example 8.7

A 6-m clear span simply supported beam carries a uniform dead load of 47.5 kN/m and a live load of 25 kN/m (Fig. 8.29). The dimensions of the beam section are $b = 350$ mm, $d = 550$ mm. The beam is reinforced with four bars of 25-mm diameter in one row. It is required to design the necessary shear reinforcement. Given: $f'_c = 28$ MPa and $f_y = 280$ MPa.

Solution

1. Factored load is

$$1.2D + 1.6L = 1.2 \times 47.5 + 1.6 \times 25 = 97 \text{ kN/m}$$

2. Factored shear force at the face of the support is

$$V_u = 97 \times \frac{6}{2} = 291 \text{ kN}$$

3. Maximum design shear at a distance d from the face of the support is

$$V_u \text{ (at distance } d) = 291 - 0.55 \times 97 = 237.65 \text{ kN}$$

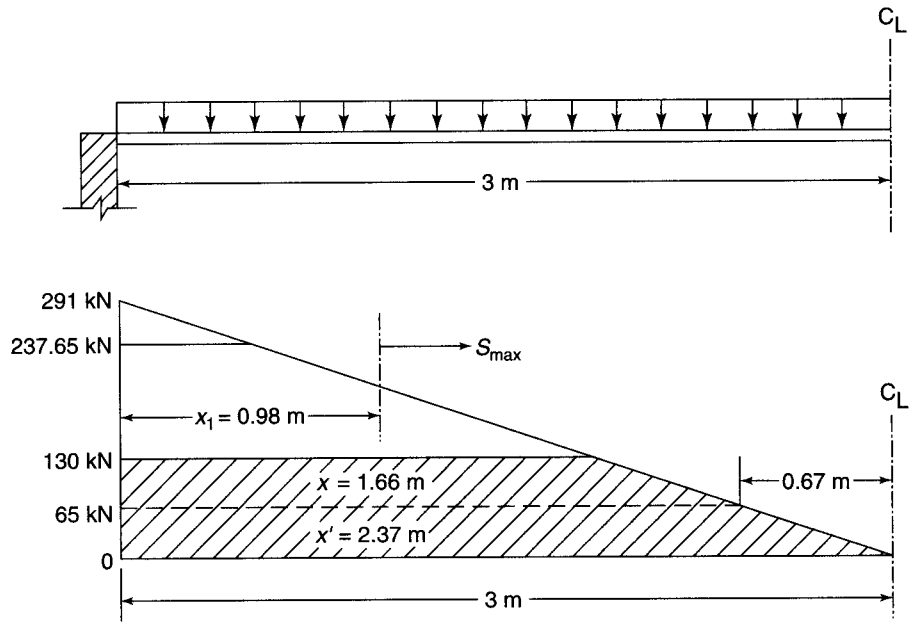


Figure 8.29 Example 8.7.

4. The nominal shear strength provided by concrete is

$$V_c = (0.17\lambda\sqrt{f'_c})bd = (0.17\sqrt{28}) \times 350 \times 550 = 173.2 \text{ kN}$$

$$V_u = \phi V_c + \phi V_s$$

$$\phi V_c = 0.75 \times 173.2 = 130 \text{ kN}$$

$$\frac{1}{2}\phi V_c = 65 \text{ kN}$$

$$\phi V_s = 237.65 - 130 = 107.65 \text{ kN}$$

$$V_s = \frac{107.65}{0.75} = 143.5 \text{ kN}$$

5. Distance from the face of the support at which $\frac{1}{2}\phi V_c = 65$ kN is

$$x' = \frac{(291 - 65)}{291}(3) = 2.33 \text{ m} \quad \text{(from triangles)}$$

6. Design of stirrups:

   a. Choose stirrups 10 mm in diameter with two branches ($A_s = 78.5\,\text{mm}^2$).

$$A_v = 2 \times 78.5 = 157\text{mm}^2$$

$$\text{Spacing } S_1 = \frac{A_y f_{yt} d}{V_s} = \frac{157 \times 280 \times 550}{143.5 \times 10^3} = 168.5 \text{ mm} < 600 \text{ mm}$$

   Thus, use 160 mm. Check maximum spacing of stirrups:

$$\text{Maximum } S_2 = \frac{d}{2} = \frac{550}{2} = 275 \text{ mm}$$

$$S_3 = \frac{A_v f_{yt}}{0.35b} = \frac{157 \times 280}{0.35 \times 350} = 359 \text{ mm}$$

   $S = S_1 = 160$ mm controls.

   b. Check for maximum spacing of $d/4$:

$$\text{If } V_s \le \left(0.33\sqrt{f'_c}\right)bd, \quad S_{\max} = \frac{d}{2}.$$

$$\text{If } V_s > \left(0.33\sqrt{f'_c}\right)bd, \quad S_{\max} = \frac{d}{4}.$$

$$bd\left(0.33\sqrt{f'_c}\right) = 0.33\sqrt{28} \times 350 \times 550 = 336.1\text{kN}$$

   Actual $V_s = 143.5$ kN < 336.1 kN. Therefore, $S_{\max}$ is limited to $d/2 = 275$ mm.

7. The shear reinforcement, stirrups 10 mm in diameter and spaced at 160 mm, will be needed only for a distance $d = 0.55$ m from the face of the support. Beyond that, the shear stress V_s decreases to 0 at a distance $x = 1.66$ m when $\phi V_c = 130$ kN. It is not practical to provide stirrups at many different spacings. One simplification is to find out the distance from the face of support where maximum spacing can be used, and then only two different spacings may be adopted.

$$\text{Maximum spacing} = \frac{d}{2} = 275 \text{ mm}$$

$$V_s \text{ (for } s_{\max} = 275 \text{ mm)} = \frac{A_v f_{yt} d}{S} = \frac{157 \times 0.280 \times 550}{275} = 87.9 \text{ kN}$$

$$\phi V_s = 87.9 \times 0.75 = 65.94 \text{ kN}$$

The distance from the face of the support where $S_{\max} = 275$ mm can be used (from the triangles):

$$x_1 = \frac{291 - (130 + 65.94)}{291}(3) = 0.98 \text{ m}$$

Then, for 0.98 m from the face of support, use stirrups of 10-mm diameter at 160 mm, and for the rest of the beam, minimum stirrups (with maximum spacings) can be used.

8. Distribution of stirrups:

$$\text{one stirrup at } \frac{S}{2} = \frac{160}{2} = 80 \text{ mm}$$

$$\text{six stirrups at 160 mm} = \underline{960 \text{ mm}}$$

$$\text{Total} = 1040 \text{ mm} = 1.04 \text{ m} > 0.98 \text{ m}$$

$$\text{six stirrups at 270 mm} = \underline{1620 \text{ mm}}$$

$$\text{Total} = 2660 \text{ mm} = 2.66 \text{ m} < 3 \text{ m}$$

The last stirrup is $(3 - 2.66) = 0.34$ m $= 340$ mm from the centerline of the beam, which is adequate. A similar stirrup distribution applies to the other half of the beam, giving a total number of stirrups of 28.

The other examples in this chapter can be worked out in a similar way using SI equations.

SUMMARY

Sections 8.1–8.2

The shear stress in a homogeneous beam is $v = VQ/Ib$. The distribution of the shear stress above the neutral axis in a singly reinforced concrete beam is parabolic. Below the neutral axis, the maximum shear stress is maintained down to the level of the steel bars.

Section 8.3

The development of shear resistance in reinforced concrete members occurs by

- Shear resistance of the uncracked concrete
- Interface shear transfer

- Arch action
- Dowel action

Section 8.4

The shear stress at which a diagonal crack is expected is

$$v_c = \frac{V}{bd} = \left(1.9\lambda\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u}\right) \leq 3.5\sqrt{f'_c}$$

The nominal shear strength is

$$V_c = v_c b_w d = 2\lambda\sqrt{f'_c} b_w d$$

Sections 8.5–8.6

1. The common types of shear reinforcement are stirrups (perpendicular or inclined to the main bars), bent bars, or combinations of stirrups and bent bars.

$$V_u = \phi V_n = \phi V_c + \phi V_s \quad \text{and} \quad V_s = \frac{1}{\phi}(V_u - \phi V_c)$$

2. The ACI Code design requirements are summarized in Table 8.4.

Sections 8.7–8.8

Design of vertical stirrups and shear summary is given in these sections.

Sections 8.9–8.10

1. Variation of shear force along the span due to live load may be considered.
2. For members with variable depth,

$$\phi V_n = V_u \pm \frac{M_u(\tan\alpha)}{d} \tag{8.29}$$

Section 8.11

For deep beams, the shear capacity, V_c, may be determined from the following expressions:

$$V_c = 2\lambda\sqrt{f'_c} b_w d \tag{8.35}$$

or

$$V_c = \left(3.5 - \frac{2.5M_u}{V_u d}\right)\left(1.9\lambda\sqrt{f'_c} + \frac{2500\rho_w V_u d}{M_u}\right) b_w d \tag{8.36}$$

The critical section for shear design is at $X = 0.15l_n$ for uniform loads and $X = 0.5a$ for concentrated loads.

Also, refer to Section 5.7 in text.

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PROBLEMS

8.1 Design the necessary shear reinforcement (if needed) in the form of U-stirrups (two legs) for the T-section shown in Fig. 8.30. Use $f'_c = 4$ ksi (28 MPa) and $f_y = 60$ ksi (420 MPa).

a. $V_u = 22$ K (98 kN)

b. $V_u = 56$ K (246 kN)

c. $V_u = 69$ K (306 kN)

8.2 Repeat Problem 8.1 for the section shown in Fig. 8.31.

8.3 Design the necessary shear reinforcement (if needed) in the form of U-stirrups (two legs) for the rectangular section shown in Fig. 8.32 using $f'_c = 3$ ksi (21 MPa) and $f_{yt} = 60$ ksi (420 MPa).

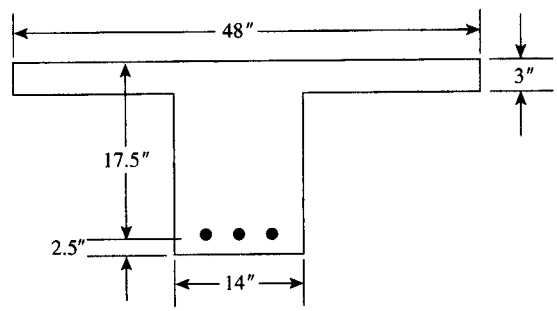


Figure 8.30 Problem 8.1.

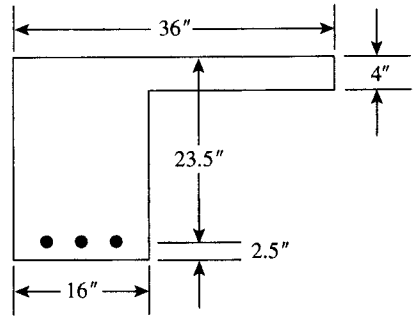


Figure 8.31 Problem 8.2.

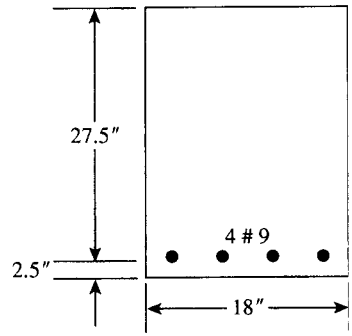


Figure 8.32 Problem 8.3.

a. $V_u = 55$ K (245 kN)
b. $V_u = 110$ K (490 kN)
c. $V_u = 144$ K (640 kN)

8.4 A 16-ft- (4.8-m-)span simply supported beam, Fig. 8.33; has a clear span of 15 ft (4.5 m) and is supported by 12 × 12-in. (300 × 300-mm) columns. The beam carries a factored uniform load of 11.1 K/ft (166 kN/m). The dimensions of the beam section and the flexural steel reinforcement are shown in Fig. 8.33. Design the necessary shear reinforcements using $f'_c = 3$ksi (21 MPa) and $f_{yt} =$ 60 ksi (420 MPa). Show the distribution of stirrups along the beam.

8.5 An 18-ft- (5.4-m-)span simply supported beam carries a uniform dead load of 4 K/ft (60 kN/m) and a live load of 1.5 K/ft (22 kN/m). The beam has a width of $b = 12$ in. (300 mm) and a depth of d $= 24$ in. (600 mm) and is reinforced with six no. 9 bars (6 × 28 mm) in two rows. Check the beam for shear and design the necessary shear reinforcement. Given: $f'_c = 3$ ksi (21 MPa) and $f_{yt} = 50$ ksi (280 MPa).

8.6 Design the necessary shear reinforcement for a 14-ft (4.2-m) simply supported beam that carries a factored uniform load of 10 K/ft (150 kN/m) (including self-weight) and a factored concentrated load at midspan of $P_u = 24$ K (108 kN). The beam has a width of $b = 14$ in. (350 mm) and a depth of d $= 16.5$ (400 mm) and is reinforced with four no. 8 bars (4 × 25 mm). Given: $f'_c = 4$ ksi (28 MPa) and $f_{yt} = 60$ ksi (420 MPa).

8.7 A cantilever beam with 7.4-ft (2.20-m) span carries a uniform dead load of 2.5 K/ft (36 kN/m) (including self-weight) and a concentrated live load of 18 K (80 kN) at a distance of 3 ft (0.9 m) from the face of the support. Design the beam for moment and shear. Given: $f'_c = 3$ ksi (21 MPa), $f_{yt} = 60$ ksi (420 MPa), and $b = 12$ in. (200 mm), and use $\rho = 3/4\rho_{max}$.

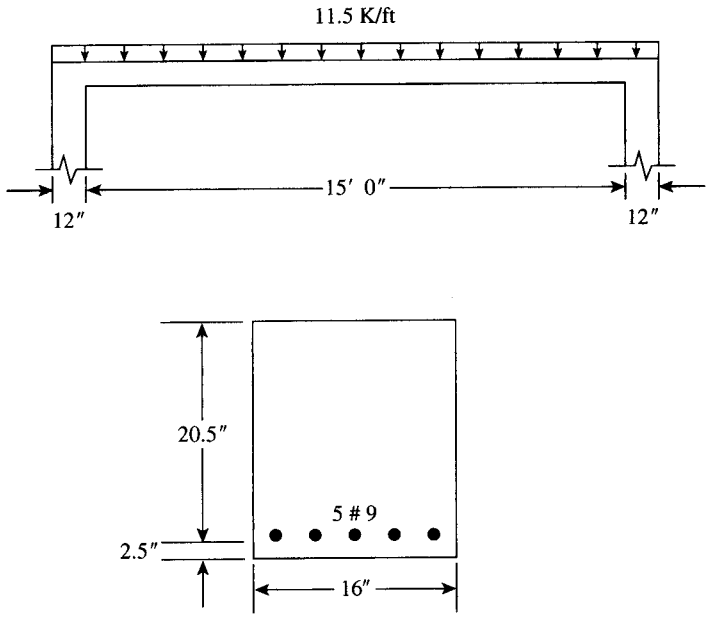


Figure 8.33 Problem 8.4.

8.8 Design the critical sections of an 11-ft-(3.3-m-)span simply supported beam for bending moment and shearing forces using $\rho = 0.016$. Given: $f'_c = 3$ ksi (21 MPa), $f_{yt} = 60$ ksi (420 MPa), and $b = 10$ in. (250 mm). Dead load is 2.75 K/ft (40 kN/m) and live load is 1.375 K/ft (20 kN/m).

8.9 A rectangular beam is to be designed to carry a factored shearing force of 75 kips (335 kN). Determine the minimum beam section if controlled by shear ($V_c = 2\lambda\sqrt{f'_c}bd$) using the minimum shear reinforcement as specified by the ACI Code and no. 3 stirrups. Given: $f'_c = 4$ ksi (28 MPa), $f_{yt} = 40$ ksi (280 MPa), and $b = 16$ in. (400 mm).

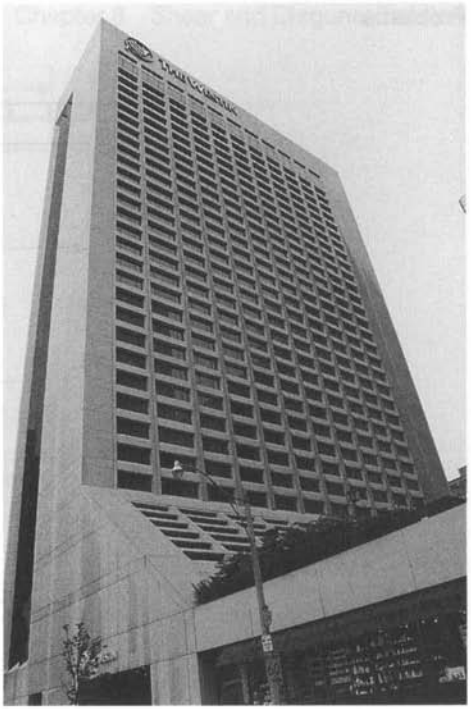
8.10 Redesign Problem 8.5 using $f_{yt} = 60$ ksi.

8.11 Redesign the shear reinforcement of the beam in Problem 8.6 if the uniform factored load of 6 K/ft (90 kN/m) is due to dead load and the concentrated load $P_u = 24$ k (108 kN) is due to a moving live load. Change the position of the live load to cause maximum shear at the support and at midspan.

8.12 Design a cantilever beam that has a span of 9 ft (2.7 m) to carry a factored triangular load that varies from 0 load at the free end to maximum load of 8 K/ft (120 kN/m) at the face of the support. The beam shall have a variable depth, with minimum depth at the free end of 10 in. (250 mm) and increasing linearly toward the support. Use steel percentage $\rho = 0.016$ for flexural design. Given: $f'_c = 4$ ksi (28 MPa), $f_{yt} = 60$ ksi (420 MPa) or flexural reinforcement, $f_{yt} = 40$ ksi (280 MPa) for stirrups, and $b = 11$ in. (275 mm).

CHAPTER 9

ONE-WAY SLABS



The Westin Hotel, Toronto, Canada.

9.1 TYPES OF SLABS

Structural concrete slabs are constructed to provide flat surfaces, usually horizontal, in building floors, roofs, bridges, and other types of structures. The slab may be supported by walls, by reinforced concrete beams usually cast monolithically with the slab, by structural steel beams, by columns, or by the ground. The depth of a slab is usually very small compared to its span. See Fig. 9.1.

Structural concrete slabs in buildings may be classified as follows:

1. *One-way slabs:* If a slab is supported on two opposite sides only, it will bend or deflect in a direction perpendicular to the supported edges. The structural action is one way, and the loads are carried by the slab in the deflected short direction. This type of slab is called a *one-way slab* (Fig. 9.1*a*). If the slab is supported on four sides and the ratio of the long side to the short side is equal to or greater than 2, most of the load (about 95% or more) is carried in the short direction, and one-way action is considered for all practical purposes (Fig. 9.1*b*). If the slab is made of reinforced concrete with no voids, then it is called a *one-way solid slab*. Fig. 9.1*c*, *d*, and *e* shows cross-sections and bar distribution.
2. *One-way joist floor system:* This type of slab is also called a *ribbed slab*. It consists of a floor slab, usually 2 to 4 in. (50 to 100 mm) thick, supported by reinforced concrete ribs (or joists). The ribs are usually tapered and are uniformly spaced at distances that do not exceed 30 in. (750 mm). The ribs are supported on girders that rest on columns. The spaces between the ribs may be formed using removable steel or fiberglass form fillers (pans), which may be used many times (Fig. 9.2). In some ribbed slabs, the spaces between ribs may be filled with permanent fillers to provide a horizontal slab.

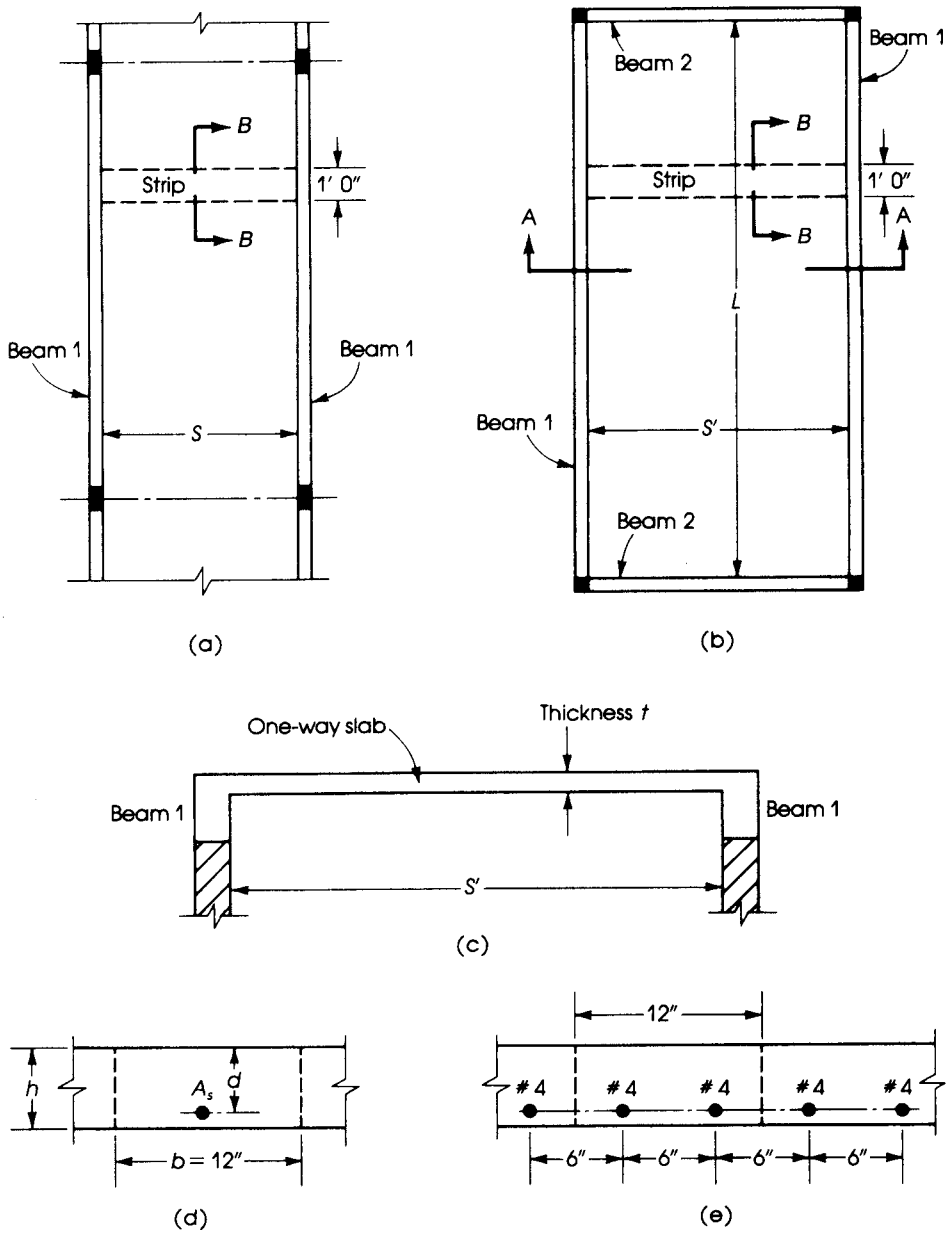


Figure 9.1 One-way slabs.

3. *Two-way floor systems:* When the slab is supported on four sides and the ratio of the long side to the short side is less than 2, the slab will deflect in double curvature in both directions. The floor load is carried in two directions to the four beams surrounding the slab (refer to Chapter 17). Other types of *two-way floor systems* are flat plate floors, flat slabs, and waffle slabs, all explained in Chapter 17. This chapter deals only with one-way floor systems.

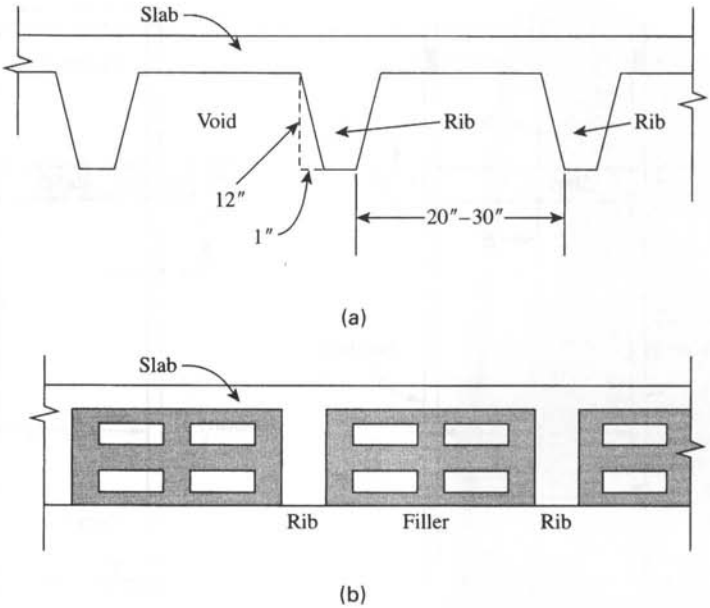


Figure 9.2 Cross-sections of one-way ribbed slab: (*a*) without fillers and (*b*) with fillers.

9.2 DESIGN OF ONE-WAY SOLID SLABS

If the concrete slab is cast in one uniform thickness without any type of voids, it can be referred to as a *solid slab*. In a one-way slab, the ratio of the length of the slab to its width is greater than 2. Nearly all the loading is transferred in the short direction, and the slab may be treated as a beam. A unit strip of slab, usually 1 ft (or 1 m) at right angles to the supporting girders, is considered a rectangular beam. The beam has a unit width with a depth equal to the thickness of the slab and a span length equal to the distance between the supports. A one-way slab thus consists of a series of rectangular beams placed side by side (Fig. 9.1).

If the slab is one span only and rests freely on its supports, the maximum positive moment M for a uniformly distributed load of w psf is $M = (wL^2)/8$, where L is the span length between the supports. If the same slab is built monolithically with the supporting beams or is continuous over several supports, the positive and negative moments are calculated either by structural analysis or by moment coefficients as for continuous beams. The ACI Code, Section 8.3, permits the use of moment and shear coefficients in the case of two or more approximately equal spans (Fig. 9.3). This condition is met when the larger of two adjacent spans does not exceed the shorter span by more than 20%. For uniformly distributed loads, the unit live load shall not exceed three times the unit dead load. When these conditions are not satisfied, structural analysis is required. In structural analysis, the negative bending moments at the centers of the supports are calculated. The value that may be considered in the design is the negative moment at the face of the support. To obtain this value, subtract from the maximum moment value at the center of the support a quantity equal to $Vb/3$, where V is the shearing force calculated from the analysis and b is the width of the support:

$$M_f \text{ (at face of the support)} = M_c \text{ (at centerline of support)} - \frac{Vb}{3} \tag{9.1}$$

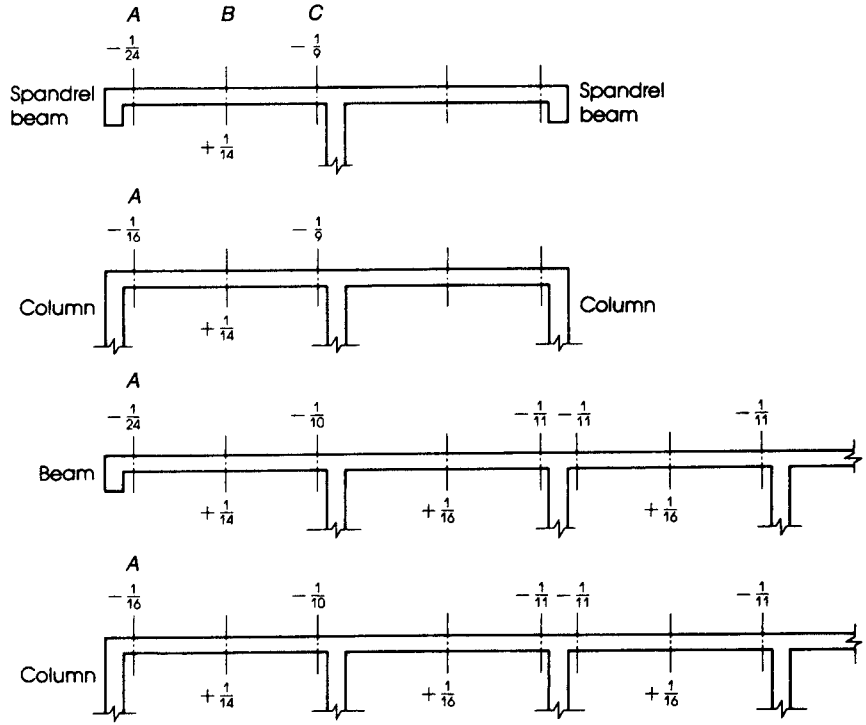


Figure 9.3 Moment coefficients for continuous beams and slabs (ACI Code, Section 8.3).

In addition to moment, diagonal tension and development length of bars should also be checked for proper design.

The conditions under which the moment coefficients for continuous beams and slabs given in Fig. 9.3 should be used can be summarized as follows:

1. Spans are approximately equal: Longer span $\leq$ 1.2 (shorter span).
2. Loads are uniformly distributed.
3. The ratio (live load/dead load) is less than or equal to 3.
4. For slabs with spans less than or equal to 10 ft, negative bending moment at face of all supports is $\left(\frac{1}{12}\right) w_u l_n^2$.
5. For an unrestrained discontinuous end at A, the coefficient is 0 at A and $+\frac{1}{11}$ at B.
6. Shearing force at C is $1.15 w_u l_n/2$ and at the face of all other support is $\frac{1}{2} w_u l_n$.
7. M_u = (coefficient) $(w_u l_n^2)$ and l_n = clear span.

9.3 DESIGN LIMITATIONS ACCORDING TO THE ACI CODE

The following limitations are specified by the ACI Code.

1. A typical imaginary strip 1 ft (or 1 m) wide is assumed.

2. The minimum thickness of one-way slabs using grade 60 steel according to the ACI Code, Table 9.5*a*, for solid slabs and for beams or ribbed one-way slabs should be equal to the following:
   - For simply supported spans: solid slabs, $h = L/20$ (ribbed slabs, $h = L/16$).
   - For one-end continuous spans: solid slabs, $h = L/24$ (ribbed slabs, $h = L/18.5$).
   - For both-end continuous spans: solid slabs, $h = L/28$ (ribbed slabs, $h = L/21$).
   - For cantilever spans: solid slabs, $h = L/10$ (ribbed slabs, $h = L/8$).
   - For f_y other than 60 ksi, these values shall be multiplied by $0.4 + 0.01 f_y$, where f_y is in ksi. This minimum thickness should be used unless computation of deflection indicates a lesser thickness can be used without adverse effects.
3. Deflection is to be checked when the slab supports are attached to construction likely to be damaged by large deflections. Deflection limits are set by the ACI Code, Table 9.5*b*.
4. It is preferable to choose slab depth to the nearest $\frac{1}{2}$ in. (or 10 mm).
5. Shear should be checked, although it does not usually control.
6. Concrete cover in slabs shall not be less than $\frac{3}{4}$ in. (20 mm) at surfaces not exposed to weather or ground. In this case, $d = h - (\frac{3}{4}$ in.$)$ − (half-bar diameter). Refer to Fig. 9.1*d*.
7. In structural slabs of uniform thickness, the minimum amount of reinforcement in the direction of the span shall not be less than that required for shrinkage and temperature reinforcement (ACI Code, Section 7.12).
8. The principal reinforcement shall be spaced not farther apart than three times the slab thickness nor more than 18 in. (ACI Code, Section 7.6.5).
9. Straight-bar systems may be used in both tops and bottoms of continuous slabs. An alternative bar system of straight and bent (trussed) bars placed alternately may also be used.
10. In addition to main reinforcement, steel bars at right angles to the main must be provided. This additional steel is called *secondary, distribution, shrinkage*, or *temperature reinforcement*.

9.4 TEMPERATURE AND SHRINKAGE REINFORCEMENT

Concrete shrinks as the cement paste hardens, and a certain amount of shrinkage is usually anticipated. If a slab is left to move freely on its supports, it can contract to accommodate the shrinkage. However, slabs and other members are joined rigidly to other parts of the structure, causing a certain degree of restraint at the ends. This results in tension stresses known as *shrinkage stresses*. A decrease in temperature and shrinkage stresses is likely to cause hairline cracks. Reinforcement is placed in the slab to counteract contraction and distribute the cracks uniformly. As the concrete shrinks, the steel bars are subjected to compression.

Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement should be provided in a structural slab in which the principal reinforcement extends in one direction only. The ACI Code, Section 7.12.2, specifies the following minimum steel ratios: For slabs in which grade 40 or 50 deformed bars are used, $\rho = 0.2\%$, and for slabs in which grade 60 deformed bars or welded bars or welded wire fabric are used, $\rho = 0.18\%$. In no case shall such reinforcement be placed farther apart than five times the slab thickness or more than 18 in.

For temperature and shrinkage reinforcement, the whole concrete depth h exposed to shrinkage shall be used to calculate the steel area. For example, if a slab has a total depth of $h = 6$ in.

and $f_y = 60$ ksi, then the area of steel required per 1-ft width of slab is $A_s = 6(12)(0.0018) = 0.129\,\text{in.}^2$. The spacings of the bars, S, can be determined as follows:

$$S = \frac{12A_b}{A_s} \tag{9.2}$$

where A_b = area of the bar chosen and A_s = calculated area of steel.

For example, if no. 3 bars are used ($A_b = 0.11\,\text{in.}^2$), then $S = 12(0.11)/0.129 = 10.33$ in., say, 10 in. If no. 4 bars are chosen ($A_b = 0.2\,\text{in.}^2$), then $S = 12(0.2)/0.129 = 18.6$ in., say, 18 in. Maximum spacing is the smaller of five times slab thickness (30 in.) or 18 in. Then no. 4 bars spaced at 18 in. are adequate (or no. 3 bars at 10 in.). These bars act as secondary reinforcement and are placed normal to the main reinforcement calculated by flexural analysis. Note that areas of bars in slabs are given in Table A.14.

9.5 REINFORCEMENT DETAILS

In continuous one-way slabs, the steel area of the main reinforcement is calculated for all critical sections, at midspans, and at supports. The choice of bar diameter and detailing depends mainly on the steel areas, spacing requirements, and development length. Two bar systems may be adopted.

In the straight-bar system (Fig. 9.4), straight bars are used for top and bottom reinforcement in all spans. The time and cost to produce straight bars is less than that required to produce bent bars; thus, the straight-bar system is widely used in construction.

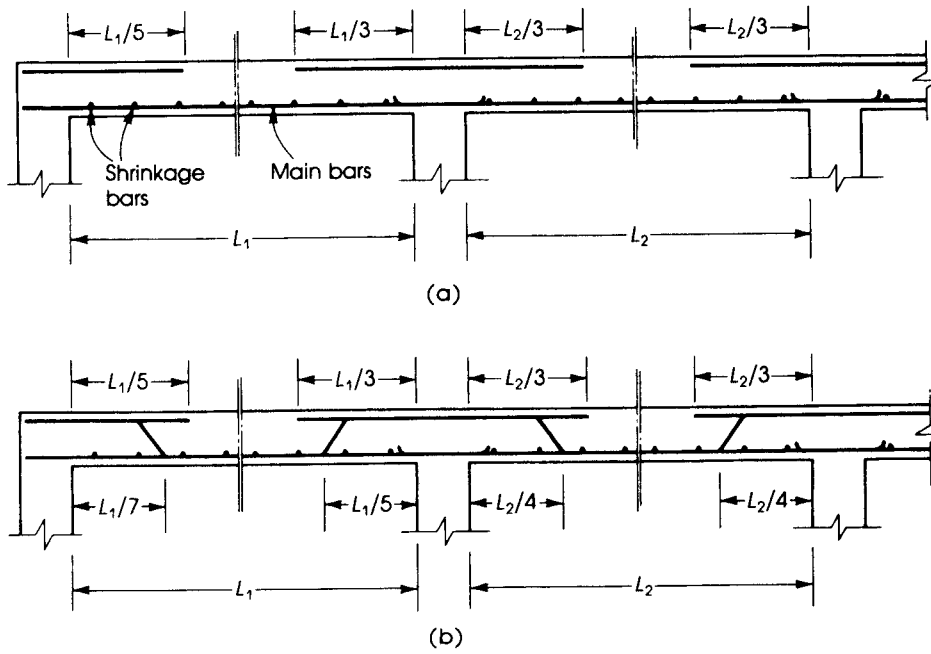


Figure 9.4 Reinforcement details in continuous one-way slabs: (*a*) straight bars and (*b*) bent bars.

In the bent-bar, or trussed, system, straight and bent bars are placed alternately in the floor slab. The location of bent points should be checked for flexural, shear, and development length requirements. For normal loading in buildings, the bar details at the end and interior spans of one-way solid slabs may be adopted as shown in Fig. 9.4.

9.6 DISTRIBUTION OF LOADS FROM ONE-WAY SLABS TO SUPPORTING BEAMS

In one-way floor slab systems, the loads from slabs are transferred to the supporting beams along the long ends of the slabs. The beams transfer their loads in turn to the supporting columns.

From Fig. 9.5 it can be seen that beam B_2 carries loads from two adjacent slabs. Considering a 1-ft length of beam, the load transferred to the beam is equal to the area of a strip 1 ft wide and S feet in length multiplied by the intensity of load on the slab.

This load produces a uniformly distributed load on the beam:

$$U_B = U_S \cdot S$$

The uniform load on the end beam, B_1, is half the load on B_2, because it supports a slab from one side only.

The load on column C_4 is equal to the reactions from two adjacent B_2 beams,

$$\text{Load on column } C_4 = U_B L = U_S L S$$

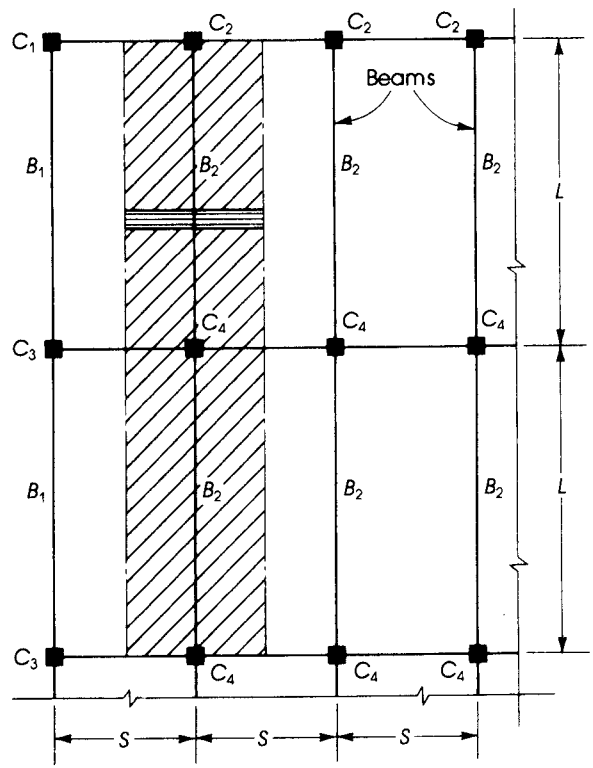


Figure 9.5 Distribution of loads on beams.

The load on column C_3 is one-half the load on column C_4, because it supports loads from slabs on one side only. Similarly, the loads on columns C_2 and C_1 are

$$\text{Load on } C_2 = U_S \frac{L}{2} S = \text{load on } C_3$$

$$\text{Load on } C_1 = U_S \left(\frac{L}{2}\right)\left(\frac{S}{2}\right)$$

From this analysis, it can be seen that each column carries loads from slabs surrounding the column and up to the centerline of adjacent slabs: up to $L/2$ in the long direction and $S/2$ in the short direction.

Distribution of loads from two-way slabs to their supporting beams and columns is discussed in Chapter 17.

Example 9.1

Calculate the design moment strength of a one-way solid slab that has a total depth of $h = 7$ in. and is reinforced with no. 6 bars spaced at $S = 7$ in. Use $f'_c = 3$ ksi and $f_y = 60$ ksi.

Solution

1. Determine the effective depth, d:

$$d = h - \frac{3}{4} \text{ in. (cover)} - \text{half-bar diameter} \quad \text{(See Fig. 9.1}d\text{)}.$$

$$d = 7 - \frac{3}{4} - \frac{6}{16} = 5.875 \text{ in.}$$

2. Determine the average A_s provided per 1-ft width (12 in.) of slab. The area of no. 6 bar is $A_b = 0.44 \text{ in.}^2$.

$$A_s = \frac{12A_b}{S} = \frac{12(0.44)}{7} = 0.754 \text{ in.}^2/\text{ft}$$

Areas of bars in slabs are given in Table A.14 in Appendix A.

3. Compare the steel ratio used with ρ_{max} and ρ_{min}. For $f'_c = 3$ ksi and $f_y = 60$ ksi, $\rho_{max} = 0.01356$ and $\rho_{min} = 0.00333$. ρ (used) $= 0.754/(12 \times 5.875) = 0.0107$, which is adequate ($\phi = 0.9$).

4. Calculate $\phi M_n = \phi A_s f_y (d - a/2)$.

$$a = A_s f_y / (0.85 f'_c b) = 0.754(60)/(0.85 \times 3 \times 12) = 1.48 \text{ in.}$$

$$\phi M_n = 0.9(0.754)(60)(5.875 - 1.48/2) = 209 \text{ K·in.} = 17.42 \text{ K·ft}$$

Example 9.2

Determine the allowable uniform live load that can be applied on the slab of the previous example if the slab span is 16 ft between simple supports and carries a uniform dead load (excluding self-weight) of 100 psf.

Solution

1. The design moment strength of the slab is 17.42 K·ft per 1-ft width of slab.

$$M_u = \phi M_n = 17.42 = \frac{W_u L^2}{8} = \frac{W_u (16)^2}{8}$$

The factored uniform load is $W_u = 0.544 \text{ K/ft}^2 = 544$ psf.

2. $W_u = 1.2D + 1.6L$

$$D = 100 \text{ psf} + \text{self-weight} = 100 + \frac{7}{12}(150) = 187.5 \text{ psf}$$

$$544 = 1.2(187.5) + 1.6L \quad L = 200 \text{ psf}$$

Example 9.3

Design a 12-ft simply supported slab to carry a uniform dead load (excluding self-weight) of 120 psf and a uniform live load of 100 psf. Use $f'_c = 3$ ksi, $f_y = 60$ ksi, $\lambda = 1$, and the ACI Code limitations.

Solution

1. Assume a slab thickness. For $f_y = 60$ ksi, the minimum depth to control deflection is $L/20 = 12(12)/20 = 7$ in. Assume a total depth of $h = 7$ in. and assume $d = 6$ in. (to be checked later).
2. Calculate factored load: weight of slab $= \frac{7}{12}(150) = 87.5$ psf.

$$W_u = 1.2D + 1.6L = 1.2(87.5 + 120) + 1.6(100) = 409 \text{ psf}$$

For a 1-ft width of slab, $M_u = W_u L^2/8$.

$$M_u = \frac{0.409(12)^2}{8} = 7.362 \text{ K·ft}$$

3. Calculate A_s: For $M_u = 7.362$ K·ft, $b = 12$ in., and $d = 6$ in., $R_u = M_u/bd^2 = 7.362(12{,}000)/(12)(6)^2 = 205$ psi. From tables in Appendix A, $\rho = 0.0040 < \rho_{\max} = 0.01356$, $\phi = 0.9$.

$$A_s = \rho bd = 0.0040(12)(6) = 0.28 \text{ in.}^2$$

Choosing no. 4 bars ($A_b = 0.2$ in.^{2}), and $S = 12A_b/A_s = 12(0.2)/0.28 = 8.6$ in. Check actual $d = h - \frac{3}{4} - \frac{4}{16} = 6$ in. It is acceptable. Let $S = 8$ in. and $A_s = 0.3$ in^2.

4. Check the moment capacity of the final section.

$$a = \frac{A_s f_y}{(0.85 f'_c b)} = \frac{0.3(60)}{0.85 \times 3 \times 12} = 0.59 \text{ in.}$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) = 0.9(0.3)(60)(6 - 0.59/2) = 92.42 \text{ K·in.} = 7.7 \text{ K·ft} > M_u$$

$$= 7.362 \text{ K·ft}$$

5. Calculate the secondary (shrinkage) reinforcement normal to the main steel. For $f_y = 60$ ksi,

$$\rho_{\min} = 0.0018$$

$$A_{\text{sh}} = \rho bh = 0.0018(12)(7) = 0.1512 \text{ in.}^2$$

Choose no. 4 bars, $A_b = 0.2$ in.^{2}, $S = 12A_b/A_s = 12(0.2)/0.1512 = 15.9$ in. Use no. 4 bars spaced at 15 in.

6. Check shear requirements: V_u at a distance d from the support is $0.409\left(\frac{12}{2} - \frac{6}{12}\right) = 2.25$ K.

$$\phi V_c = \phi 2\lambda \sqrt{f'_c} bd = \frac{0.75(2)(1)(\sqrt{3000})(12 \times 6)}{1000} = 5.9 \text{ K}$$

$\frac{1}{2}\phi V_c = 2.95 \text{ K} > V_u$, so the shear is adequate.

7. Final section: $h = 7$ in., main bars = no. 4 spaced at 8 in., and secondary bars = no. 4 spaced at 15 in.

Example 9.4

The cross-section of a continuous one-way solid slab in a building is shown in Fig. 9.6. The slabs are supported by beams that span 12 ft between simple supports. The dead load on the slabs is that due to self-weight plus 77 psf; the live load is 130 psf. Design the continuous slab and draw a detailed section. Given: $f'_c = 3$ ksi and $f_y = 40$ ksi.

Solution

1. The minimum thickness of the first slab is $L/30$, because one end is continuous and the second end is discontinuous. The distance between centers of beams may be considered the span L, here equal to 12 ft. For $f_y = 40$ ksi,

$$\text{Minimum total depth} = \frac{L}{30} = \frac{12 \times 12}{30} = 4.8 \text{ in.}$$

$$\text{Minimum total depth for interior span} = \frac{L}{35} = 4.1 \text{ in.}$$

Assume a uniform thickness of 5 in., which is greater than 4.8 in.; therefore, it is not necessary to check deflection.

2. Calculate loads and moments in a unit strip:

$$\text{Dead load} = \text{weight of slab} + 60 \text{ psf}$$

$$= \left(\frac{5}{12} \times 150\right) + 77 = 139.5 \text{ psf}$$

$$\text{Factored load } (U) = 1.2D + 1.6L = 1.2 \times 139.5 + 1.6 \times 130 = 375.5 \text{ psf}$$

The clear span is 11.0 ft. The required moment in the first span is over the support and equals $UL^2/10$.

$$M_u = \frac{U(11)^2}{10} = (0.3755)\frac{121}{10} = 4.54 \text{ K·ft} = 54.5 \text{ K·in.}$$

3. Assume $\rho = 1.4\%$; then $R_u = 450$ psi $= 0.45$ ksi. This value is less than ρ_{max} of 0.0203 (Table 4.1), and greater than ρ_{min} of 0.005 ($\phi = 0.9$).

$$d = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{54.5}{0.45 \times 12}} = 3.18 \text{ in.}$$

$$A_s = \rho bd = 0.014(12)(3.18) = 0.53 \text{ in.}^2$$

Choosing no. 5 bars,

$$\text{Total depth} = d + \tfrac{1}{2} \text{ bar diameter} + \text{cover} = 3.18 + \tfrac{5}{16} + \tfrac{3}{4} = 4.25 \text{ in.}$$

Use slab thickness of 5 in., as assumed earlier.

$$\text{Actual } d \text{ used} = 5 - \tfrac{3}{4} - \tfrac{5}{16} = 3.9 \text{ in.}$$

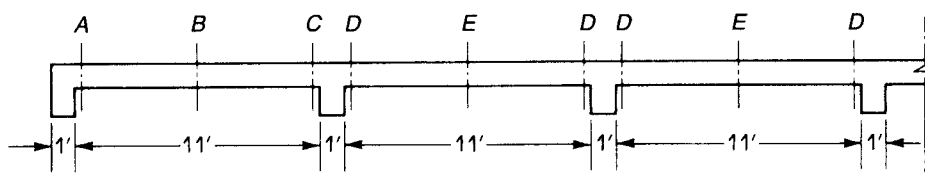


Figure 9.6 Example 9.4.

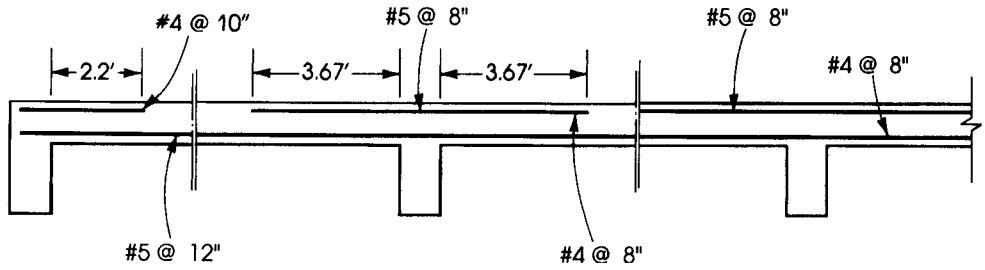


Figure 9.7 Example 9.4: Reinforcement details.

4. Moments and steel reinforcement required at other sections using $d = 3.9$ in. are as follows:

Location	Moment Coefficient	M_u (K·in.)	$R_u = M_u/bd^2$ (psi)	ρ (%)	A_s (in.^{2})	Bars and Spacings
A	$-\frac{1}{24}$	22.7	Small	0.50	0.23	No. 4 at 10 in.
B	$+\frac{1}{14}$	38.9	213	0.65	0.30	No. 5 at 12 in.
C	$-\frac{1}{10}$	54.5	300	0.90	0.44	No. 5 at 8 in.
D	$-\frac{1}{11}$	49.6	271	0.80	0.38	No. 5 at 8 in.
E	$+\frac{1}{16}$	34.1	187	0.55	0.26	No. 4 at 8 in.

The arrangement of bars is shown in Fig. 9.7.

5. Maximum shear occurs at the exterior face of the second support, section C.

$$V_u \text{ (at } C) = 1.15UL_n/2 = \frac{1.15(0.3755)(11)}{2} = 2.375 \text{ K/ft of width}$$

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}bd = \frac{0.75(2)(1)(\sqrt{3000})(12)(3.9)}{1000} = 3.84 \text{ K}$$

This result is acceptable. Note that the provision of minimum area of shear reinforcement when V_u exceeds $\frac{1}{2}\phi V_c$ does not apply to slabs (ACI Code, Section 11.5.5).

Example 9.5

Determine the uniform factored load on an intermediate beam supporting the slabs of Example 9.4. Also calculate the axial load on an interior column; refer to the general plan of Fig. 9.5.

Solution

1. The uniform factored load per foot length on an intermediate beam is equal to the factored uniform load on slab multiplied by S, the short dimension of the slab. Therefore,

$$U \text{ (beam)} = U \text{ (slab)} \times S = 0.3755 \times 12 = 4.5 \text{ K/ft}$$

The weight of the web of the beam shall be added to this value. Span of the beam is 24 ft.

$$\text{Estimated total depth} = \frac{L}{20} \times 0.8 = \left(\frac{24}{20} \times 0.8\right) \times 12 = 11.5 \text{ in.} \quad \text{say, 12 in.}$$

Slab thickness is 5 in. and height of the web is $12 - 5 = 7$ in.

$$\text{Factored weight of beam web} = \left(\frac{7}{12} \times 150\right) \times 1.2 = 105 \text{ lb/ft}$$

$$\text{Total uniform load on beam} = 4.5 + 0.105 = 4.605 \text{ K/ft}$$

2. Axial load on an interior column:

$$P_u = 4.605 \times 24 \text{ ft} = 110.5 \text{ K}$$

9.7 ONE-WAY JOIST FLOOR SYSTEM

A one-way joist floor system consists of hollow slabs with a total depth greater than that of solid slabs. The system is most economical for buildings where superimposed loads are small and spans are relatively large, such as schools, hospitals, and hotels. The concrete in the tension zone is ineffective; therefore, this area is left open between ribs or filled with lightweight material to reduce the self-weight of the slab.

The design procedure and requirements of ribbed slabs follow the same steps as those for rectangular and T-sections explained in Chapter 3. The following points apply to design of one-way ribbed slabs:

1. Ribs are usually tapered and uniformly spaced at about 16 to 30 in. (400 to 750 mm). Voids are usually formed by using pans (molds) 20 in. (500 mm) wide and 6 to 20 in. (150 to 500 mm) deep, depending on the design requirement. The standard increment in depth is 2 in. (50 mm).
2. The ribs shall not be less than 4 in. (100 mm) wide and must have a depth of not more than 3.5 times the width. Clear spacing between ribs shall not exceed 30 in. (750 mm) (ACI Code, Section 8.13).
3. Shear strength, V_c, provided by concrete for the ribs may be taken 10% greater than that for beams. This is mainly due to the interaction between the slab and the closely spaced ribs (ACI Code, Section 8.13.8).
4. The thickness of the slab on top of the ribs is usually 2 to 4 in. (50 to 100 mm) and contains minimum reinforcement (shrinkage reinforcement). This thickness shall not be less than $\frac{1}{12}$ of the clear span between ribs or 1.5 in. (38 mm) (ACI Code, Section 8.13.5.2).
5. The ACI coefficients for calculating moments in continuous slabs can be used for continuous ribbed slab design.
6. There are additional practice limitations, which can be summarized as follows:
   - The minimum width of the rib is one-third of the total depth or 4 in. (100 mm), whichever is greater.
   - Secondary reinforcement in the slab in the transverse directions of ribs should not be less than the shrinkage reinforcement or one-fifth of the area of the main reinforcement in the ribs.
   - Secondary reinforcement parallel to the ribs shall be placed in the slab and spaced at distances not more than half of the spacings between ribs.
   - If the live load on the ribbed slab is less than 3 kN/m^2 (60 psf) and the span of ribs exceeds 5 m (17 ft), a secondary transverse rib should be provided at midspan (its direction

is perpendicular to the direction of main ribs) and reinforced with the same amount of steel as the main ribs. Its top reinforcement shall not be less than half of the main reinforcement in the tension zone. These transverse ribs act as floor stiffeners.

- If the live load exceeds 3 kN/m^2 (60 psf) and the span of ribs varies between 4 and 7 m (13 and 23 ft), one traverse rib must be provided, as indicated before. If the span exceeds 7 m (23 ft), at least two transverse ribs at one-third span must be provided with reinforcement, as explained before.

Example 9.6

Design an interior rib of a concrete joist floor system with the following description: Span of rib = 20 ft (simply supported), dead load (excluding own weight) = 16 psf, live load = 85 psf, $f'_c = 4$ ksi, and $f_y = 60$ ksi.

Solution

1. Design of the slab: Assume a top slab thickness of 2 in. that is fixed to ribs that have a clear spacing of 20 in. No fillers are used. The self-weight of the slab is $\frac{2}{12} \times 150 = 25$ psf.

$$\text{Total D.L.} = 16 + 16 = 41 \text{ psf}$$

$$U = 1.2D + 1.6L = 1.2 \times 41 + 1.6 \times 85 = 185 \text{ psf}$$

$$M_u = \frac{UL^2}{12} \quad \text{(Slab is assumed fixed to ribs.)}$$

$$= \frac{0.185}{12}\left(\frac{20}{12}\right)^2 = 0.043 \text{ K·ft} = 0.514 \text{ K·in.}$$

Considering that the moment in slab will be carried by plain concrete only, the allowable flexural tensile strength is $f_t = 5\sqrt{f'_c}$, with a capacity-reduction factor $\phi = 0.55$, $f_t = 5\sqrt{4000} = 316$ psi.

$$\text{Flexural tensile strength} = \frac{Mc}{I} = \phi f_t \quad I = \frac{bh^3}{12} = \frac{12(2)^3}{12} = 8 \text{ in.}^4 \quad c = \frac{h}{2} = \frac{2}{2} = 1 \text{ in.}$$

$$M = \phi f_t \frac{I}{c} = 0.55 \times 0.316 \times \frac{8}{1} = 1.39 \text{ K·in.}$$

This value is greater than $M_u = 0.514$ K·in., and the slab is adequate. For shrinkage reinforcement, $A_s = 0.0018 \times 12 \times 2 = 0.043$ in.^{2} Use no. 3 bars spaced at 12 in. laid transverse to the direction of the ribs. Welded wire fabric may be economically used for this low amount of steel reinforcement. Use similar shrinkage reinforcement no. 3 bars spaced at 12 in. laid parallel to the direction of ribs, one bar on top of each rib and one bar in the slab between ribs.

2. Calculate moment in a typical rib:

$$\text{Minimum depth} = \frac{L}{20} = \frac{20 \times 12}{20} = 12 \text{ in.}$$

The total depth of rib and slab is 10 + 2 = 12 in. Assume a rib width of 4 in. at the lower end that tapers to 6 in. at the level of the slab (Fig. 9.8). The average width is 5 in. Note that the increase in the rib width using removable forms has a ratio of about 1 horizontal to 12 vertical.

$$\text{Weight of rib} = \frac{5}{12} \times \frac{10}{12} \times 150 = 52 \text{ lb/ft}$$

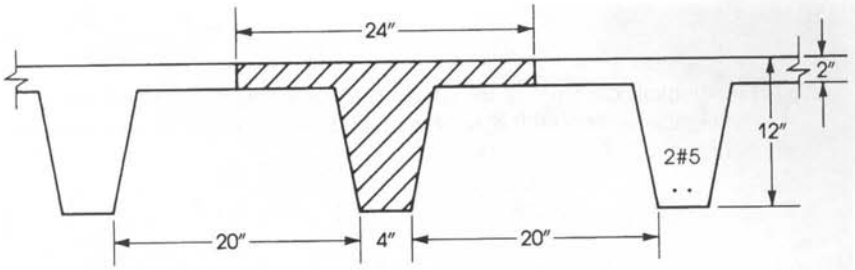
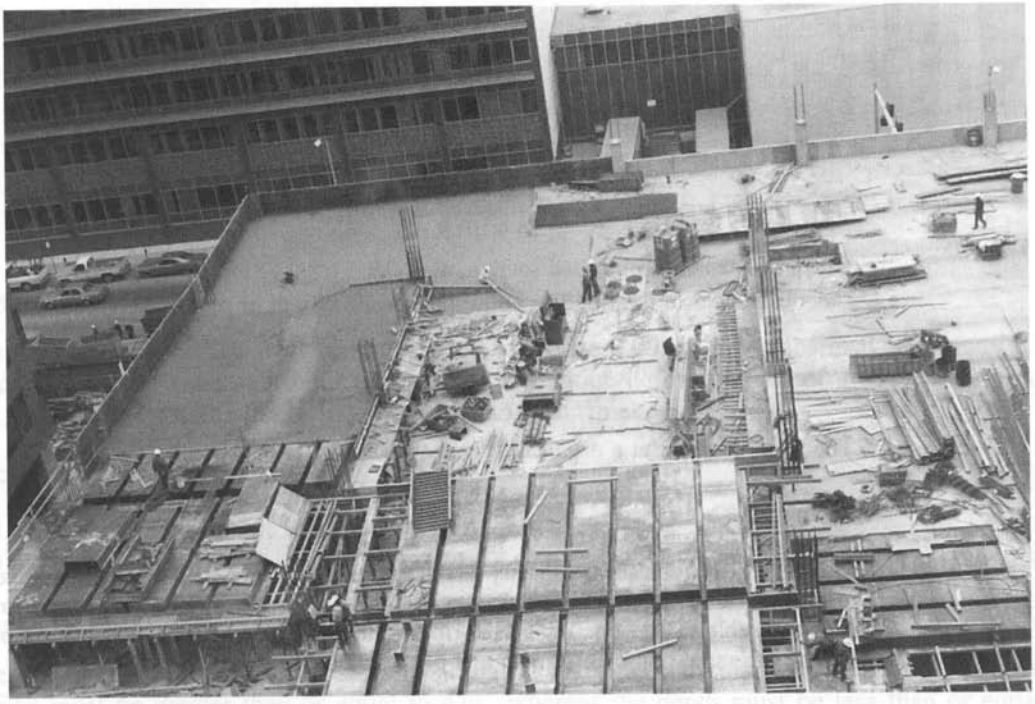


Figure 9.8 Example 9.6.



Rectangular steel pans used in one-way ribbed slab construction.

The rib carries a load from (20 + 4)-in.-wide slab plus its own weight:

$$U = \frac{24}{12} \times 185 + (1.2 \times 52) = 432.4 \text{ lb/ft}$$

$$M_u = \frac{UL^2}{8} = \frac{0.4324}{8}(20)^2 \times 12 = 259.4 \text{ K·in.}$$

3. Design of rib: The total depth is 12 in. Assuming no. 5 bars and concrete cover of $\frac{3}{4}$ in., the effective depth d is $12 - \frac{3}{4} - \frac{5}{16} = 10.9$ in. Check the moment capacity of the flange (assume tension-controlled section, $\phi = 0.9$):

$$\phi M_n \text{ (flange)} = \phi C \left(d - \frac{t}{2}\right), \text{ where } C = 0.85 f_c' bt$$

$$M_u = 0.9(0.85 \times 4 \times 24 \times 2)\left(10.9 - \frac{2}{2}\right) = 1454 \text{ K·in.}$$

The moment capacity of the flange is greater than the applied moment; thus, the rib acts as a rectangular section with $b = 24$ in., and the depth of the equivalent compressive block a is less than 2 in.

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) = \phi A_s f_y \left(d - \frac{A_s f_y}{1.7 f_c' b}\right)$$

$$259.4 = 0.9A_s \times 60\left(10.9 - \frac{A_s \times 60}{1.7 \times 4 \times 24}\right) \quad A_s = 0.45 \text{ in.}^2$$

$$a = \frac{A_s f_y}{0.85 \times f_c' b} = 0.33 \text{ in.} < 2 \text{ in.}$$

Use two no. 5 bars per rib ($A_s = 0.65 \text{ in.}^2$).

$$A_{s\,\min} = 0.0033 b_w d = 0.0033(5)(10.9) = 0.18 \text{ in.}^2 < 0.45 \text{ in.}^2$$

Check

$$\rho = \frac{0.45}{24 \times 10.9} = 0.00172 < \rho_{\max} = 0.01806$$

which is a tension-controlled section, $\phi = 0.9$.

4. Calculate shear in the rib: The allowable shear strength of the rib web is

$$\phi V_c = \phi(1.1) \times 2\lambda\sqrt{f_c'} b_w d$$

$$= 0.75 \times 1.1 \times 2(1)\sqrt{4000} \times 5 \times 10.9 = 5687 \text{ lb}$$

The factored shear at a distance d from the support is

$$V_u = 432.4\left(10 - \frac{10.9}{12}\right) = 3931 \text{ lb}$$

This is less than the shear capacity of the rib. Minimum stirrups may be used, and in this case an additional no. 4 bar will be placed within the slab above the rib to hold the stirrups in place. It is advisable to add one transverse rib at midspan perpendicular to the direction of the ribs having the same reinforcement as that of the main ribs to act as a stiffener.

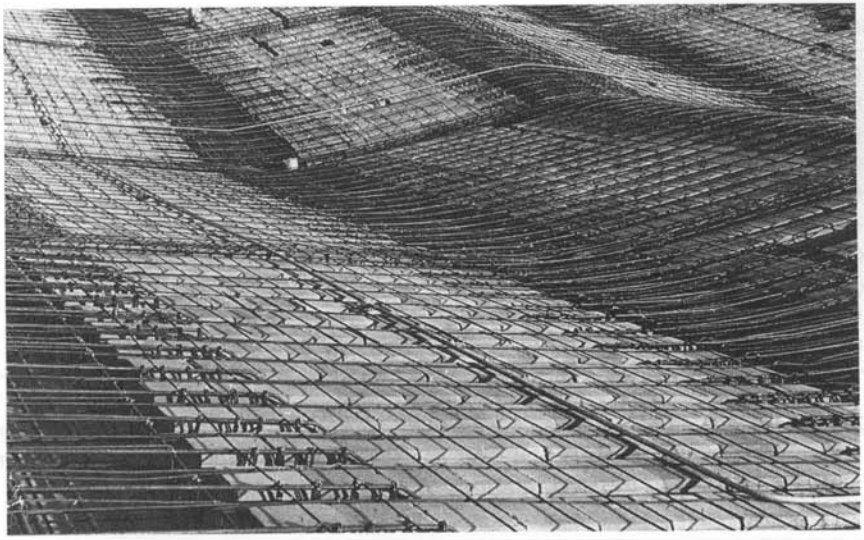
SUMMARY

Section 9.1

Slabs are of different types, one way (solid or joist floor systems) and two way (solid, ribbed, waffle, flat slabs, and flat plates).

Sections 9.2–9.3

1. The ACI Code moment and shear coefficients for continuous one-way slabs are given in Fig. 9.3.
2. The minimum thickness of one-way slabs using grade 60 steel is $L/20$, $L/24$, $L/28$, and $L/10$ for simply supported, one-end continuous, both-end continuous, and cantilever slabs, respectively.



One-way ribbed slab roof. The wide beams have the same total depth as the ribbed slab.

Section 9.4

The minimum shrinkage steel ratios, ρ_{min}, in slabs are 0.002 in. for slabs in which grade 40 or grade 50 bars are used and 0.0018 in. for slabs in which deformed bars of grade 60 are used.

Maximum spacings between bars $\leq$ 5 times rib thickness $\leq$ 18 in.

Sections 9.5–9.6

1. Reinforcement details are shown in Fig. 9.4.
2. Distribution of loads from one-way slabs to the supporting beams is shown in Fig. 9.5.

Section 9.7

The design procedure of ribbed slabs is similar to that of rectangular and T-sections. The width of ribs must be greater than or equal to 4 in., whereas the depth must be less than or equal to 3.5 times the width. The minimum thickness of the top slab is 2 in. or not less than one-twelfth of the clear span between ribs.

REFERENCES

1. Concrete Reinforcing Steel Institute. *CRSI Design Handbook*. Chicago, 2002.
2. Portland Cement Association. *Continuity in Concrete Building Frames*. Chicago, 1959.
3. American Concrete Institute. ACI Code 318-08, Building Code Requirements for Structural Concrete. Detroit, Michigan, 2008.

PROBLEMS

9.1 For each problem, calculate the factored moment capacity of each concrete slab section using $f_y = 60$ ksi.

Number	f'_c(ksi)	h (in.)	Bars and Spacings (in.)	Answer ϕM_n (K·ft)
(a)	3	5	No. 4 at 6	6.35
(b)	3	6	No. 5 at 8	9.29
(c)	3	7	No. 6 at 9	14.06
(d)	3	8	No. 8 at 12	21.01
(e)	4	$5\frac{1}{2}$	No. 5 at 10	6.93
(f)	4	6	No. 7 at 12	11.80
(g)	4	$7\frac{1}{2}$	No. 6 at 6	22.68
(h)	4	8	No. 8 at 12	21.23
(i)	5	5	No. 5 at 10	6.19
(j)	5	6	No. 5 at 8	9.66

9.2 For each slab problem, determine the required steel reinforcement, A_s, and the total depth, if required; then choose adequate bars and their spacings. Use $f_y = 60$ ksi for all problems, $b = 12$ in., and a steel ratio close to the steel ratio $\rho = A_s/bd$ given in some problems.

					One Answer	
Number	f'_c(ksi)	M_u (K·ft)	h (in.)	ρ (%)	h (in.)	Bars
(a)	3	5.4	6	—	6	No. 4 at 9 in.
(b)	3	13.8	$7\frac{1}{2}$	—	$7\frac{1}{2}$	No. 6 at 10 in.
(c)	3	24.4	—	0.85	9	No. 8 at 12 in.
(d)	3	8.1	5	—	5	No. 5 at 7 in.
(e)	4	22.6	—	1.18	$7\frac{1}{2}$	No. 7 at 8 in.
(f)	4	13.9	$8\frac{1}{2}$	—	$8\frac{1}{2}$	No. 6 at 12 in.
(g)	4	13.0	—	1.10	6	No. 6 at 8 in.
(h)	4	11.2	—	0.51	$7\frac{1}{2}$	No. 5 at 9 in.
(i)	5	20.0	9	—	9	No. 7 at 12 in.
(j)	5	10.6	—	0.90	6	No. 6 at 10 in.

9.3 A 16-ft- (4.8-m-)span simply supported slab carries a uniform dead load of 200 psf (10 kN/m^2) (excluding its own weight). The slab has a uniform thickness of 7 in. (175 mm) and is reinforced with no. 6 (20-mm) bars spaced at 5 in. (125 mm). Determine the allowable uniformly distributed load that can be applied on the slab if $f'_c = 4$ ksi (28 MPa) and $f_y = 60$ ksi (420 MPa)

9.4 Design a 10-ft (3-m) cantilever slab to carry a uniform total dead load of 170 psf (8.2 kN/m^2) and a concentrated live load at the free end of 2 K/ft (30 kN/m), when $f'_c = 4$ ksi (28 MPa) and $f_y = 60$ ksi (420 MPa).

9.5 A 6-in. (150-mm) solid one-way slab carries a uniform dead load of 190 psf (9.2 kN/m^2) (including its own weight) and a live load of 80 psf (3.9 kN/m^2). The slab spans 12 ft (3.6 m) between 10-in.-(250-mm-)wide simple supports. Determine the necessary slab reinforcement using $f'_c = 4$ ksi (28 MPa) and $f_y = 50$ ksi (350 MPa).

9.6 Repeat Problem 9.4 using a variable section with a minimum total depth at the free end of 4 in. (100 mm).

9.7 Design a continuous one-way solid slab supported on beams spaced at 14 ft (4.2 m) on centers. The width of the beams is 12 in. (300 mm), leaving clear slab spans of 13 ft (3.9 m). The slab carries a uniform dead load of 126 psf (6.0 kN/m$_2$) (including self-weight of slab) and a live load of 120 psf (5.8 kN/m$_2$). Use $f'_c = 3$ ksi (21 MPa), $f_y = 40$ ksi (280 MPa), and the ACI coefficients. Show bar arrangements using straight bars for all top and bottom reinforcement.

9.8 Repeat Problem 9.7 using equal clear spans of 10 ft (3 m), $f'_c = 3$ ksi (21 MPa), and $f_y = 60$ ksi (420 MPa).

9.9 Repeat Problem 9.7 using $f'_c = 4$ ksi (28 MPa) and $f_y = 60$ ksi (420 MPa).

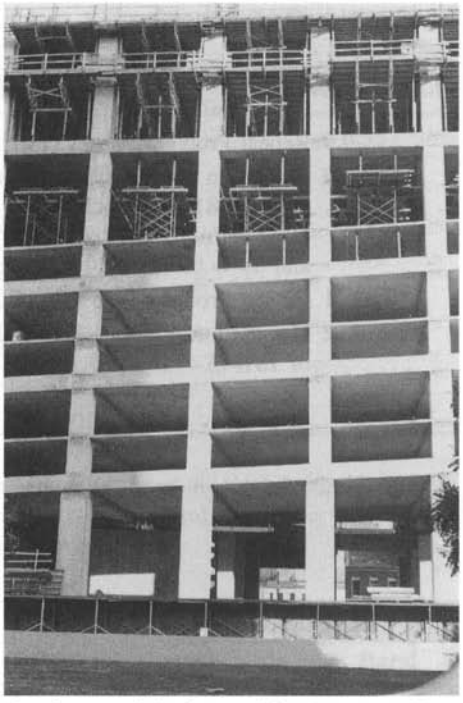
9.10 Design an interior rib of a concrete joist floor system with the following description: Span of ribbed slab is 18 ft (5.4 m) between simple supports; uniform dead load (excluding self-weight) is 30 psf (1.44 kN/m^2); live load is 100 psf (4.8 kN/m^2); support width is 14 in. (350 mm); $f'_c = 3$ ksi (21 MPa) and $f_y = 60$ ksi (420 MPa). Use 30-in.- (750-mm-)wide removable pans.

9.11 Repeat Problem 9.10 using 20-in.- (500-mm-)wide removable pans.

9.12 Use the information given in Problem 9.10 to design a continuous ribbed slab with three equal spans of 18 ft (5.4 m) each.

CHAPTER 10

AXIALLY LOADED COLUMNS



Continuous slabs in a parking structure, New Orleans, Louisiana.

10.1 INTRODUCTION

Columns are members used primarily to support axial compressive loads and have a ratio of height to the least lateral dimension of 3 or greater. In reinforced concrete buildings, concrete beams, floors, and columns are cast monolithically, causing some moments in the columns due to end restraint. Moreover, perfect vertical alignment of columns in a multistory building is not possible, causing loads to be eccentric relative to the center of columns. The eccentric loads will cause moments in columns. Therefore, a column subjected to pure axial loads does not exist in concrete buildings. However, it can be assumed that axially loaded columns are those with relatively small eccentricity, e, of about $0.1h$ or less, where h is the total depth of the column and e is the eccentric distance from the center of the column. Because concrete has a high compressive strength and is an inexpensive material, it can be used in the design of compression members economically. This chapter deals only with short columns; slender columns are covered in detail in Chapter 12.

10.2 TYPES OF COLUMNS

Columns may be classified based on the following different categories (Fig. 10.1):

1. Based on loading, columns may be classified as follows:
   a. Axially loaded columns, where loads are assumed acting at the center of the column section.

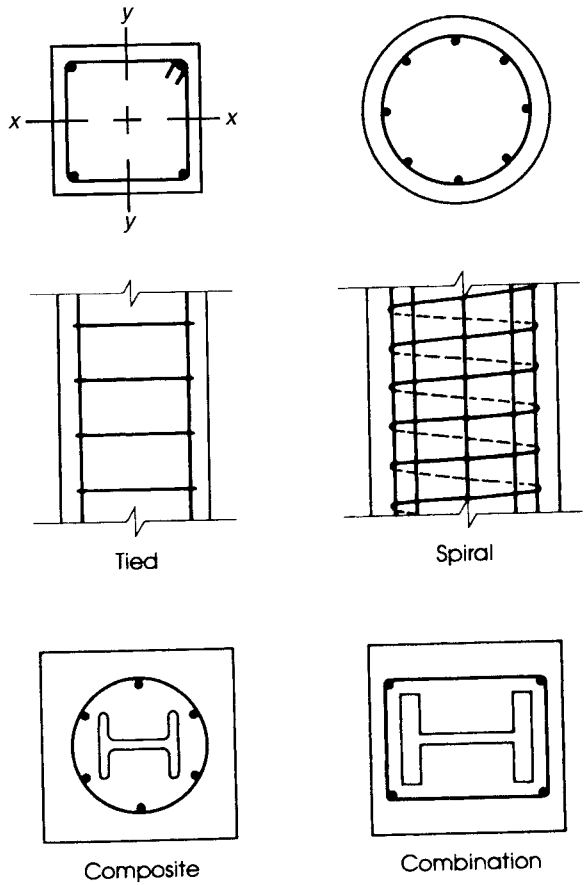


Figure 10.1 Types of columns.

b. Eccentrically loaded columns, where loads are acting at a distance e from the center of the column section. The distance e could be along the x- or y-axis, causing moments either about the x- or y-axis.

c. Biaxially loaded columns, where the load is applied at any point on the column section, causing moments about both the x- and y-axes simultaneously.

2. Based on length, columns may be classified as follows:

a. Short columns, where the column's failure is due to the crushing of concrete or the yielding of the steel bars under the full load capacity of the column.

b. Long columns, where buckling effect and slenderness ratio must be taken into consideration in the design, thus reducing the load capacity of the column relative to that of a short column.

3. Based on the shape of the cross-section, column sections may be square, rectangular, round, L-shaped, octagonal, or any desired shape with an adequate side width or dimensions.

4. Based on column ties, columns may be classified as follows:

a. Tied columns containing steel ties to confine the main longitudinal bars in the columns. Ties are normally spaced uniformly along the height of the column.

b. Spiral columns containing spirals (spring-type reinforcement) to hold the main longitudinal reinforcement and to help increase the column ductility before failure. In general, ties and spirals prevent the slender, highly stressed longitudinal bars from buckling and bursting the concrete cover.

5. Based on frame bracing, columns may be part of a frame that is braced against sidesway or unbraced against sidesway. Bracing may be achieved by using shear walls or bracings in the building frame. In braced frames, columns resist mainly gravity loads, and shear walls resist lateral loads and wind loads. In unbraced frames, columns resist both gravity and lateral loads, which reduce the load capacity of the columns.
6. Based on materials, columns may be reinforced, prestressed, composite (containing rolled steel sections such as I-sections), or a combination of rolled steel sections and reinforcing bars. Concrete columns reinforced with longitudinal reinforcing bars are the most common type used in concrete buildings.

10.3 BEHAVIOR OF AXIALLY LOADED COLUMNS

When an axial load is applied to a reinforced concrete short column, the concrete can be considered to behave elastically up to a low stress of about $(\frac{1}{3})f'_c$. If the load on the column is increased to reach its ultimate strength, the concrete will reach the maximum strength and the steel will reach its yield strength, f_y. The nominal load capacity of the column can be written as follows:

$$P_o = 0.85f'_cA_n + A_{st}f_y \tag{10.1}$$

where A_n and A_{st} = the net concrete and total steel compressive areas, respectively.

$$A_n = A_g - A_{st}$$

$$A_g = \text{gross concrete area}$$

Two different types of failure occur in columns, depending on whether ties or spirals are used. For a tied column, the concrete fails by crushing and shearing outward, the longitudinal steel bars fail by buckling outward between ties, and the column failure occurs suddenly, much like the failure of a concrete cylinder.

A spiral column undergoes a marked yielding, followed by considerable deformation before complete failure. The concrete in the outer shell fails and spalls off. The concrete inside the spiral is confined and provides little strength before the initiation of column failure. A hoop tension develops in the spiral, and for a closely spaced spiral, the steel may yield. A sudden failure is not expected. Figure 10.2 shows typical load deformation curves for tied and spiral columns. Up to point a, both columns behave similarly. At point a, the longitudinal steel bars of the column yield, and the spiral column shell spalls off. After the factored load is reached, a tied column fails suddenly (curve b), whereas a spiral column deforms appreciably before failure (curve c).

10.4 ACI CODE LIMITATIONS

The ACI Code presents the following limitations for the design of compression members:

1. For axially as well as eccentrically loaded columns, the ACI Code sets the strength-reduction factors at $\phi = 0.65$ for tied columns and $\phi = 0.75$ for spirally reinforced columns. The

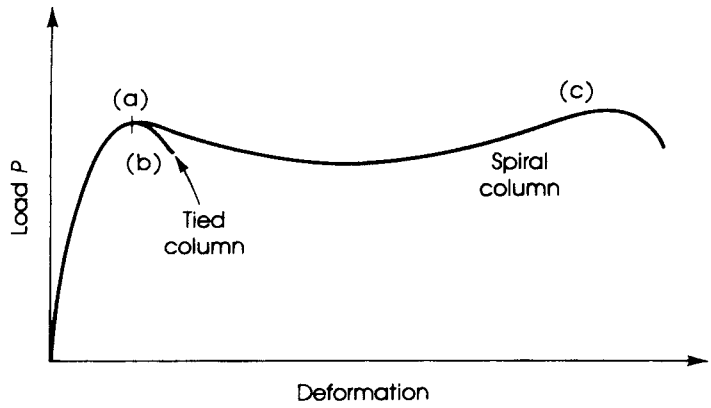


Figure 10.2 Behavior of tied and spiral columns.

difference of 0.05 between the two values shows the additional ductility of spirally reinforced columns.

The strength-reduction factor for columns is much lower than those for flexure ($\phi = 0.9$) and shear ($\phi = 0.75$). This is because in axially loaded columns, the strength depends mainly on the concrete compression strength, whereas the strength of members in bending is less affected by the variation of concrete strength, especially in the case of an under-reinforced section. Furthermore, the concrete in columns is subjected to more segregation than in the case of beams. Columns are cast vertically in long, narrow forms, but the concrete in beams is cast in shallow, horizontal forms. Also, the failure of a column in a structure is more critical than that of a floor beam.

2. The minimum longitudinal steel percentage is 1%, and the maximum percentage is 8% of the gross area of the section (ACI Code, Section 10.9.1). Minimum reinforcement is necessary to provide resistance to bending, which may exist, and to reduce the effects of creep and shrinkage of the concrete under sustained compressive stresses. Practically, it is very difficult to fit more than 8% of steel reinforcement into a column and maintain sufficient space for concrete to flow between bars.
3. At least four bars are required for tied circular and rectangular members and six bars are needed for circular members enclosed by spirals (ACI Code, Section 10.9.2). For other shapes, one bar should be provided at each corner, and proper lateral reinforcement must be provided. For tied triangular columns, at least three bars are required. Bars shall not be located at a distance greater than 6 in. clear on either side from a laterally supported bar. Figure 10.3 shows the arrangement of longitudinal bars in tied columns and the distribution of ties. Ties shown in dotted lines are required when the clear distance on either side from laterally supported bars exceeds 6 in. The minimum concrete cover in columns is 1.5 in.
4. The minimum ratio of spiral reinforcement, ρ_s, according to the ACI Code, Eq. 10.5, and as explained in Section 10.9.3, is limited to

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}} \tag{10.2}$$

where

A_g = gross area of section

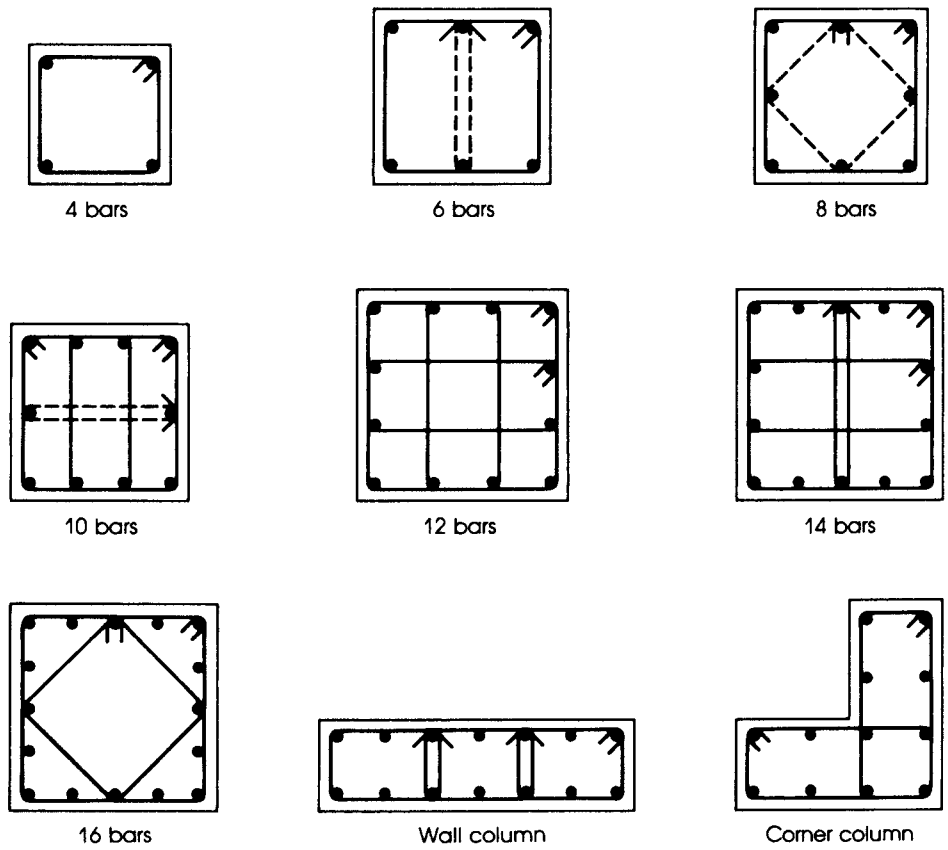


Figure 10.3 Arrangement of bars and ties in columns.

A_{ch} =area of core of spirally reinforced column measured to the outside diameter of spiral

f_{yt} = yield strength of spiral reinforcement (60 ksi; ACI Code, Section 10.9.3)

5. The minimum diameter of spirals is $\frac{3}{8}$ in., and their clear spacing should not be more than 3 in. nor less than 1 in., according to the ACI Code, Section 7.10.4. Splices may be provided by welding or lapping the deformed uncoated spiral bars by 48 diameters or a minimum of 12 in. Lap splices for plain uncoated bar or wire = $72d_p \leq 12$ in. The same applies for epoxy-coated deformed bar or wire. The Code also allows full mechanical splices.
6. Ties for columns must have a minimum diameter of $\frac{3}{8}$ in. to enclose longitudinal bars of no. 10 size or smaller and a minimum diameter of $\frac{1}{2}$ in. for larger bar diameters (ACI Code, Section 7.10.5).
7. Spacing of ties shall not exceed the smallest of 48 times the tie diameter, 16 times the longitudinal bar diameter, or the least dimension of the column. Table 10.1 gives spacings for no. 3 and no. 4 ties. The Code does not give restrictions on the size of columns to allow wider utilization of reinforced concrete columns in smaller sizes.

Table 10.1 Maximum Spacings of Ties

Column Least Side or Diameter (in.)	Spacings of Ties (in.) for Bar					
	No. 6	No. 7	No. 8	No. 9	No. 10	No. 11
12	12	12	12	12	12	12
14	12	14	14	14	14	14
16	12	14	16	16	16	16
18	12	14	16	18	18	18
20	12	14	16	18	18	20
22–40	12	14	16	18	18	22
Ties	No. 3	No. 3	No. 3	No. 3	No. 3	No. 4

10.5 SPIRAL REINFORCEMENT

Spiral reinforcement in compression members prevents a sudden crushing of concrete and buckling of longitudinal steel bars. It has the advantage of producing a tough column that undergoes gradual and ductile failure. The minimum spiral ratio required by the ACI Code is meant to provide an additional compressive capacity to compensate for the spalling of the column shell. The strength contribution of the shell is

$$P_u(\text{shell}) = 0.85 f'_c (A_g - A_{ch}) \tag{10.3}$$

where A_g is the gross concrete area and A_{ch} is the core area (Fig. 10.4).

In spirally reinforced columns, spiral steel is at least twice as effective as longitudinal bars; therefore, the strength contribution of spiral equals $2\rho_s A_{ch} f_{yt}$, where ρ_s is the ratio of volume of spiral reinforcement to total volume of core.

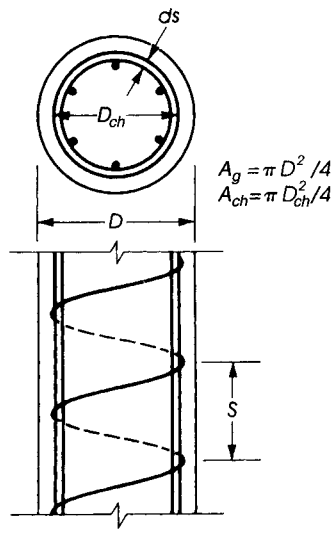


Figure 10.4 Dimensions of a column spiral.

Table 10.2 Spirals for Circular Columns ($f_y = 60$ ksi)

	$f'_c = 4$ ksi No. 3 Spirals	$f'_c = 5$ ksi No. 3 and no. 4 Spirals		$f'_c = 6$ ksi No. 4 Spirals
Column Diameter (in.)	Spacing (in.)	Spiral No.	Spacing (in.)	Spacing (in.)
12	2.0	4	2.75	2.25
14	2.0	4	3.00	2.25
16	2.0	4	3.00	2.50
18	2.0	4	3.00	2.50
20	2.0	4	3.00	2.50
22	2.0	4	3.00	2.50
24	2.0	3	1.75	2.50
26 to 40	2.25	3	1.75	2.75

If the strength of the column shell is equated to the spiral strength contribution, then

$$0.85 f'_c (A_g - A_{ch}) = 2\rho_s A_{ch} f_{yt} \tag{10.4}$$

$$\rho_s = 0.425 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$$

The ACI Code adopted a minimum ratio of ρ_s according to the following equation:

$$\text{Minimum } \rho_s = 0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \tag{10.2}$$

The design relationship of spirals may be obtained as follows (Fig. 10.4):

$$\rho_s = \frac{\text{volume of spiral in one loop}}{\text{volume of core for a spacing } S}$$

$$= \frac{a_s \pi (D_{ch} - d_s)}{\left(\frac{\pi}{4} D_{ch}^2\right) S} = \frac{4 a_s (D_{ch} - d_s)}{D_{ch}^2 S} \tag{10.5}$$

where

a_s = area of spiral reinforcement
D_{ch} =diameter of the core measured to the outside diameter of spiral
D = diameter of the column
d_s = diameter of the spiral
S = spacing of the spiral

Table 10.2 gives spiral spacings for no. 3 and no. 4 spirals with $f_y = 60$ ksi.

10.6 DESIGN EQUATIONS

The nominal load strength of an axially loaded column was given in Eq. 10.1. Because a perfect axially loaded column does not exist, some eccentricity occurs on the column section, thus

reducing its load capacity, P_o. To take that into consideration, the ACI Code specifies that the maximum nominal load, P_o, should be multiplied by a factor equal to 0.8 for tied columns and 0.85 for spirally reinforced columns. Introducing the strength reduction factor, the axial load strength of columns according to the ACI Code, Section 10.3.6, are as follows:

$$P_u = \phi P_n = \phi(0.80)[0.85f'_c(A_g - A_{st}) + A_{st}f_y] \tag{10.6}$$

for tied columns and

$$P_u = \phi P_n = \phi(0.85)[0.85f'_c(A_g - A_{st}) + A_{st}f_y] \tag{10.7}$$

for spiral columns, where

A_g = gross concrete area
A_{st} = total steel compressive area
ϕ = 0.65 for tied columns and 0.70 for spirally reinforced columns

Equations 10.8 and 10.9 may be written as follows:

$$P_u = \phi P_n = \phi K[0.85f'_c A_g + A_{st}(f_y - 0.85f'_c)] \tag{10.8}$$

where $\phi = 0.65$ and $K = 0.8$ for tied columns and $\phi = 0.75$ and $K = 0.85$ for spiral columns.

If the gross steel ratio is $\rho_g = A_{st}/A_g$, or $A_{st} = \rho_g A_g$, then Eq. 10.8 may be written as follows:

$$P_u = \phi P_n = \phi K A_g[0.85f'_c + \rho_g(f_y - 0.85f'_c)] \tag{10.9}$$

Equation 10.8 can be used to calculate the axial load strength of the column, whereas Eq. 10.9 is used when the external factored load is given and it is required to calculate the size of the column section, A_g, based on an assumed steel ratio, ρ_g, between a minimum of 1% and a maximum of 8%.

It is a common practice to use grade 60 reinforcing steel bars in columns with a concrete compressive strength of 4 ksi or greater to produce relatively small concrete column sections.

10.7 AXIAL TENSION

Concrete will not crack as long as stresses are below its tensile strength; in this case, both concrete and steel resist the tensile stresses, but when the tension force exceeds the tensile strength of concrete (about one-tenth of the compressive strength), cracks develop across the section, and the entire tension force is resisted by steel. The nominal load that the member can carry is that due to tension steel only:

$$T_n = A_{st}f_y \tag{10.10}$$

$$T_u = \phi A_{st}f_y \tag{10.11}$$

where $\phi = 0.9$ for axial tension.

Tie rods in arches and similar structures are subjected to axial tension. Under working loads, the concrete cracks and the steel bars carry the whole tension force. The concrete acts as a fire and corrosion protector. Special provisions must be taken for water structures, as in the case of water tanks. In such designs, the concrete is not allowed to crack under the tension caused by the fluid pressure.

10.8 LONG COLUMNS

The equations developed in this chapter for the strength of axially loaded members are for short columns. In the case of long columns, the load capacity of the column is reduced by a reduction factor.

A long column is one with a high slenderness ratio, h/r, where h is the effective height of the column and r is the radius of gyration. The design of long columns is explained in detail in Chapter 12.

Example 10.1

Determine the allowable design axial load on a 12-in. square, short tied column reinforced with four no. 9 bars. Ties are no. 3 spaced at 12 in. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. Using Eq. 10.9,

$$P_u = \phi P_n = \phi K[0.85 f'_c A_g + A_{st}(f_y - 0.85 f'_c)]$$

For a tied column, $\phi = 0.65$, $K = 0.8$, and $A_{st} = 4.0\,\text{in.}^2$

$$P_u = \phi P_u = 0.65(0.8)[0.85(4)(12 \times 12) + 4(60 - 0.85 \times 4)] = 372\ \text{K}$$

2. Check steel percentage: $\rho_g = \frac{4}{144} = 0.02778 = 2.778\%$. This is less than 8% and greater than 1%.
3. Check tie spacings: Minimum tie diameter is no. 3. Spacing is the smallest of the 48-tie diameter, 16-bar diameter, or least column side. $S_1 = 48(\frac{3}{8}) = 18$ in., $S_2 = 16(\frac{9}{8}) = 18$ in., $S_3 = 12.0$ in. Ties are adequate (Table 10.1).

Example 10.2

Design a square tied column to support an axial dead load of 400 K and a live load of 232 K using $f'_c = 5$ksi, $f_y = 60$ ksi and a steel ratio of about 5%. Design the necessary ties.

Solution

1. Calculate $P_u = 1.2P_D + 1.6P_L = 1.2(400) + 1.6(232) = 851$ K. Using Eq. 10.10, $P_u = 851 = 0.65(0.8)A_g[0.85 \times 5 + 0.05(60 - 0.85 \times 5)]$, $A_g = 232.5\,\text{in.}^2$, and column side $= 15.25$ in., so use 16 in. (Actual $A_g = 256\,\text{in.}^2$)
2. Because a larger section is adopted, the steel percentage may be reduced by using $A_g = 256\,\text{in.}^2$ in Eq. 10.8:

$$851 = 0.65(0.8)[0.85 \times 5 \times 256 + A_{st}(60 - 0.85 \times 5)]$$

$$A_{st} = 9.84\ \text{in.}^2$$

Use eight no. 11 bars ($A_{st} = 12.50\,\text{in.}^2$). See Fig. 10.5.

3. Design of ties (by calculation or from Table 10.1): Choose no. 3 ties with spacings equal to the least of $S_1 = 16(\frac{11}{8}) = 22$ in., $S_2 = 48(\frac{3}{8}) = 18$ in., or $S_3 =$ column side $= 16$ in. Use no. 3 ties spaced at 16 in. Clear distance between bars is 4.23 in., which is less than 6 in. Therefore, no additional ties are required.

Example 10.3

Repeat Example 10.2 using a rectangular section that has a width of $b = 14$ in.

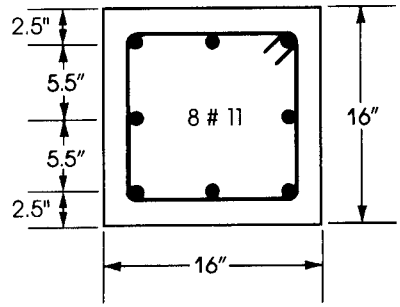


Figure 10.5 Example 10.2.

Solution

1. $P_u = 851$ K and calculated $A_g = 232.5\,\text{in.}^2$ For $b = 14$ in., $h = 232.5/14 = 16.6$ in. Choose a column 14 × 18 in.; actual $A_g = 252\,\text{in.}^2$.
2. $P_u = 851 = 0.65(0.8)[0.85 \times 5 \times 252 + A_{st}(60 - 0.85 \times 5)]$

$$A_{st} = 10.14\ \text{in.}^2$$

   Use eight no. 10 bars. ($A_{st} = 10.16\,\text{in.}^2$)

3. Design of ties: Choose no. 3 ties, $S_1 = 20$ in., $S_2 = 18$ in., and $S_3 = 14$ in. (least side). Use no. 3 ties spaced at 14 in. Clear distance between bars in the long direction is (18 − 5)/2 − bar diameter of 1.27 = 5.23 in. < 6 in. No additional ties are needed. Clear distance in the short direction is (14 − 5)/2 − 1.27 = 3.23 in. < 6 in.

Example 10.4

Design a circular spiral column to support an axial dead load of 475 K and a live load of 250 K using $f'_c = 4$ ksi, $f_y = 60$ ksi, and a steel ratio of about 3%. Also, design the necessary spirals.

Solution

1. Calculate $P_u = 1.2P_D + 1.6P_L = 1.2(475) + 1.6(250) = 970$ K. Using Eq. 10.10 and spiral columns,

$$P_u = 970 = 0.75(0.85)A_g[0.85 \times 4 + 0.03(60 - 0.85 \times 4)]$$

   $A_g = 299\,\text{in.}^2$ and column diameter = 19.5 in., so use 20 in. Actual $A_g = 314.2\,\text{in.}^2$

2. Calculate A_{st} needed from Eq. 10.8:

$$P_u = 970 = 0.75(0.85)[0.85 \times 4 \times 314.2 + A_{st}(60 - 0.85 \times 4)]$$

$$A_{st} = 8\ \text{in.}^3$$

   Use eight no. 10 bars. ($A_{st} = 10.16\,\text{in.}^2$)

3. Design of spirals: The diameter of core is 20 − 2(1.5) = 17 in. The area of core is

$$A_{ch} = \frac{\pi}{4}(17)^2 \quad A_g = \frac{\pi}{4}(20)^2$$

$$\text{Minimum } \rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}} = 0.45\left(\frac{20^2}{17^2} - 1\right)\left(\frac{4}{60}\right) = 0.01152$$

Assume no. 3 spiral, $a_s = 0.11\,\text{in.}^2$, and $d_s = 0.375\,\text{in.}$

$$\rho_s = 0.01152 = \frac{4a_s(D_{ch} - d_b)}{SD_{ch}^2} = \frac{4(0.11)(17 - 0.375)}{S(17)^2}$$

Spacing s is equal to 2.2 in; use no. 3 spiral at $s = 2\,\text{in.}$ (as shown in Table 10.2).

Example 10.5

Design a rectangular tied short column to carry a factored axial load of 1765 kN. Use $f'_c = 30$ MPa, $f_y = 400$ MPa, column width (b) = 300 mm, and a steel ratio of about 2%.

Solution SI Units

1. Using Eq. 10.9,

$$P_u = 0.8\phi A_g[0.85f'_c + \rho_g(f_y - 0.85f'_c)]$$

Assuming a steel percentage of 2%,

$$1765 \times 10^3 = 0.8 \times 0.65A_g[0.85 \times 30 + 0.02(400 - 0.85 \times 30)]$$

$$A_g = 102{,}887\ \text{mm}^2$$

For $b = 300$ mm, the other side of the rectangular column is 343 mm. Therefore, use a section of 300 by 350 mm ($A_g = 105{,}000\,\text{mm}^2$).

2. $A_s = 0.02 \times 102{,}887 = 2057\,\text{mm}^2$. Choose six bars, 22 mm in diameter ($A_s = 2280\,\text{mm}^2$).
3. Check the axial load strength of the section using Eq. 10.6:

$$\phi P_n = 0.8\phi[0.85f'_c(A_g - A_{st}) + A_{st}f_y]$$

$$= 0.8 \times 0.65[0.85 \times 30(105{,}000 - 2280) + 2280 \times 400] \times 10^{-3}$$

$$= 1836\ \text{kN}$$

This meets the required P_u of 1765 kN.

4. Choose ties 10 mm in diameter. Spacing is the least of (1) $16 \times 22 = 352$ mm, (2) $48 \times 10 = 480$ mm, or (3) 300 mm. Choose 10-mm ties spaced at 300 mm.

SUMMARY

Sections 10.1–10.4

Columns may be tied or spirally reinforced.

$\phi = 0.65$ for tied columns

$\phi = 0.75$ for spirally reinforced columns

ρ_g must be $\leq 8\%$ and $\geq 1\%$.

Section 10.5

Minimum ratio of spirals is

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}} \tag{10.2}$$

$$\rho_s = \frac{4a_s(D_{ch} - d_s)}{D_{ch}^2 S} \tag{10.5}$$

The minimum diameter of spirals is $\frac{3}{8}$ in., and their clear spacings should be not more than 3 in. or less than 1 in.

Section 10.6

For tied columns,

$$P_u = \phi P_n = 0.8\phi[0.85f'_c(A_g - A_{st}) + A_{st}f_y] \tag{10.6}$$

or

$$P_u = \phi P_n = 0.8\phi A_g[0.85f'_c + \rho_g(f_y - 0.85f'_c)]$$

For spiral columns,

$$P_u = \phi P_n = 0.85\phi[0.85f'_c(A_g - A_{st}) + A_{st}f_y] \tag{10.7}$$

or

$$P_u = \phi P_n = 0.85\phi A_g[0.85f'_c + \rho_g(f_y - 0.85f'_c)]$$

where $\rho_g = A_{st}/A_g$.

Section 10.7

1. For axial tension,

$$T_u = \phi A_{st} f_y \quad (\phi = 0.9) \tag{10.11}$$

2. Arrangements of vertical bars and ties in columns are shown in Fig. 10.3.

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PROBLEMS

10.1 For each problem, determine the allowable design load-bearing strength ($0.8\phi P_o$) for each of the following short rectangular columns according to the ACI Code limitations. Assume $f_y = 60$ ksi and properly tied columns (b = width of column, in., and h = total depth, in.).

Number	f'_c(ksi)	b (in.)	h (in.)	Bars	Answer (ϕkP_o) K
(a)	4	16	16	8 no. 9	688
(b)	4	20	20	16 no. 11	1442
(c)	4	12	12	8 no. 8	439
(d)	4	12	24	12 no. 10	955
(e)	5	14	14	10 no. 9	722
(f)	5	16	16	4 no. 10	712
(g)	5	14	26	12 no. 10	1244
(h)	5	18	32	8 no. 11	1634
(i)	6	16	16	8 no. 10	968
(j)	6	12	20	6 no. 10	852

10.2 For each problem, determine the allowable design load-bearing strength of each of the following short, spirally reinforced circular columns according to the ACI Code limitations. Assume $f_y = 60$ ksi and the spirals are adequate (D = diameter of column, in.).

Number	f'_c(ksi)	D (in.)	Bars	Answer (ϕkP_o) K
(a)	4	14	8 no. 9	581
(b)	4	16	6 no. 10	663
(c)	5	18	8 no. 10	980
(d)	5	20	12 no. 10	1300
(e)	6	15	8 no. 9	797

10.3 For each problem, design a short square, rectangular, or circular column, as indicated, for each set of axial loads given, according to ACI limitations. Also, design the necessary ties or spirals and draw sketches of the column sections showing all bar arrangements. Use $f_y = 60$ ksi and a steel ratio close to the ρ_g given (P_D = dead load, P_L = live load, b = width of a rectangular column, and = $\rho_g = A_{st}/A_g$).

Number	f'_c(ksi)	P_D (K)	P_L (K)	ρ_g%	Section	One Solution
(a)	4	200	200	4	Square	14 × 14, 8 no. 9
(b)	4	750	400	3.5	Square	24 × 24, 16 no. 10
(c)	4	220	165	7	Square	12 × 12, 8 no. 10
(d)	5	330	230	3	Square	16 × 16, 8 no. 9
(e)	4	190	170	2	Rectangular, b = 12 in.	12 × 18, 6 no. 8
(f)	4	280	315	4.5	Rectangular, b = 14 in.	14 × 20, 10 no. 10
(g)	4	210	150	3	Rectangular, b = 12 in.	12 × 16, 6 no. 9
(h)	5	690	460	2	Rectangular, b = 18 in.	18 × 32, 8 no. 10
(i)	4	350	130	4	Circular—spiral	16, 7 no. 9
(j)	4	475	220	3.25	Circular—spiral	20, 7 no. 10
(k)	4	400	260	5	Circular—spiral	18, 9 no. 10
(l)	5	285	200	4.25	Circular—spiral	15, 6 no. 10

For SI units, use 1 psi = 0.0069 MPa, 1 K = 4.45 kN, and 1 in. = 25.4 mm.

CHAPTER 11

MEMBERS IN COMPRESSION AND BENDING



Residential building, Minneapolis, Minnesota.

11.1 INTRODUCTION

Vertical members that are part of a building frame are subjected to combined axial loads and bending moments. These forces develop due to external loads, such as dead, live, and wind loads. The forces are determined by manual calculations or computer applications that are based on the principles of statics and structural analysis. For example, Fig. 11.1 shows a two-hinged portal frame that carries a uniform factored load on BC. The bending moment is drawn on the tension side of the frame for clarification. Columns AB and CD are subjected to an axial compressive force and a bending moment. The ratio of the moment to the axial force is usually defined as the eccentricity, e, where $e = M_n/P_n$ (Fig. 11.1). The eccentricity, e, represents the distance from the plastic centroid of the section to the point of application of the load. The plastic centroid is obtained by determining the location of the resultant force produced by the steel and the concrete, assuming that both are stressed in compression to f_y and 0.85 f'_c, respectively. For symmetrical sections, the plastic centroid coincides with the centroid of the section. For nonsymmetrical sections, the plastic centroid is determined by taking moments about an arbitrary axis, as explained in Example 11.1.

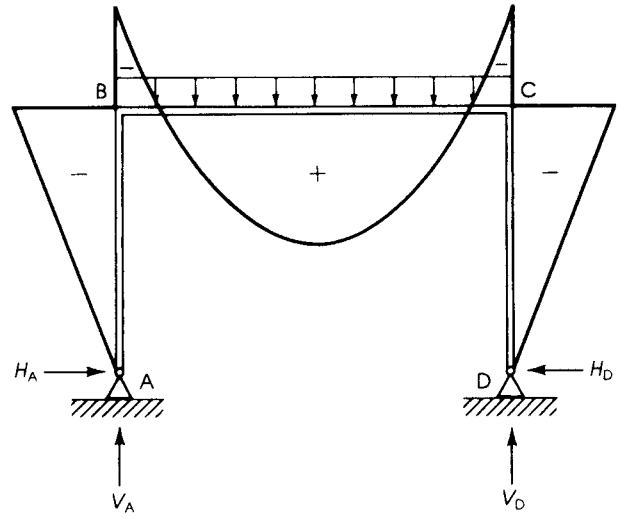


Figure 11.1 Two-hinged portal frame with bending moment diagram drawn on the tension side.

Example 11.1

Determine the plastic centroid of the section shown in Fig. 11.2. Given: $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. It is assumed that the concrete is stressed in compression to 0.85 f'_c:

$$F_c = \text{force in concrete} = (0.85\, f'_c)\, A_g$$
$$= (0.85 \times 4) \times 14 \times 20 = 952 \text{ K}$$

F_c is located at the centroid of the concrete section (at 10 in. from axis A-A).

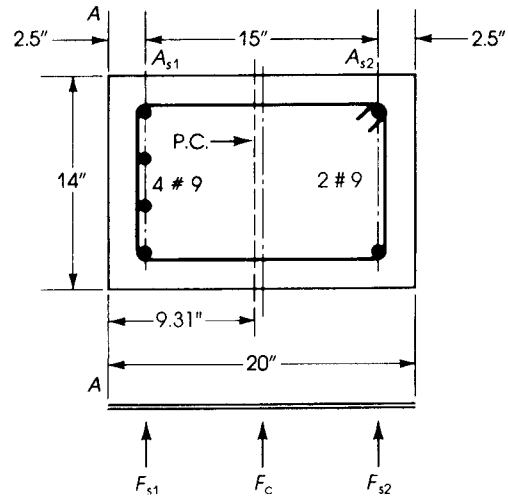


Figure 11.2 Example 11.1: Plastic centroid (P.C.) of section.

2. Forces in steel bars:

$$F_{s1} = A_{s1} f_y = 4 \times 60 = 240 \text{ K}$$

$$F_{s2} = A_{s2} f_y = 2 \times 60 = 120 \text{ K}$$

3. Take moments about A-A:

$$x = \frac{(952 \times 10) + (240 \times 2.5) + (120 \times 17.5)}{952 + 240 + 120} = 9.31 \text{ in.}$$

Therefore, the plastic centroid lies at 9.31 in. from axis A-A.

4. If $A_{s1} = A_{s2}$ (symmetrical section), then $x = 10$ in. from axis A-A.

11.2 DESIGN ASSUMPTIONS FOR COLUMNS

The design limitations for columns, according to the ACI Code, Section 10.2, are as follows:

1. Strains in concrete and steel are proportional to the distance from the neutral axis.
2. Equilibrium of forces and strain compatibility must be satisfied.
3. The maximum usable compressive strain in concrete is 0.003.
4. Strength of concrete in tension can be neglected.
5. The stress in the steel is $f_s = \varepsilon E_s \leq f_y$.
6. The concrete stress block may be taken as a rectangular shape with concrete stress of 0.85 f'_c that extends from the extreme compressive fibers a distance $a = \beta_1 c$, where c is the distance to the neutral axis and β_1 is 0.85 when $f'_c \leq 4000$ psi (30 MPa); β_1 decreases by 0.05 for each 1000 psi above 4000 psi (0.008 per 1 MPa above 30 MPa) but is not less than 0.65. (Refer to Fig. 3.6, Chapter 3.)

11.3 LOAD-MOMENT INTERACTION DIAGRAM

When a normal force is applied on a short reinforced concrete column, the following cases may arise, according to the location of the normal force with respect to the plastic centroid. Refer to Fig. 11.3*a* and 11.3*b*:

Axial compression (P_0). This is a theoretical case assuming that a large axial load is acting at the plastic centroid; $e = 0$ and $M_n = 0$. Failure of the column occurs by crushing of the concrete and yielding of steel bars. This is represented by P_0 on the curve of Fig. 11.3*a*.

1. *Maximum nominal axial load* $P_{n\,\max}$: This is the case of a normal force acting on the section with minimum eccentricity. According to the ACI Code, $P_{n\,\max} = 0.80 P_0$ for tied columns and $0.85 P_0$ for spirally reinforced columns, as explained in Chapter 10. In this case, failure occurs by crushing of the concrete and the yielding of steel bars.
2. *Compression failure:* This is the case of a large axial load acting at a small eccentricity. The range of this case varies from a maximum value of $P_n = P_{n\,\max}$ to a minimum value of $P_n = P_b$ (balanced load). Failure occurs by crushing of the concrete on the compression side with a strain of 0.003, whereas the stress in the steel bars (on the tension side) is less than the yield strength, f_y ($f_s < f_y$). In this case $P_n > P_b$ and $e < e_b$.

3. *Balanced condition (P_b):* A balanced condition is reached when the compression strain in the concrete reaches 0.003 and the strain in the tensile reinforcement reaches $\varepsilon_y = f_y/E_s$ simultaneously; failure of concrete occurs at the same time as the steel yields. The moment that accompanies this load is called the *balanced moment*, M_b, and the relevant balanced eccentricity is $e_b = M_b/P_b$.
4. *Tension failure:* This is the case of a small axial load with large eccentricity, that is, a large moment. Before failure, tension occurs in a large portion of the section, causing the tension steel bars to yield before actual crushing of the concrete. At failure, the strain in the tension steel is greater than the yield strain, ε_y, whereas the strain in the concrete reaches 0.003. The range of this case extends from the balanced to the case of pure flexure (Fig. 11.3). When tension controls, $P_n < P_b$ and $e > e_b$.
5. *Pure flexure:* The section in this case is subjected to a bending moment, M_n, whereas the axial load is $P_n = 0$. Failure occurs as in a beam subjected to bending moment only. The eccentricity is assumed to be at infinity. Note that radial lines from the origin represent constant ratios of $M_n/P_n = e$ = eccentricity of the load P_n from the plastic centroid.

Cases 1 and 2 were discussed in Chapter 10, and Case 6 was discussed in detail in Chapter 3. The other cases are discussed in this chapter.

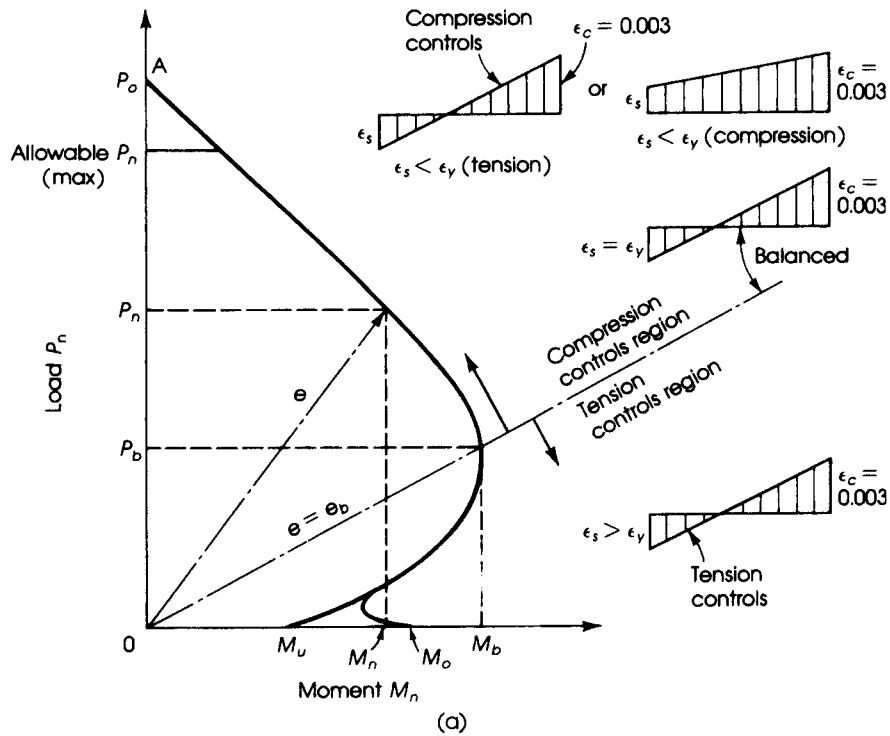
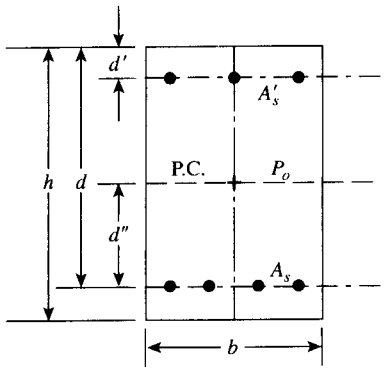
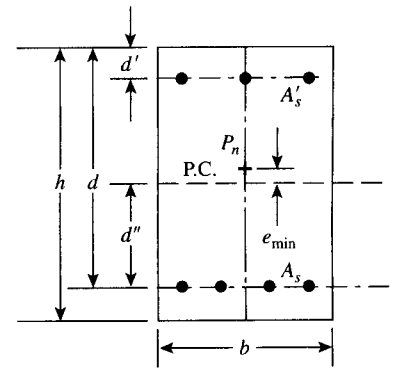


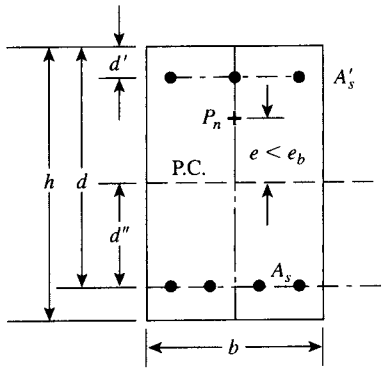
Figure 11.3 (*a*) Load-moment strength interaction diagram showing ranges of cases discussed in text, and (*b*) column sections showing the location of P_n for different load conditions.



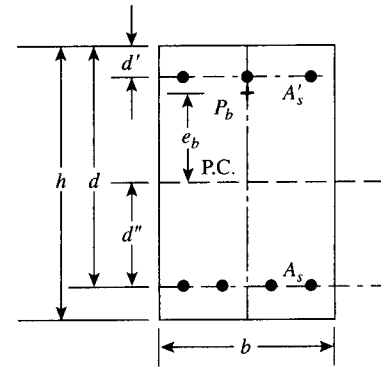
Case 1: Axial load, P_0



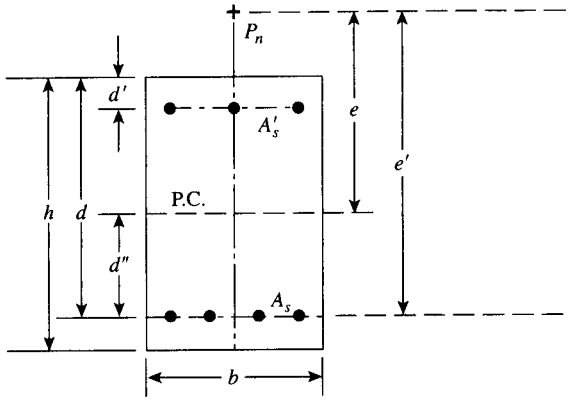
Case 2: $P_n = 0.8\,P_0$



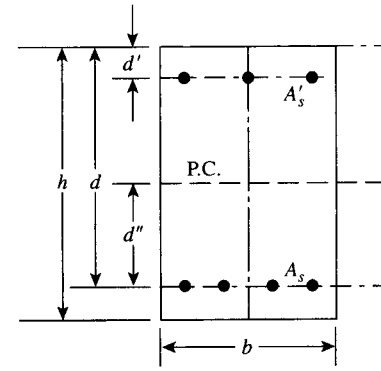
Case 3: Compression controls, $P_n > P_b$



Case 4: Balanced load, P_b



Case 5: Tension controls, $P_n < P_b$



Case 6: Pure moment, $P_n = 0$

(b)

Figure 11.3 *(continued)*

11.4 SAFETY PROVISIONS

The safety provisions for load factors were discussed earlier in Section 3.6. For columns, the safety provisions may be summarized as follows:

1. Load factors for gravity and wind loads are

$$U = 1.4D$$
$$U = 1.2D + 1.6L$$
$$U = 1.2D + 1.6L + 0.8W$$
$$U = 1.2D + 1.0L + 1.6W$$
$$U = 0.9D + 1.6W$$

   The most critical factored load should be used.

2. The strength reduction factor, ϕ, to be used for columns may vary according to the following cases:
   - **a.** When $P_u = \phi P_n \geq 0.1 f'_c A_g$, ϕ is 0.65 for tied columns and 0.75 for spirally reinforced columns. This case occurs generally when compression failure is expected. A_g is the gross area of the concrete section.
   - **b.** The sections in which the net tensile strain, ε_t, at the extreme tension steel, at nominal strength, is between 0.005 and 0.002 (transition region) ϕ varies linearly between 0.90 and 0.65 (or 0.75), respectively (Fig. 11.4). Refer to Section 3.7. For spiral sections,

$$\phi = 0.75 + (\varepsilon_t - 0.002)(50) \quad \text{or} \quad \phi = 0.75 + 0.15\left[\frac{1}{c/d_t} - \frac{5}{3}\right] \tag{11.1}$$

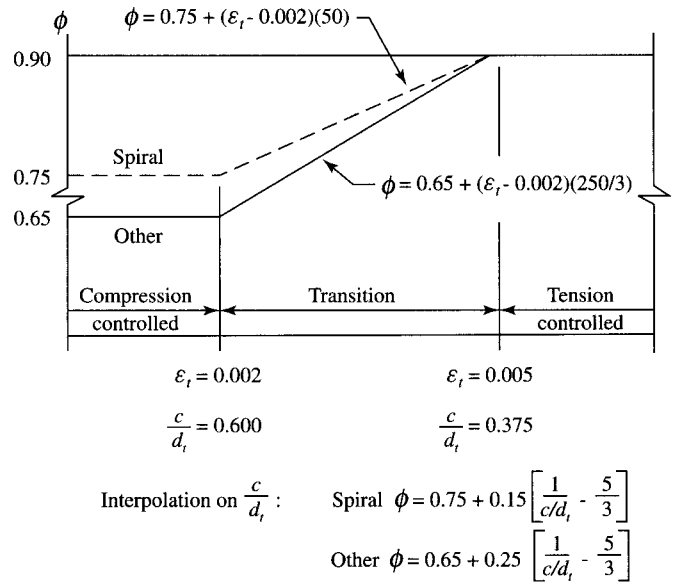


Figure 11.4 Variation in ϕ with NTS for grade 60 steel 7. Courtesy of ACI.

For spiral sections

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) \quad \text{or} \quad \phi = 0.65 + 0.25\left[\frac{1}{c/d_t} - \frac{5}{3}\right] \tag{11.2}$$

c. When $P_u = 0$, the case of pure flexure, then $\phi = 0.90$ for tension-controlled sections and varies between 0.90 and 0.65 (or 0.75) in the transition region.

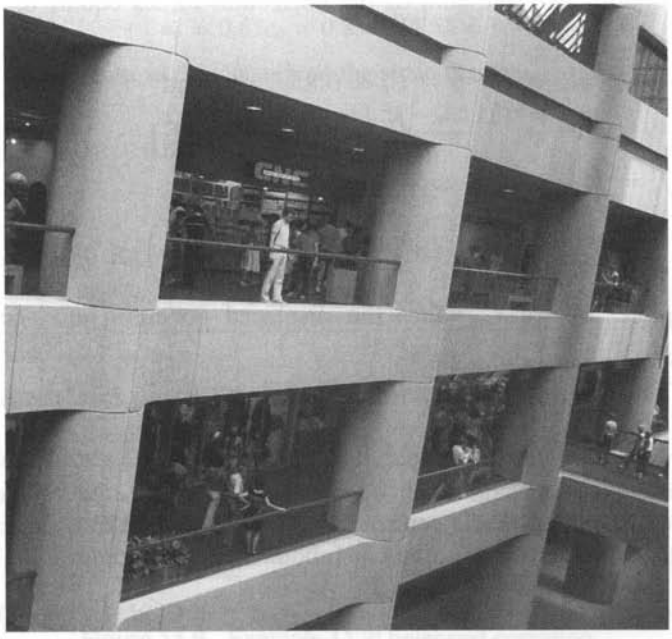
11.5 BALANCED CONDITION — RECTANGULAR SECTIONS

A balanced condition occurs in a column section when a load is applied on the section and produces, at nominal strength, a strain of 0.003 in the compressive fibers of concrete and a strain $\varepsilon_y = f_y/E_s$ in the tension steel bars simultaneously. This is a special case where the neutral axis can be determined from the strain diagram with known extreme values. When the applied eccentric load is greater than P_b, compression controls; if it is smaller than P_b, tension controls in the section.

The analysis of a balanced column section can be explained in steps (Fig. 11.5):

1. Let c equal the distance from the extreme compressive fibers to the neutral axis. From the strain diagram,

$$\frac{c_b(\text{balanced})}{d_t} = \frac{0.003}{0.003 + \dfrac{f_y}{E_s}} \qquad (\text{where } E_s = 29{,}000 \text{ ksi}) \tag{11.3}$$



Columns supporting 52-story building, Minneapolis, Minnesota. (Columns are 96 × 64 in. with round ends.)

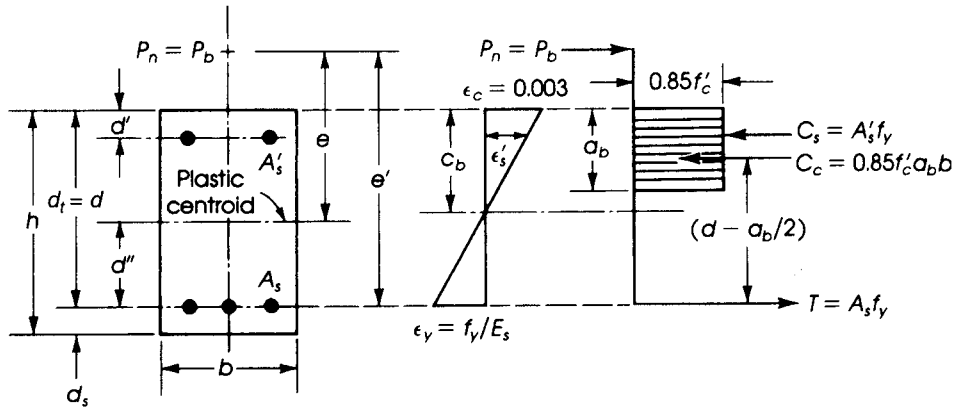


Figure 11.5 Balanced condition (rectangular section).

and

$$c_b = \frac{87d_t}{87 + f_y} \qquad \text{(where } f_y \text{ is in ksi)}$$

The depth of the equivalent compressive block is

$$a_b = \beta_1 c_b = \left(\frac{87}{87 + f_y}\right)\beta_1 d_t \tag{11.4}$$

where $\beta_1 = 0.85$ for $f'_c \leq 4000$ psi and decreases by 0.05 for each 1000-psi increase in f'_c.

2. From equilibrium, the sum of the horizontal forces equals 0: $P_b - C_c - C_s + T = 0$, where

$$C_c = 0.85f'_c ab \quad \text{and} \quad T = A_s f_y \tag{11.5}$$
$$C_s = A'_s(f'_s - 0.85f'_c)$$

(Use $f'_s = f_y$ if compression steel yields.)

$$f'_s = 87\left(\frac{c - d'}{c}\right) \leq f_y$$

The expression of C_s takes the displaced concrete into account. Therefore, Eq. 11.5 becomes

$$P_b = 0.85\, f'_c ab + A'_s(f'_s - 0.85\, f'_c) - A_s f_y \tag{11.6}$$

3. The eccentricity e_b is measured from the plastic centroid and e' is measured from the centroid of the tension steel: $e' = e + d''$ (in this case $e = eb + d''$), where d'' is the distance from the plastic centroid to the centroid of the tension steel. The value of e_b can be determined by taking moments about the plastic centroid.

$$P_b e_b = C_c\left(d - \frac{a}{2} - d''\right) + C_s(d - d' - d'') + Td'' \tag{11.7}$$

or

$$P_b e_b = M_b = 0.85 f'_c ab\left(d - \frac{a}{2} - d''\right) + A'_s(f_y - 0.85 f'_c)(d - d' - d'') + A_s f_y d'' \tag{11.8}$$

The balanced eccentricity is

$$e_b = \frac{M_b}{P_b} \tag{11.9}$$

For nonrectangular sections, the same procedure applies, taking into consideration the actual area of concrete in compression.

The strength reduction factor, ϕ, for the balanced condition with $f_y = 60$ ksi, can be assumed $= 0.65$ (or 0.75). This is because $\varepsilon_s = \varepsilon_t = f_y/E_s = 0.00207$ (or 0.002), for which $\phi = 0.65$ (Fig. 11.4).

Example 11.2

Determine the balanced compressive force P_b; then determine e_b and M_b for the section shown in Fig. 11.6. Given: $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. For a balanced condition, the strain in the concrete is 0.003 and the strain in the tension steel is

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.00207$$

2. Locate the neutral axis:

$$c_b = \frac{87}{87 + f_y} d_t = \frac{87}{87 + 60}(19.5) = 11.54 \text{ in.}$$

$$a_b = 0.85 c_b = 0.85 \times 11.54 = 9.81 \text{ in.}$$

3. Check if compression steel yields. From the strain diagram,

$$\frac{\varepsilon'_s}{0.003} = \frac{c - d'}{c} = \frac{11.54 - 2.5}{11.54} \qquad \varepsilon'_s = 0.00235$$

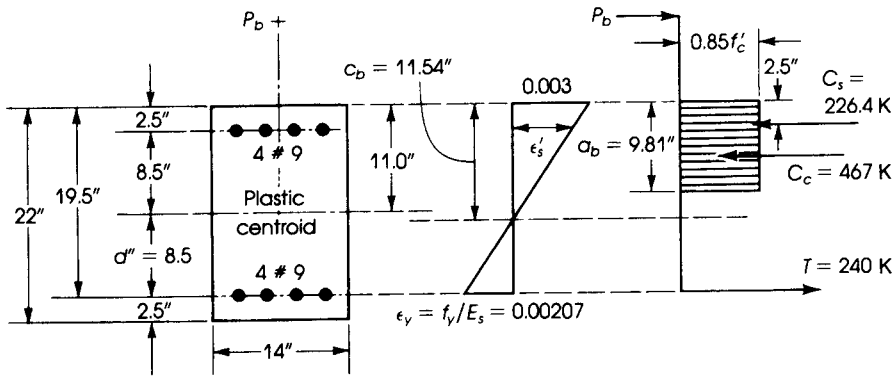


Figure 11.6 Example 11.2: balanced condition.

which exceeds ε_y of 0.00207; thus, compression steel yields. Or check that

$$f'_s = 87\left(\frac{c - d''}{c}\right) \leq f_y$$

$$f'_s = \frac{87(11.54 - 2.5)}{11.54} = 68 \text{ ksi} > 60 \text{ ksi}$$

Then $f'_s = f_y = 60$ ksi.

4. Calculate the forces acting on the section:

$$C_c = 0.85\, f'_c ab = 0.85 \times 4 \times 9.81 \times 14 = 467 \text{ K}$$

$$T = A_s f_y = 4 \times 60 = 240 \text{ K}$$

$$C_s = A'_s(f_y - 0.85\, f'_c) = 4(60 - 3.4) = 226.4 \text{ K}$$

5. Calculate P_b and e_b:

$$P_b = C_c + C_s - T = 467 + 226.4 - 240 = 453.4 \text{ K}$$

From Eq. 11.7,

$$M_b = P_b e_b = C_c\left(d - \frac{a}{2} - d''\right) + C_s(d - d' - d'') + Td''$$

The plastic centroid is at the centroid of the section, and d″ = 8.5 in.

$$M_b = 453.4 e_b = 467\left(19.5 - \frac{9.81}{2} - 8.5\right) + 226.4(19.5 - 2.5 - 8.5) + 240 \times 8.5$$

$$= 6810.8 \text{ K·in.} = 567.6 \text{ K·ft}$$

$$e_b = \frac{M_b}{P_b} = \frac{6810.8}{453.4} = 15.0 \text{ in.}$$

6. For a balanced condition, $\phi = 0.65$, $\phi P_b = 294.7$ K, and $\phi M_b = 368.9$ K·ft.

11.6 COLUMN SECTIONS UNDER ECCENTRIC LOADING

For the two cases when compression or tension failure occurs, two basic equations of equilibrium can be used in the analysis of columns under eccentric loadings: (1) the sum of the horizontal or vertical forces = 0, and (2) the sum of moments about any axis = 0. Referring to Fig. 11.7, the following equations may be established.

1. $$P_n - C_c - C_s + T = 0 \tag{11.10}$$

where

$C_c = 0.85\, f'_c ab$

$C_s = A'_s(f'_s - 0.85\, f'_c)$ (If compression steel yields, then $f'_s = f_y$.)

$T = A_s f_s$ (If tension steel yields, then $f_s = f_y$.)

2. Taking moments about A_s,

$$P_n e' - C_c\left(d - \frac{a}{2}\right) - C_s(d - d') = 0 \tag{11.11}$$



Reinforced concrete tied columns under construction. The two columns are separated by an expansion joint.

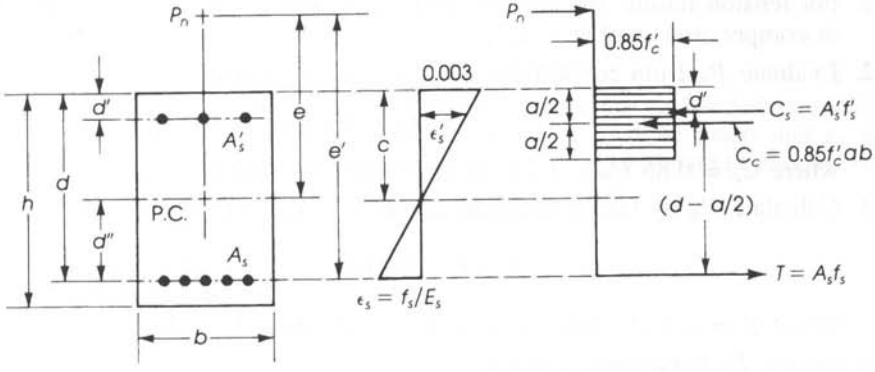


Figure 11.7 General case, rectangular section.

The quantity $e' = e + d''$, and $e' = (e + d - h/2)$ for symmetrical reinforcement (d'' is the distance from the plastic centroid to the centroid of the tension steel.)

$$P_n = \frac{1}{e'}\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right] \tag{11.12}$$

Taking moments about C_c,

$$P_n\left[e' - \left(d - \frac{a}{2}\right)\right] - T\left(d - \frac{a}{2}\right) - C_s\left(\frac{a}{2} - d'\right) = 0 \tag{11.13}$$

$$P_n = \frac{T\left(d - \frac{a}{2}\right) + C_s\left(\frac{a}{2} - d'\right)}{\left(e' + \frac{a}{2} - d\right)} \tag{11.14}$$

If $A_s = A'_s$ and $f_s = f'_s = f_y$, then

$$P_n = \frac{A_s f_y (d - d')}{\left(e' + \frac{a}{2} - d\right)} = \frac{A_s f_y (d - d')}{\left(e - \frac{h}{2} + \frac{a}{2}\right)} \tag{11.15}$$

$$A_s = A'_s = \frac{P_n \left(e - \frac{h}{2} + \frac{a}{2}\right)}{f_y (d - d')} \tag{11.16}$$

11.7 STRENGTH OF COLUMNS FOR TENSION FAILURE

When a column is subjected to an eccentric force with large eccentricity e, tension failure is expected. The column section fails due to the yielding of steel and crushing of concrete when the strain in the steel exceeds $\varepsilon_y (\varepsilon_y = f_y / E_s)$. In this case the nominal strength, P_n, will be less than P_b or the eccentricity, $e = M_n / P_n$, is greater than the balanced eccentricity, e_b. Because it is difficult in some cases to predict if tension or compression controls, it can be assumed (as a guide) a tension failure will occur when $e > d$. This assumption should be checked later.

The general equations of equilibrium, Eqs. 11.10 and 11.11, may be used to calculate the nominal strength of the column. This is illustrated in steps as follows:

1. For tension failure, the tension steel yields and its stress is $f_s = f_y$. Assume that stress in compression steel is $f'_s = f_y$.
2. Evaluate P_n from equilibrium conditions (Eq. 11.10):

$$P_n = C_c + C_s - T$$

where $C_c = 0.85 f'_c ab$, $C_s = A'_s (f_y - 0.85 f'_c)$, and $T = A_s f_y$.

3. Calculate P_n by taking moments about A_s (Eq. 11.11):

$$P_n \cdot e' = C_c \left(d - \frac{a}{2}\right) + C_s (d - d')$$

where e' $= e + d''$ and $e' = e + d - h/2$ when $A_s = A'_s$.

4. Equate P_n from steps 2 and 3:

$$C_c + C_s - T = \frac{1}{e'} \left[C_c \left(d - \frac{a}{2}\right) + C_s (d - d') \right]$$

This is a second-degree equation in a. Substitute the values of C_c, C_s, and T and solve for a.

5. The second-degree equation, after the substitution of C_c, C_s, and T, is reduced to the following equation:

$$Aa^2 + Ba + C = 0$$

where

$$A = 0.425 f'_c b$$
$$B = 0.85 f'_c b(e' - d) = 2A(e' - d)$$
$$C = A'_s (f'_s - 0.85 f'_c)(e' - d + d') - A_s f_y e'$$

Solve for a to get

$$a = \frac{-B \pm \sqrt{B^2 - 4AC}}{2A}$$

Note that the value of $(f'_s - 0.85\, f'_c)$ must be a positive value. If this value is negative, then let $(f'_s - 0.85\, f'_c) = 0$.

6. Substitute a in the equation of step 2 to obtain P_n. The moment M_n can be calculated:

$$M_n = P_n \cdot e$$

7. Check if compression steel yields as assumed. If $\varepsilon'_s \geq \varepsilon_y$, then compression steel yields; otherwise, $f'_s = E_s \varepsilon'_s$. Repeat steps 2 through 5. Note that $\varepsilon'_s = [(c - d'/c]\, 0.003$, $\varepsilon_y = f_y/E_s$ and $c = a\beta_1$.
8. Check that tension controls. Tension controls when $e > e_b$ or $P_n < P_b$. Example 11.3 illustrates this procedure.
9. The net tensile strain, ε_t, in this section, is normally greater than the limit strain of 0.002 for a compression-controlled section (Fig. 11.4). Consequently, the value of the strength reduction factor, ϕ, will vary between 0.65 (or 0.75) and 0.90. Equation 11.1 or 11.2 can be used to calculate ϕ.

Example 11.3

Determine the nominal compressive strength, P_n, for the section given in Example 11.2 if $e = 20$ in. (See Fig. 11.8.)

Solution

1. Because $e = 20$ in. is greater than $d = 19.5$ in., assume that tension failure condition controls (to be checked later). The strain in the tension steel, ε_s, will be greater than ε_y and its stress is f_y. Assume that compression steel yields $f'_s = f_y$, which should be checked later.
2. From the equation of equilibrium (Eq. 11.10),

$$P_n = C_c + C_s - T$$

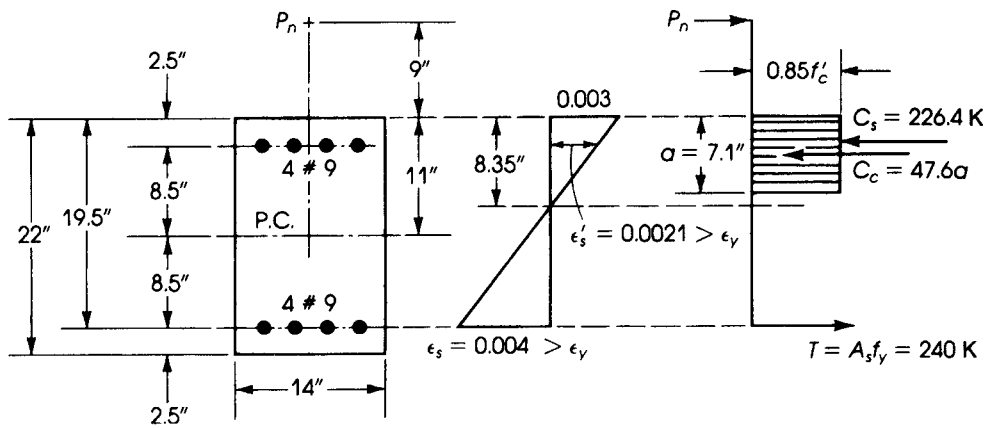


Figure 11.8 Example 11.3: tension failure.

where

$$C_c = 0.85 f'_c ab = 0.85 \times 4 \times 14a = 47.6a$$

$$C_s = A'_s(f_y - 0.85 f'_c) = 4(60 - 0.85 \times 4) = 226.4 \text{ K}$$

$$T = A_s f_y = 4 \times 60 = 240 \text{ K}$$

$$P_n = 47.6a + 226.4 - 240 = (47.6a - 13.6) \quad \text{(I)}$$

3. Taking moments about A_s (Eq. 11.12),

$$P_n = \frac{1}{e'}\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right]$$

Note that for the plastic centroid at the center of the section, $d'' = 8.5$ in.

$$e' = e + d'' = 20 + 8.5 = 28.5 \text{ in.}$$

$$P_n = \frac{1}{28.5}\left[47.6a\left(19.5 - \frac{a}{2}\right) + 226.4 \times 17\right]$$

$$P_n = 32.56a - 0.835a^2 + 135.0$$

4. Equating Eqs. I and II,

$$P_n = (47.6a - 13.6) = 32.56a - 0.835a^2 + 135.0 \quad \text{(II)}$$

or

$$a^2 + 18a - 178.0 = 0 \qquad a = 7.1 \text{ in.}$$

5. From Eq. I:

$$P_n = 47.6 \times 7.1 - 13.6 = 324.4 \text{ K}$$

$$M_n = P_n e = 324.4 \times \frac{20}{12} = 540.67 \text{ K·ft}$$

6. Check if compression steel has yielded:

$$c = \frac{a}{0.85} = \frac{7.1}{0.85} = 8.35 \text{ in.} \qquad \varepsilon_y = \frac{60}{29{,}000} = 0.00207$$

$$\varepsilon'_s = \frac{(8.35 - 2.5)}{8.35}(0.003) = 0.0021 > \varepsilon_y$$

Compression steel yields. Check strain in tension steel:

$$\varepsilon_s = \left(\frac{19.5 - 8.35}{8.35}\right) \times 0.003 = 0.004 > \varepsilon_y$$

If compression steel does not yield, use f'_s as calculated from $f'_s = \varepsilon'_s E_s$ and revise the calculations.

7. Calculate ϕ: Since $\varepsilon_t = 0.004$, the section is in the transition region.

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) = 0.817$$

$$\phi P_n = 0.817(324.4) = 264.9 \text{ K}$$

$$\phi M_n = 0.817(540.67) = 441.7 \text{ K·ft}$$

8. Because $e = 20\,\text{in.} > e_b = 15\,\text{in.}$ (Example 11.2), there is a tension failure condition.
9. The same results can be obtained using the values of A, B, and C given earlier.

$$Aa^2 + Ba + C = 0$$

$$A = 0.425 f'_c b = 0.425(4)(14) = 23.8$$

$$B = 2A(e' - d) = 2(23.8)(28.5 - 19.5) = 428.4$$

$$C = 4(60 - 0.85 \times 4)(28.5 - 19.5 + 2.5) - 4(60)(28.5)$$

$$= -4236.4$$

Solve for a to get $a = 7.1\,\text{in.}$ and $P_n = 324.4\,\text{K}$.

11.8 STRENGTH OF COLUMNS FOR COMPRESSION FAILURE

If the compressive applied force, P_n, exceeds the balanced force, P_b, or the eccentricity, $e = M_n/P_n$, is less than e_b, compression failure is expected. In this case compression controls, and the strain in the concrete will reach 0.003, whereas the strain in the steel is less than ε_y (Fig. 11.9). A large part of the column will be in compression. The neutral axis moves toward the tension steel, increasing the compression area, and therefore the distance to the neutral axis c is greater than the balanced c_b (Fig. 11.9).

Because it is difficult to predict compression or tension failure whenever a section is given, compression failure can be assumed when $e < 2d/3$, which should be checked later. The nominal load strength, P_n, can be calculated using the principles of statics. The analysis of column sections for compression failure can be achieved using Eqs. 11.10 and 11.11 given earlier and one of the following solutions.

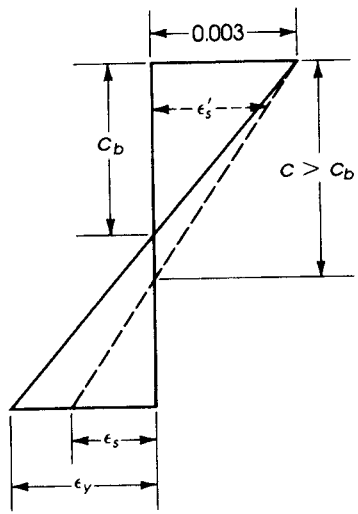


Figure 11.9 Strain diagram when compression controls. When $\varepsilon_s < \varepsilon_y$, $c > c_b$ and $\varepsilon'_s \geq \varepsilon_y$.

11.8.1 Trial Solution

This solution can be summarized as follows:

1. Calculate the distance to the neutral axis for a balanced section, c_b:

$$c_b = \left(\frac{87 d_t}{87 + f_y}\right) \tag{11.17}$$

where f_y is in ksi.

2. Evaluate P_n using equilibrium conditions:

$$P_n = C_c + C_s - T \tag{11.18}$$

3. Evaluate P_n by taking moments about the tension steel, A_s:

$$P_n \cdot e' = C_c\left(d - \frac{a}{2}\right) + C_s(d - d') \tag{11.19}$$

where $e' = e + d - h/2$ when $A_s = A_s'$ or $e' = e + d''$ in general, $C_c = 0.85 f_c' ab$, $C_s = A_s'(f_s' - 0.85 f_c')$, and $T = A_s f_s$.

4. Assume a value for c such that $c > c_b$ (calculated in step 1). Calculate $a = \beta_1 c$. Assume $f_s' = f_y$.

5. Calculate f_s based on the assumed c:

$$f_s = \varepsilon_s E_s = 87\left(\frac{d_t - c}{c}\right) \text{ ksi} \leq f_y$$

6. Substitute the preceding values in Eq. 11.10 to calculate P_{n1} and in Eq. 11.11 to calculate P_{n2}. If P_{n1} is close to P_{n2}, then choose the smaller or average of P_{n1} and P_{n2}. If P_{n1} is not close to P_{n2}, assume a new c or a and repeat the calculations starting from step 4 until P_{n1} is close to P_{n2}. (1% is quite reasonable.)

7. Check that compression steel yields by calculating $\varepsilon_s' = 0.003[(c - d')/c]$ and comparing it with $\varepsilon_y = f_y/E_s$. When $\varepsilon_s' \geq \varepsilon_y$, compression steel yields; otherwise, $f_s' = \varepsilon_s' E_s$ or, directly,

$$f_s' = 87\left(\frac{c - d'}{c}\right) \leq f_y \text{ ksi}$$

8. Check that $e < e_b$ or $P_n > P_b$ for compression failure. Example 11.4 illustrates the procedure.

9. The net tensile strain, ε_t, in the section is normally less than 0.002 for compression-controlled sections (Fig. 11.4). Consequently, the strength reduction factor (ϕ) = 0.65 (or 0.70 for spiral columns).

Example 11.4

Determine the nominal compressive strength, P_n, for the section given in Example 11.2 if $e = 10$ in. (See Fig. 11.10.)

Solution

1. Because $e = 10$ in. $< (2/3)d = 13$ in., assume compression failure. This assumption will be checked later. Calculate the distance to the neutral axis for a balanced section, c_b:

$$c_b = \frac{87}{87 + f_y} d_t = \frac{87}{87 + 60}(19.5) = 11.54 \text{ in.}$$

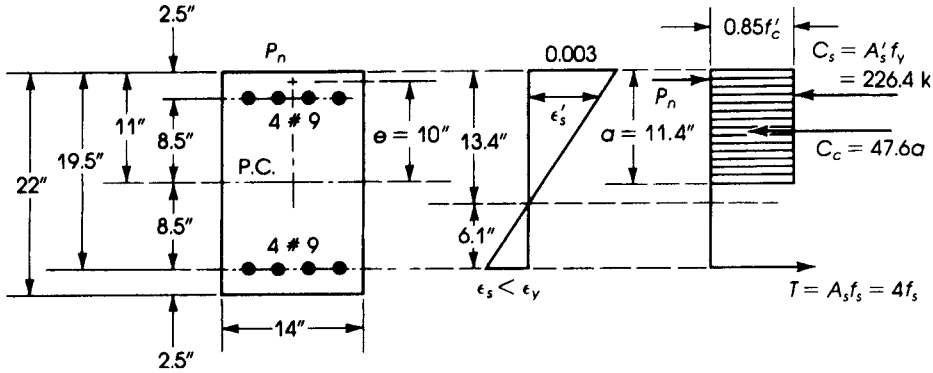


Figure 11.10 Example 11.4: compression controls.

2. From the equations of equilibrium,

$$P_n = C_c + C_s - T \tag{11.10}$$

where

$$C_c = 0.85\, f'_c ab = 0.85 \times 4 \times 14a = 47.6a$$

$$C_s = A'_s(f_y - 0.85\, f'_c) = 4(60 - 0.85 \times 4) = 226.4 \text{ K}$$

Assume compression steel yields. (This assumption will be checked later.)

$$T = A_s f_s = 4f_s \qquad (f_s < f_y) \tag{I}$$

$$P_n = 47.6a + 226.4 - 4f_s$$

3. Taking moments about A_s,

$$P_n = \frac{1}{e'}\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right] \tag{11.11}$$

The plastic centroid is at the center of the section and $d'' = 8.5$ in.

$$e' = e + d'' = 10 + 8.5 = 18.5 \text{ in.}$$

$$P_n = \frac{1}{18.5}\left[47.6a\left(19.5 - \frac{a}{2}\right) + 226.4(19.5 - 2.5)\right] \tag{II}$$

$$P_n = 50.17a - 1.29a^2 + 208$$

4. Assume $c = 13.45$ in., which exceeds c_b (11.54 in.).

$$a = 0.85 \times 13.45 = 11.43 \text{ in.}$$

Substitute $a = 11.43$ in Eq. II:

$$P_{n1} = 50.17 \times 11.43 - 1.29(11.43)^2 + 208 = 612.9 \text{ K}$$

5. Calculate f_s from the strain diagram when $c = 13.45$ in.

$$f_s = \left(\frac{19.5 - 13.45}{13.45}\right)87 = 39.13 \text{ ksi} \quad \varepsilon_s = \varepsilon_t = f_s/E_s = 0.00135$$

6. Substitute $a = 11.43$ in. and $f_s = 39.13$ ksi in Eq. I to calculate P_{n2}:

$$P_{n2} = 47.6(11.43) + 226.4 - 4(39.13) = 613.9 \text{ K}$$

which is very close to the calculated P_{n1} of 612.9 K (less than 1% difference).

$$M_n = P_n \cdot e = 612.9\left(\frac{10}{12}\right) = 510.8 \text{ K·ft}$$

7. Check if compression steel yields. From the strain diagram,

$$\varepsilon_s' = \frac{13.45 - 2.5}{13.45}(0.003) = 0.00244 > \varepsilon_y = 0.00207$$

Compression steel yields, as assumed.

8. $P_n = 612.9$ K is greater than $P_b = 453.4$ K, and $e = 10$ in. $< e_b = 15$ in., both calculated in the previous example, indicating that compression controls, as assumed. Note that it may take a few trials to get P_{n1} close to P_{n2}.

9. Calculate ϕ:

$$d_t = d = 19.5 \text{ in.} \qquad c = 13.45 \text{ in.}$$

ε_t (at the tension steel level) $= 0.003(d_t - c)/c$.

$$\varepsilon_t = 0.003(19.5 - 13.45)/13.45 = 0.00135$$

Since $\varepsilon_t < 0.002$, then $\phi = 0.65$.

$$\phi P_n = 0.65(612.9) = 398.4 \text{ K}$$

$$\phi M_n = 0.65(510.8) = 332 \text{ K·ft.}$$

11.8.2 Numerical Analysis Solution

The analysis of columns when compression controls can also be performed by reducing the calculations into one cubic equation in the form

$$Aa^3 + Ba^2 + Ca + D = 0$$

and then solving for a by a numerical method, or a can be obtained directly by using one of many inexpensive scientific calculators with built-in programs that are available. From the equations of equilibrium,

$$\begin{aligned} P_n &= C_c + C_s - T \\ &= (0.85 f_c' ab) + A_s'(f_y - 0.85 f_c') - A_s f_s \end{aligned} \tag{11.10}$$

Taking moments about the tension steel, A_s,

$$\begin{aligned} P_n &= \frac{1}{e'}\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right] \\ &= \frac{1}{e'}\left[0.85 f_c' ab\left(d - \frac{a}{2}\right) + A_s'(f_y - 0.85 f_c')(d - d')\right] \end{aligned} \tag{11.11}$$

From the strain diagram,

$$\varepsilon_s = \left(\frac{d_t - c}{c}\right)(0.003) = \frac{\left(d - \dfrac{a}{\beta_1}\right)}{\dfrac{a}{\beta_1}}(0.003)$$

The stress in the tension steel is

$$f_s = \varepsilon_s E_s = 29{,}000\varepsilon_s = \frac{87}{a}(\beta_1 d - a)$$

Substituting this value of f_s in Eq. 11.10 and equating Eqs. 11.10 and 11.11 and simplifying gives

$$\left(\frac{0.85\,f'_c b}{2}\right)a^3 + [0.85 f'_c b(e' - d)]a^2 + [A'_s(f_y - 0.85\,f'_c)(e' - d + d') + 87A_s e']a - 87\,A_s e'\beta_1 d = 0$$

This is a cubic equation in terms of a:

$$Aa^3 + Ba^2 + Ca + D = 0$$

where

$$A = \frac{0.85\,f'_c b}{2}$$
$$B = 0.85\,f'_c b(e' - d)$$
$$C = A'_s(f_y - 0.85\,f'_c)(e' - d + d') + 87\,A_s e'$$
$$D = -87\,A_s e'\beta_1 d$$

Once the values of A, B, C, and D are calculated, a can be determined by trial or directly by a scientific calculator. Also, the solution of the cubic equation can be obtained by using the well known Newton-Raphson method. This method is very powerful for finding a root of $f(x) = 0$. It involves a simple technique, and the solution converges rapidly by using the following steps:

1. Let $f(a) = Aa^3 + Ba^2 + Ca + D$, and calculate A, B, C, and D.
2. Calculate the first derivative of $f(a)$:
$$f'(a) = 3Aa^2 + 2Ba + C$$
3. Assume any initial value of a, say, a_0, and compute the next value:
$$a_1 = a_0 - \frac{f(a_0)}{f'(a_0)}$$
4. Use the obtained value a_1 in the same way to get
$$a_2 = a_1 - \frac{f(a_1)}{f'(a_1)}$$
5. Repeat the same steps to get the answer up to the desired accuracy. In the case of the analysis of columns when compression controls, the value a is greater than the balanced $a(a_b)$. Therefore, start with $a_0 = a_b$ and repeat twice to get reasonable results.

Example 11.5

Repeat Example 11.4 using numerical solution.

Solution

1. Calculate A, B, C, and D and determine $f(a)$.

$$A = 0.85 \times 4 \times \frac{14}{2} = 23.8$$
$$B = 0.85 \times 4 \times 14(18.5 - 19.5) = -47.6$$

$$C = 4(60 - 0.85 \times 4)(18.5 - 19.5 + 2.5) + 87 \times 4 \times 18.5$$
$$= 6777.6$$
$$D = -87 \times 4 \times 18.5 \times (0.85 \times 19.5) = -106{,}710$$
$$f(a) = 23.8a^3 - 47.6a^2 + 6777.6a - 106{,}710$$

2. Calculate the first derivative:

$$f'(a) = 71.4a^2 - 95.2a + 6777.6$$

3. Let $a_0 = a_b = 9.81$ in. For a balanced section, $c_b = 11.54$ in. and $a_b = 9.81$ in.

$$a_1 = 9.81 - \frac{f(9.81)}{f'(9.81)} = 9.81 - \frac{-22{,}334}{12{,}715} = 11.566 \text{ in.}$$

4. Calculate a_2:

$$a_2 = 11.566 - \frac{f(11.566)}{f'(11.566)} = 11.566 - \frac{2136}{15{,}228} = 11.43 \text{ in.}$$

This value of a is similar to that obtained earlier in Example 11.3. Substitute the value of a in Eq. 11.10 or 11.11 to get $P_n = 612.9$ K.

11.8.3 Approximate Solution

An approximate equation was suggested by Whitney to estimate the nominal compressive strength of short columns when compression controls, as follows [15]:

$$P_n = \frac{bhf'_c}{\frac{3he}{d^2} + 1.18} + \frac{A'_s f_y}{\frac{e}{(d - d')} + 0.5} \tag{11.17}$$

This equation can be used only when the reinforcement is symmetrically placed in single layers parallel to the axis of bending.

A second approximate equation was suggested by Hsu [16]:

$$\frac{P_n - P_b}{P_o - P_b} + \left(\frac{M_n}{M_b}\right)^{1.5} = 1.0 \tag{11.18}$$

where

P_n = nominal axial strength of the column section
P_b, M_b = nominal load and moment of the balanced section
M_n = nominal bending moment = $P_n \cdot e$
P_o = nominal axial load at $e = 0$
= $0.85 f'_c(A_g - A_{st}) + A_{st} f_y$
A_g = gross area of the section = bh
A_{st} = total area of nonprestressed longitudinal reinforcement

Example 11.6

Determine the nominal compressive strength, P_n, for the section given in Example 11.4 by Eqs. 11.17 and 11.18 using the same eccentricity, $e = 10$ in., and compare results.

Solution

1. Solution by Whitney equation (Eq. 11.29):
   a. Properties of the section shown in Fig. 11.10 are $b = 14$ in., $h = 22$ in., $d = 19.5$ in., $d' = 2.5$ in., $A'_s = 4.0$ in.^{2}, and $(d - d') = 17$ in.

b. Apply the Whitney equation:

$$P_n = \frac{14 \times 22 \times 4}{(3 \times 22 \times 10)/(19.5)^2 + 1.18} = \frac{4 \times 60}{\left(\frac{10}{17}\right) + 0.5} = 643 \text{ K}$$

$$\phi P_n = 0.65 P_n = 418 \text{ K}$$

c. P_n calculated by the Whitney equation is not a conservative value in this example, and the value of $P_n = 643$ K is greater than the more accurate value of 612.9 K calculated by statics in Example 11.4.

2. Solution by Hsu equation (Eq. 11.18):

a. For a balanced condition, $P_b = 453.4$ K and $M_b = 6810.8$ K·in. (Example 11.2).

b. $P_0 = 0.85 f'_c(A_g - A_{st}) + A_{st} f$
$= 0.85(4)(14 \times 22 - 8) + 8(60) = 1500$ K

c. $\dfrac{P_n - 453.4}{1500 - 453.4} + \left(\dfrac{10P_n}{6810.8}\right)^{1.5} = 1$

Multiply by 1000 and solve for P_n.

$$0.9555 P_n + 0.05626 P_n^{1.5} = 1433.2 \text{ K}$$

By trial, $P_n = 611$ K, which is very close to 612.9 K, as calculated by statics.

11.9 INTERACTION DIAGRAM EXAMPLE

In Example 11.2, the balanced loads P_b, M_b, and e_b were calculated for the section shown in Fig. 11.6 ($e_b = 15$ in.). Also, in Examples 11.3 and 11.4, the load capacity of the same section was calculated for the case when $e = 20$ in. (tension failure) and when $e = 10$ in. (compression failure). These values are shown in Table 11.1.

To plot the load–moment interaction diagram, different values of ϕP_n and ϕM_n were calculated for various e values that varied between $e = 0$ and $e =$ maximum for the case of

Table 11.1 Summary of the Load Strength of the Column Section in the Previous Examples

e (in.)	*a* (in.)	ϕ	P_n (K)	ϕP_n (K)	ϕM_n (K·ft)	Notes
0	—	0.65	1500	975	0.0	ϕP_{n0}
2.25	19.39	0.65	1200	780	146.3	0.8 ϕP_{n0}
4	16.82	0.65	1018	661.7	220.6	Compression
6	14.19	0.65	843.3	548.1	274.0	Compression
10*	11.43	0.65	612.9	398.4	332.0	Compression
12	10.63	0.65	538.0	349.7	349.7	Compression
15*	9.81	0.65	453.4	294.7	368.9	Balanced
20*	7.10	0.81	324.4	263.4	439.0	Transition
30	5.06	0.90	189.4	170.5	426.2	Tension
50	4.01	0.90	100.6	90.5	377.2	Tension
80	3.59	0.90	58.8	52.9	352.0	Tension
P.M.	3.08	0.90	0.0	0.0	352.0	Tension
P.M.	3.08	0.65	0.0	0.0	254.2	P.M. (X)

*= values calculated in Examples 11.2,11.3, and 11.4.
P.M. = pure moment.
X = Not applicable, for comparison only.

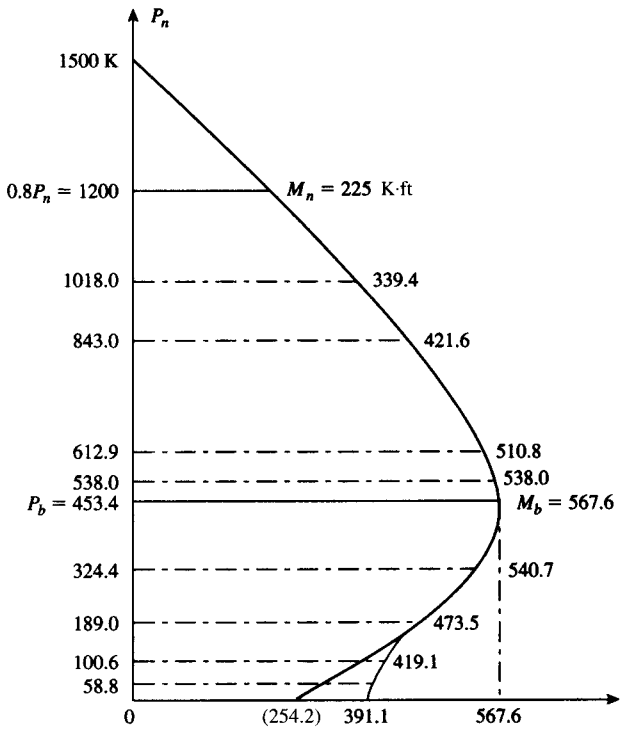


Figure 11.11 Interaction diagram of the column section shown in Fig. 11.10.

pure moment when $P_n = 0$. These values are shown in Table 11.1. The interaction diagram is shown in Fig. 11.11. The load $\phi P_{n0} = 975$ K represents the theoretical axial load when $e = 0$, whereas $0.8\ \phi P_{n0} = 780$ K represents the maximum axial load allowed by the ACI Code based on minimum eccentricity. Note that for compression failure, $e < e_b$ and $P_n > P_b$, and for tension failure, $e > e_b$ and $P_n < P_b$. The last two cases in the table represent the pure moment (P.M.) or beam-action case for $\phi = 0.9$ and $\phi = 0.65$ ($M_n = 391$ K·ft). To be consistent with the design of beams due to bending moments, the ACI Code allows the use of $\phi = 0.9$ with pure moment, so $\phi M_n = 352$ K·ft instead of 254.2 K·ft. Also note that ϕ varies between 0.65 and 0.9 according to Eq. 11.2 for tied columns. Note that $M_n = 391.1$ K·ft.

11.10 RECTANGULAR COLUMNS WITH SIDE BARS

In some column sections, the steel reinforcement bars are distributed around the four sides of the column section. The side bars are those placed on the sides along the depth of the section in addition to the tension and compression steel, A_s and A'_s, and can be denoted by A_{ss} (Fig. 11.12). In this case the same procedure explained earlier can be applied, taking into consideration the strain variation along the depth of the section and the relative force in each side bar either in the compression or tension zone of the section. These are added to those of C_c, C_s, and T to determine P_n Equation 11.10 becomes

$$P_n = C_c + \sum C_s - \Sigma T \tag{11.10a}$$

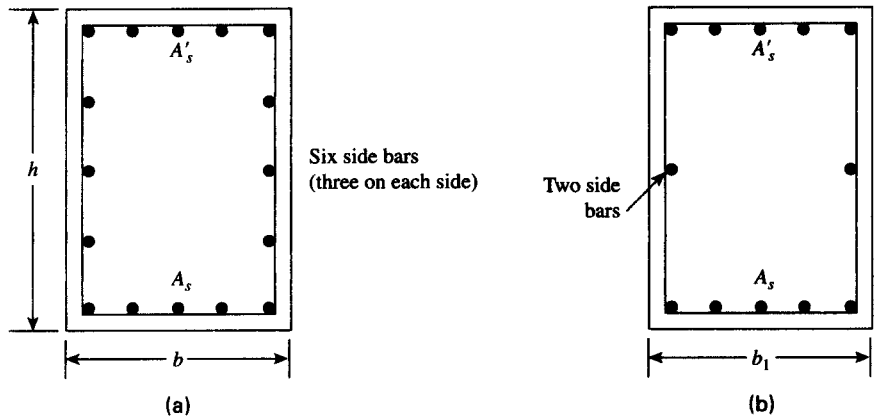


Figure 11.12 Side bars in rectangular sections: (*a*) six side bars and (*b*) two side bars (may be neglected).

Example 11.7 explains this analysis. Note that if the side bars are located near the neutral axis (Fig. 11.12*b*), the strains—and, consequently, the forces—in these bars are very small and can be neglected. Those bars close to A_s and A'_s have appreciable force and increase the load capacity of the section.

Example 11.7

Determine the balanced load, P_b moment, M_b, and e_b for the section shown in Fig. 11.13. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

The balanced section is similar to Example 11.2. Given: $b = h = 22$ in., $d = 19.5$ in., $d' = 2.5$ in., $A_s = A'_s = 6.35$ in^2. (five no. 10 bars), and six no. 10 side bars (three on each side).

1. Calculate the distance to the neutral axis:

$$c_b = \left(\frac{87}{87 + f_y}\right) d_t = \left(\frac{87}{87 + 60}\right) 19.5 = 11.54 \text{ in.}$$

$$a_b = 0.85(11.54) = 9.81 \text{ in.}$$

2. Calculate the forces in concrete and steel bars; refer to Fig. 11.13*a*. In the compression zone, $C_c = 0.85 f'_c ab = 0.85(4)(9.81)(22) = 733.8$ K.

$$f'_s = 87\left(\frac{c - d'}{c}\right) = 87\left(\frac{11.54 - 2.5}{11.54}\right) = 68.15 \text{ ksi} > 60 \text{ ksi}$$

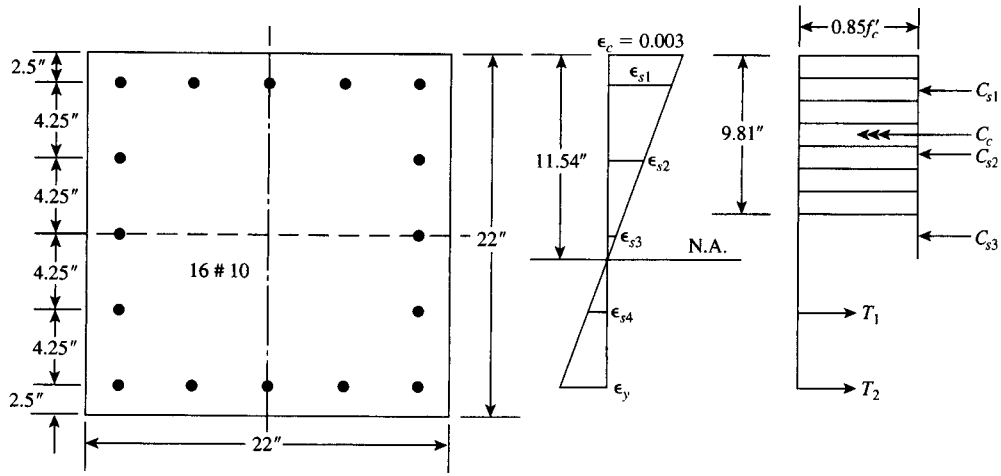
Then $f'_s = 60$ ksi.

$$C_{s1} = A'_s(f_y - 0.85\, f'_c) = 6.35(60 - 0.85 \times 4) = 359.4 \text{ K}$$

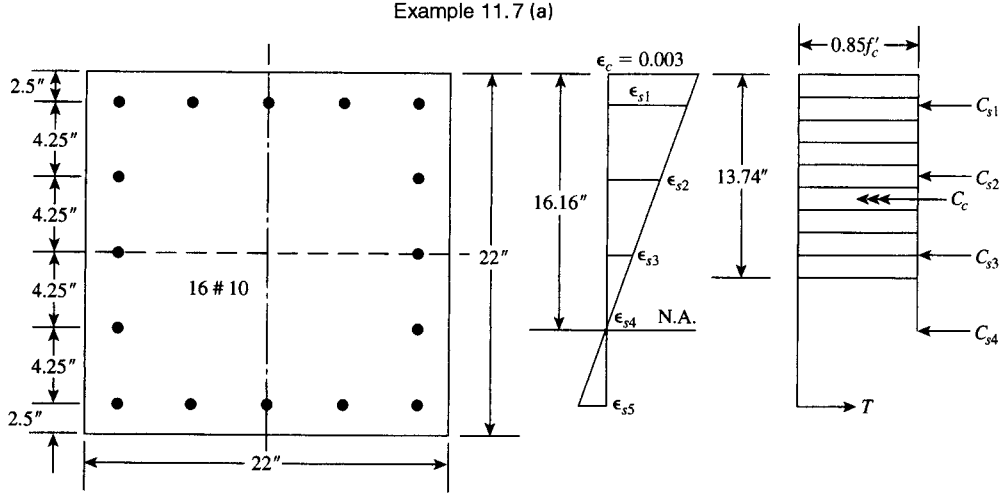
$$f_{s2} = 87\left(\frac{11.54 - 2.5 - 4.25}{11.54}\right) = 36.11 \text{ ksi}$$

$$C_{s2} = 2(1.27)(36.11 - 0.85 \times 4) = 83.1 \text{ K}$$

Similarly, $f_{s3} = 4.07$ ksi and $C_{s3} = 2(1.27)(4.07 - 0.85 \times 4) = 1.7$ K.



Example 11.7 (a)



Example 11.8 (b)

Figure 11.13 Example 11.7: (*a*) balanced section. Example 11.8: (*b*) for compression failure, $e = 6$ in.

In the tension zone,

$$\varepsilon_{s4} = 964.50 \times 10^{-6} \qquad f_{s4} = 28 \text{ ksi}$$

$$T_1 = 2(1.27)(28) = 71 \text{ K}$$

$$T_2 = A_s f_y = 6.35(60) = 381 \text{ K}$$

3. Calculate $P_b = C_c + \Sigma C_s - \Sigma T$.

$$P_b = 733.8 + (359.4 + 83.1 + 1.7) - (71 + 381)$$

$$= 726 \text{ K}$$

4. Taking moments about the plastic centroid,

$$M_b = 733.8(6.095) + 359.4(8.5) + 83.1(4.25) + 71(4.25) + 381(8.5)$$
$$= 11{,}421 \text{ K·in.} = 952 \text{ K·ft}$$
$$e_b = \frac{M_b}{P_b} = 15.735 \text{ in.}$$

5. Determine ϕ: For a balanced section, $\varepsilon_t = \varepsilon_y = 0.002$, $\phi = 0.65$,

$$\phi P_b = 0.65\, P_b = 472 \text{ K, and } \phi M_b = 0.65\, M_b = 618.8 \text{ K·ft.}$$

Example 11.8

Repeat the previous example when $e = 6.0$ in.

Solution

1. Because $e = 6$ in. $< e_b = 15.735$ in., this is a compression failure condition. Assume $c = 16.16$ in. (by trial) and $a = 0.85(16.16) = 13.74$ in. (Fig. 11.13*b*).
2. Calculate the forces in concrete and steel bars:

$$C_c = 0.85(4)(13.74)(22) = 1027.75 \text{ K}$$

In a similar approach to the balanced case, $f_{s1} = 60$ ksi and $C_{s1} = 359.41$.

$$f_{s2} = 50.66 \text{ ksi} \quad C_{s2} = 120.0 \text{ K}$$
$$f_{s3} = 27.78 \text{ ksi} \quad C_{s3} = 61.92 \text{ K}$$
$$f_{s4} = 4.9 \text{ ksi} \quad C_{s4} = 3.81 \text{ K}$$
$$f_{s5} = 18 \text{ ksi} \quad T = 6.35(18) = 114.2 \text{ K}$$

3. Calculate $P_n = C_c + \Sigma C_s - \Sigma T = 1458.7$ K.

$$M_n = P_n \cdot e = 729.35 \text{ K·ft} \quad (e = 6 \text{ in.})$$

4. Check P_n by taking moments about A_s,

$$P_n = \frac{1}{e'}\left[C_c\left(d - \frac{a}{2}\right) + C_{s1}(d - d') + C_{s2}(d - d' - s) + C_{s3}(d - d' - 2s) + C_{s4}(d - d' - 3s)\right]$$

$$e' = e + d - \frac{h}{2} = 6 + 19.5 - \frac{22}{2} = 14.5 \text{ in.}$$

s = distance between side bars

$= 4.25$ in. (s = constant in this example.)

$$P_n = \frac{1}{14.5}\left[1027.75\left(19.5 - \frac{13.74}{2}\right) + 359.41(17) + 120(17 - 4.25) + 61.92(17 - 8.5) + 3.81(17 - 12.75)\right] = 1459 \text{ K}$$

5. Calculate ϕ:

$$d_t = d = 19.5 \text{ in.} \quad c = 16.16 \text{ in.}$$

ε_t (at the tension steel level) $= 0.003(d_t - c)/c$

$$\varepsilon_t = 0.003(19.5 - 16.16)/16.16 = 0.00062$$

Since $\varepsilon_t < 0.002$, then $\phi = 0.65$.

$$\phi P_n = 0.65(1459) = 948.3 \text{ K}$$

$$\phi M_n = 0.65(729.5) = 474 \text{ K·ft}$$

Note: If side bars are neglected, then

$$P_b = 733.8 + 359.4 - 381 = 712.2 \text{ K}$$

$$P_n \text{ (at } e = 6 \text{ in.)} = 1027.75 + 359.4 - 114.2 = 1273 \text{ K}$$

If side bars are considered, the increase in P_b is about 2% and that in P_n is about 14.6%.

11.11 LOAD CAPACITY OF CIRCULAR COLUMNS

11.11.1 Balanced Condition

The values of the balanced load P_b and the balanced moment M_b for circular sections can be determined using the equations of equilibrium, as was done in the case of rectangular sections. The bars in a circular section are arranged in such a way that their distance from the axis of plastic centroid varies, depending on the number of bars in the section. The main problem is to find the depth of the compressive block a and the stresses in the reinforcing bars. The following example explains the analysis of circular sections under balanced conditions. A similar procedure can be adopted to analyze sections when tension or compression controls.

Example 11.9

Determine the balanced load P_b and the balanced moment M_b for the 16-in. diameter circular spiral column reinforced with eight no. 9 bars shown in Fig. 11.14. Given: $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. Because the reinforcement bars are symmetrical about the axis A-A passing through the center of the circle, the plastic centroid lies on that axis.
2. Determine the location of the neutral axis:

$$d_t = 13.1 \text{ in.} \qquad \varepsilon_y = \frac{f_y}{E_s} \qquad (E_s = 29{,}000 \text{ ksi})$$

$$\frac{c_b}{d_t} = \frac{0.003}{0.003 + \varepsilon_y} = \frac{0.003}{0.003 + \frac{f_y}{E_s}} = \frac{87}{87 + f_y}$$

$$c_b = \frac{87}{87 + 60}(13.1) = 7.75 \text{ in.}$$

$$a_b = 0.85 \times 7.75 = 6.59 \text{ in.}$$

3. Calculate the properties of a circular segment (Fig. 11.15):

$$\text{Area of segment} = r^2(\alpha - \sin \alpha \cos \alpha) \tag{11.19}$$

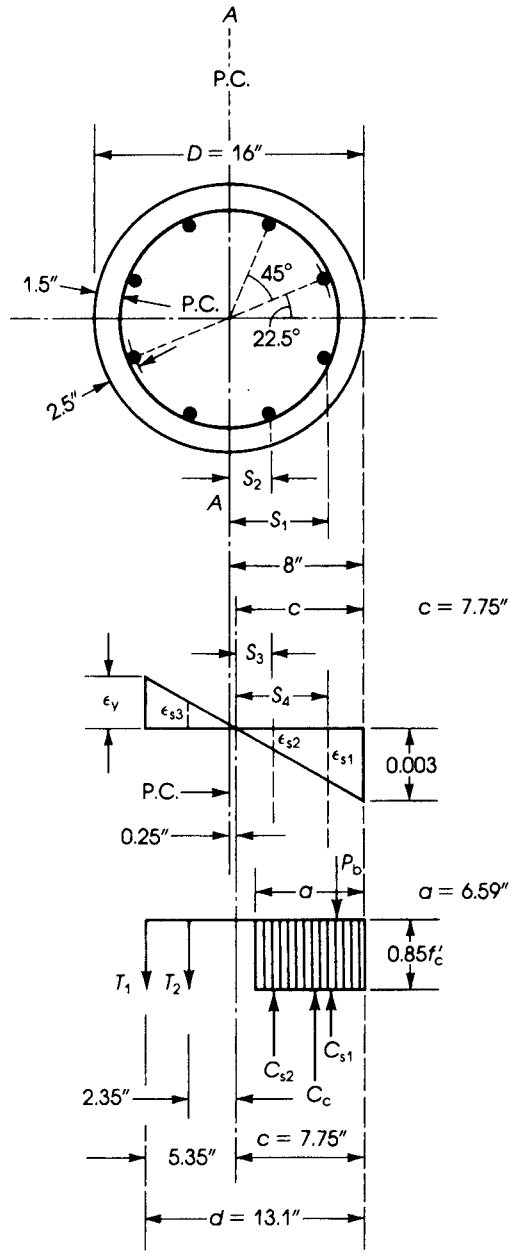


Figure 11.14 Example 11.9: eight no. 9 bars

$S = 8 - 2.5 = 5.5$ in.
$S_1 = S\cos 22.5° = 5.1$ in.
$S_2 = S\cos 67.5° = 2.1$ in.
$d = 8 + 5.1 = 13.1$ in.
$S_3 = 1.85$ in.
$S_4 = 4.85$ in.

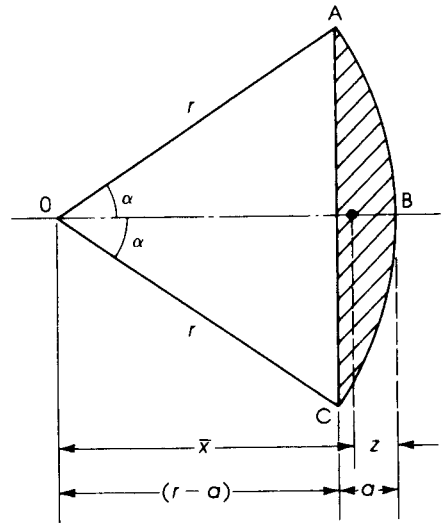


Figure 11.15 Example 11.9: Properties of circular segments.

Location of centroid $\bar{x}$ (from the circle center 0):

$$\bar{x} = \frac{2}{3}\frac{(r \sin^3 \alpha)}{(\alpha - \sin \alpha \cos \alpha)} \tag{11.20}$$

$$Z = r - \bar{x} \tag{11.21}$$

$$r \cos \alpha = (r - a) \quad \text{or} \quad \cos \alpha = \left(1 - \frac{a}{r}\right) \tag{11.22}$$

$$\cos \alpha = \left(1 - \frac{6.59}{8}\right) = 0.176$$

and $\alpha = 79.85°$, $\sin \alpha = 0.984$, and $\alpha = 1.394$ rad.

$$\text{Area for segment} = (8)^2(1.394 - 0.984 \times 0.176)$$

$$= 78.12 \text{ in.}^2$$

$$\bar{x} = \left(\frac{2}{3}\right)\frac{8(0.984)^3}{(1.394 - 0.984 \times 0.176)} = 4.16 \text{ in.}$$

$$Z = r - \bar{x} = 8 - 4.16 = 3.84 \text{ in.}$$

4. Calculate the compressive force C_c:

$$C_c = 0.85 f_c' \times \text{area of segment}$$

$$= 0.85 \times 4 \times 78.12 = 265.6 \text{ K}$$

It acts at 4.16 in. from the center of the column.

5. Calculate the strains, stresses, and forces in the tension and the compression steel. Determine the strains from the strain diagram. For T_1,

$$\varepsilon = e_y = 0.00207 \qquad f_s = f_y = 60 \text{ ksi}$$

$$T_1 = 2 \times 60 = 120 \text{ K}$$

For T_2,

$$\varepsilon_{s3} = \frac{2.35}{5.35}\varepsilon_y = \frac{2.35}{5.35} \times 0.00207 = 0.00091$$

$$f_{s3} = 0.00091 \times 29{,}000 = 26.4 \text{ ksi}$$

$$T_2 = 26.4 \times 2 = 52.8 \text{ K}$$

For C_{s1},

$$\varepsilon_{s1} = \frac{4.85}{7.75} \times 0.003 = 0.00188$$

$$f_{s1} = 0.00188 \times 29{,}000 = 54.5 \text{ ksi} < 60 \text{ ksi}$$

$$C_{s1} = 2(54.5 - 3.4) = 102.2 \text{ K}$$

For C_{s2},

$$\varepsilon_{s2} = \frac{1.85}{7.75} \times 0.003 = 0.000716$$

$$f_{s2} = 0.000716 \times 29{,}000 = 20.8 \text{ ksi}$$

$$C_{s2} = 2(20.8 - 3.4) = 34.8 \text{ K}$$

The stresses in the compression steel have been reduced to take into account the concrete displaced by the steel bars.

6. The balanced force is $P_b = C_c + \Sigma C_s - \Sigma T$ ($\phi = 0.75$).

$$P_b = 265.6 + (102.2 + 34.8) - (120 + 52.8) = 230 \text{ K}$$

For a balanced section,

$$\varepsilon_t = 0.002 \quad \text{and} \quad \phi = 0.65$$

$$\phi P_b = 149.5 \text{ K}$$

7. Take moments about the plastic centroid (axis A-A through the center of the section) for all forces:

$$M_b = P_b e_b = [C_c \times 4.16 + C_{s1} \times 5.1 + C_{s2} \times 2.1 + T_1 \times 5.1 + T_2 \times 2.1]$$

$$= 2422.1 \text{ K·in.} = 201.9 \text{ K·ft}$$

$$\phi M_b = 131.2 \text{ K·ft}$$

$$e_b = \frac{2422.1}{230} = 10.5 \text{ in.}$$

11.11.2 Strength of Circular Columns for Compression Failure

A circular column section under eccentric load can be analyzed in similar steps as the balanced section. This is achieved by assuming a value for $c > c_b$ or $a > a_b$ and calculating the forces in concrete and steel at different locations to determine P_{n1} $P_{n1} = C_c + \Sigma C_s - \Sigma T$. Also, M_n can be calculated by taking moments about the plastic centroid (center of the section) and determining $P_{n2} = M_n/e$. If they are not close enough, within about 1%, assume a new c or a and repeat the calculations. (See also Section 11.8.) Compression controls when $e < e_b$ or $P_n > P_b$.

For example, if it is required to determine the load capacity of the column section of Example 11.9 when $e = 6$ in., P_n can be determined in steps similar to those of Example 11.9:

1. Because $e = 6$ in. is less than $e_b = 10.5$ in., compression failure condition occurs.
2. Assume $c = 9.0$ in. (by trial) $> c_b = 7.75$ in. and $a = 7.65$ in.
3. Calculate $\bar{x} = 3.585$ in., $Z = 4.415$ in., and the area of concrete segment $= 94.93$ in.^{2}

4–5. Calculate forces: and $C_c = 322.7$ K, $C_{s1} = 110.7$ K, $C_{s2} = 53.1$ K, $T_1 = 21.6$ K, and $T_2 = 78.9$ K.

6. Calculate $P_{n1} = C_c + \Sigma C_s - \Sigma T = 386$ K.
7. Taking moments about the center of the column (plastic centroid): $M_n = 191$ K·ft, $P_{n2} = M_n/6 = 382$ K, which is close to P_{n1} (the difference is about 1%). Therefore, $P_n = 382$ K. Note that if the column is spirally reinforced, $\phi = 0.70$.

An approximate equation for estimating P_n in a circular section when compression controls was suggested by Whitney [15]:

$$P_n = \frac{A_g f'_c}{\left[\dfrac{9.6he}{(0.8h + 0.67D_s)^2} + 1.18\right]} + \frac{A_{st} f_y}{\left(\dfrac{3e}{D_s} + 1\right)} \tag{11.23}$$

where

A_g = gross area of the section
h = diameter of section
D_s = diameter measured through the centroid of the bar arrangement
A_{st} = total vertical steel area
e = eccentricity measured from the plastic centroid

Example 11.10

Calculate the nominal compressive strength P_n for the section of Example 11.9 using the Whitney equation if the eccentricity is $e = 6$ in.

Solution

1. $e = 6$ in. is less than $e_b = 10.5$ in., calculated earlier; thus, compression controls.
2. Using the Whitney equation,

$$A_g = \frac{\pi}{4} h^2 = \frac{\pi}{4} (16)^2 = 20.1 \text{ in.}^2$$

$$h = 16 \text{ in.} \quad D_s = 16 - 5 = 11.0 \text{ in.} \quad A_{st} = 8 \times 1 = 8 \text{ in.}^2$$

$$P_n = \frac{(201.1 \times 4)}{\left[\dfrac{(9.6 \times 16 \times 6)}{(0.8 \times 16 + 0.67 \times 11)^2} + 1.18\right]} + \frac{8 \times 60}{\left(\dfrac{3 \times 6}{11}\right) + 1}$$

$$= 415.5 \text{ K}$$

3. $M_n = P_n e = 415.5 \times \frac{6}{12} = 207.8$ K·ft. The value of P_n here is greater than $P_n = 382$ K calculated earlier by statics.

11.11.3 Strength of Circular Columns for Tension Failure

Tension failure occurs in circular columns when the load is applied at an eccentricity $e > e_b$, or $P_n < P_b$. In this case, the column section can be analyzed in steps similar to those of the balanced section and Example 11.8. This is achieved by assuming $c < c_b$ or $a < a_b$ and then following the steps explained in Section 11.11.1. Note that because the steel bars are uniformly distributed along the perimeter of the circular section, the tension steel A_s provided could be relatively low, and the load capacity becomes relatively small. Therefore, it is advisable to avoid the use of circular columns for tension failure cases.

11.12 ANALYSIS AND DESIGN OF COLUMNS USING CHARTS

The analysis of column sections explained earlier is based on the principles of statics. For preliminary analysis or design of columns, special charts or tables may be used either to determine ϕP_n and ϕM_n for a given section or determine the steel requirement for a given load P_u and moment M_u. These charts and tables are published by the American Concrete Institute (ACI) [7], the Concrete Reinforcing Steel Institute (CRSI), and the Portland Cement Association (PCA). Final design of columns must be based on statics by using manual calculations or computer programs. The use of the ACI charts is illustrated in the following examples. The charts are given in Figs. 11.16 and 11.17 [7]. These data are limited to the column sections shown on the top right corner of the charts.

Example 11.11

Determine the necessary reinforcement for a short tied column shown in Fig. 11.18*a* to support a factored load of 483 K and a factored moment of 322 K·ft. The column section has a width of 14 in. and a total depth, h, of 20 in. Use $f'_c = 4$ ksi, $f_y = 60$ ksi.

Solution

1. The eccentricity $e = M_u/P_u = 322 \times 12/483 = 8$ in. Let $d = 20 - 2.5 = 17.5$ in., $\gamma h = 20 - 5 = 15$ in., and $\gamma = 15/20 = 0.75$.
2. Since $e = 8$ in. $< d$, assume compression-controlled section with $\phi = 0.65$.

$$P_n = 483/0.65 = 743 \text{ K} \quad \text{and} \quad M_n = 322/0.65 = 495.4 \text{ K·ft.}$$

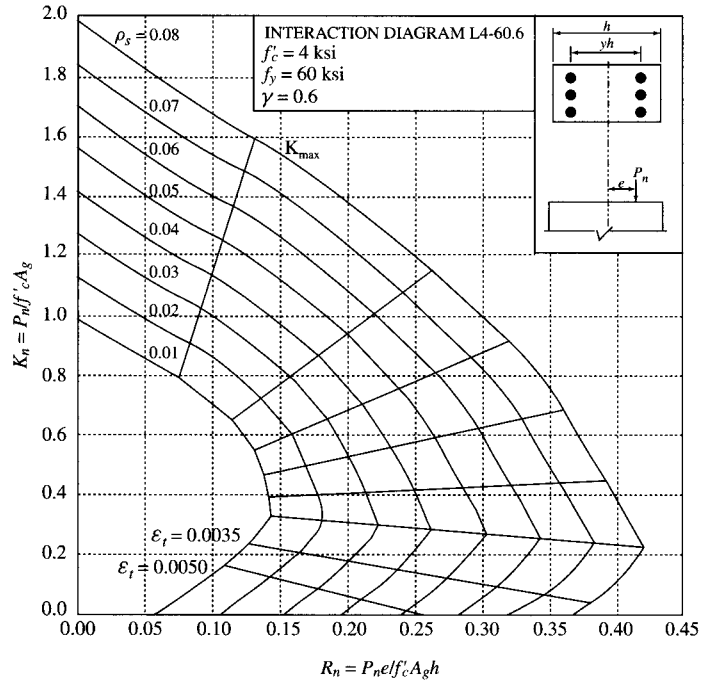
$$K_n = \frac{743}{(4 \times 14 \times 20)} = 0.663$$

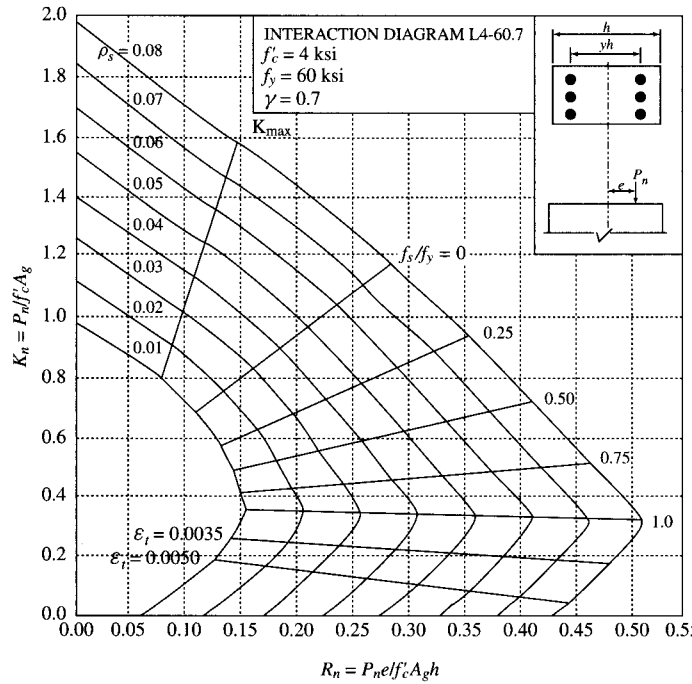
$$R_n = K_n \left(\frac{e}{h}\right) = 0.663 \left(\frac{8}{20}\right) = 0.265$$

3. From the charts of Fig. 11.16, for $\gamma = 0.7$, $\rho = 0.034$. Also, for $\gamma = 0.8$, $\rho = 0.039$. By interpolation, for $\gamma = 0.75$, $\rho = 0.0365$.

$$A_s = 0.0365\,(14 \times 20) = 10.22 \text{ in.}^2$$

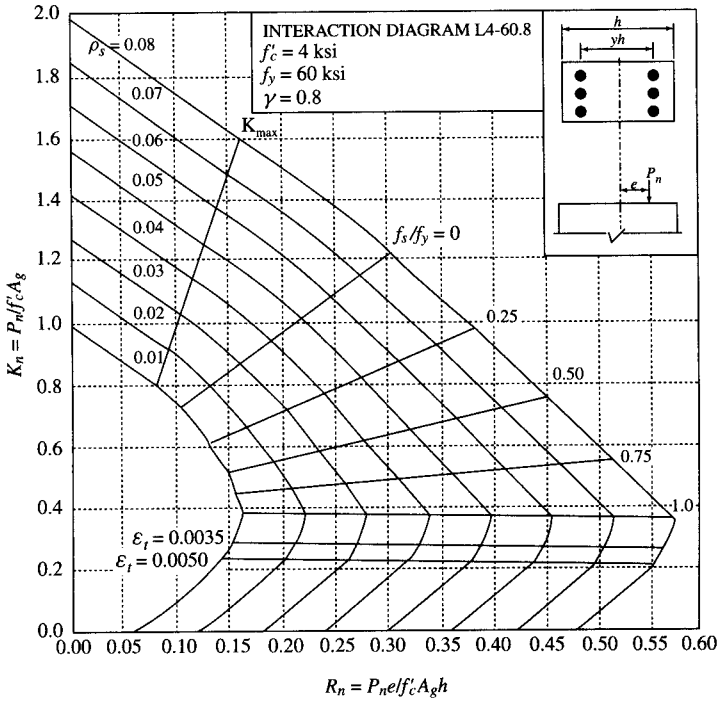
Use eight no. 10 bars ($A_s = 10.16$ in.$_2$), four on each short side. Use no. 3 ties spaced at 14 in. (Fig. 11.18*a*).



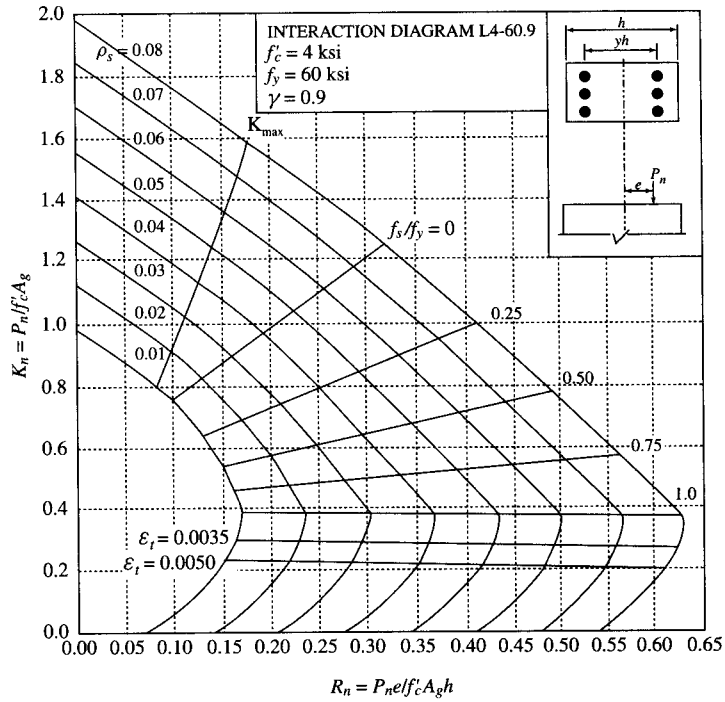


(*b*)

Figure 11.16 Load-moment strength interaction diagram for rectangular columns where $f'_c = 4$ ksi, $f_y = 60$ ksi, and (*a*) $\gamma = 0.60$, (*b*) $\gamma = 0.70$, (*c*) $\gamma = 0.80$, and (*d*) $\gamma = 0.90$. Courtesy of American Concrete Institute [7].

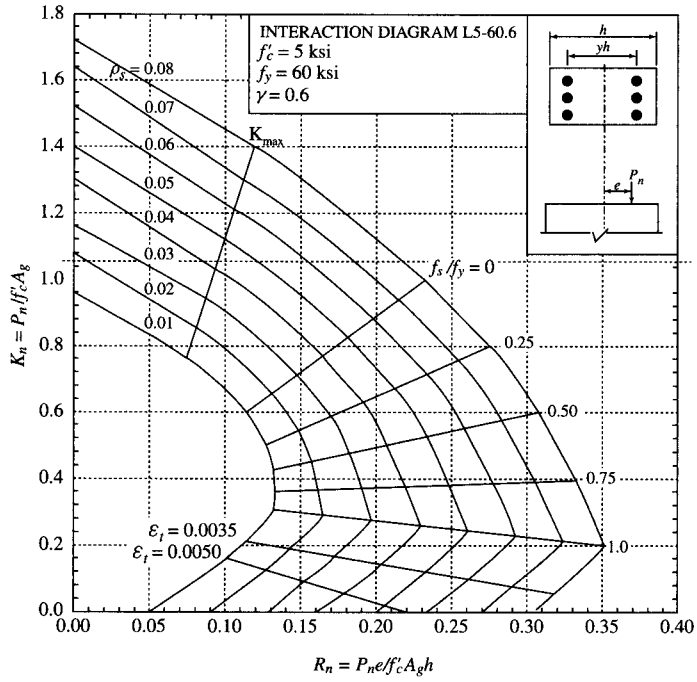


(*c*)

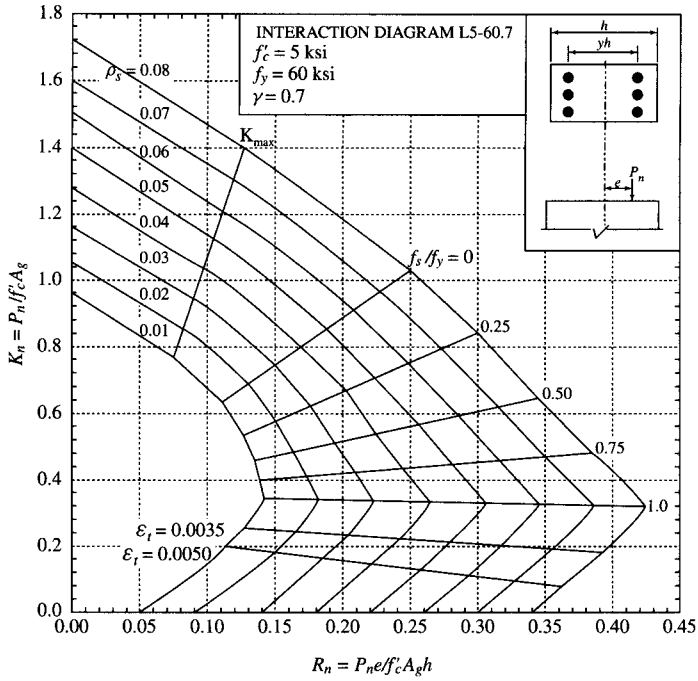


(*d*)

Figure 11.16 (*continued*)

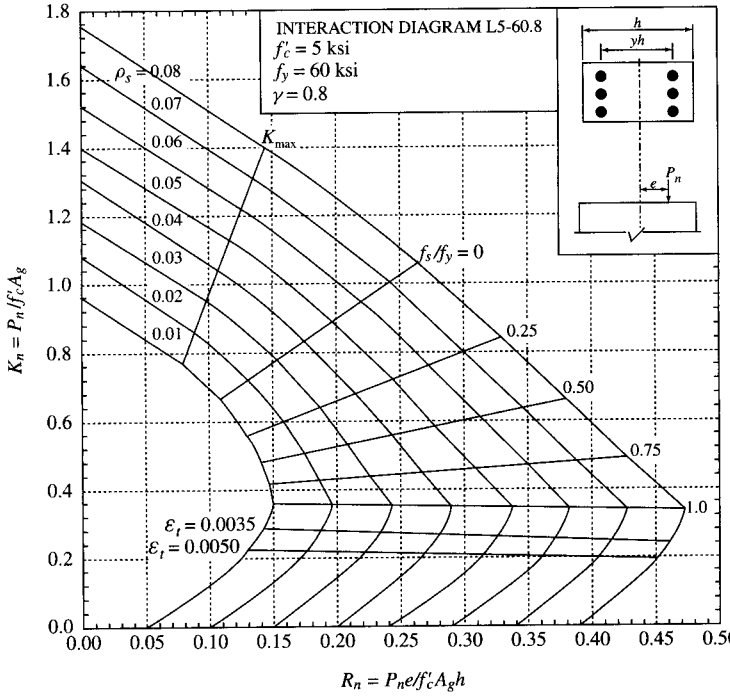


(*a*)

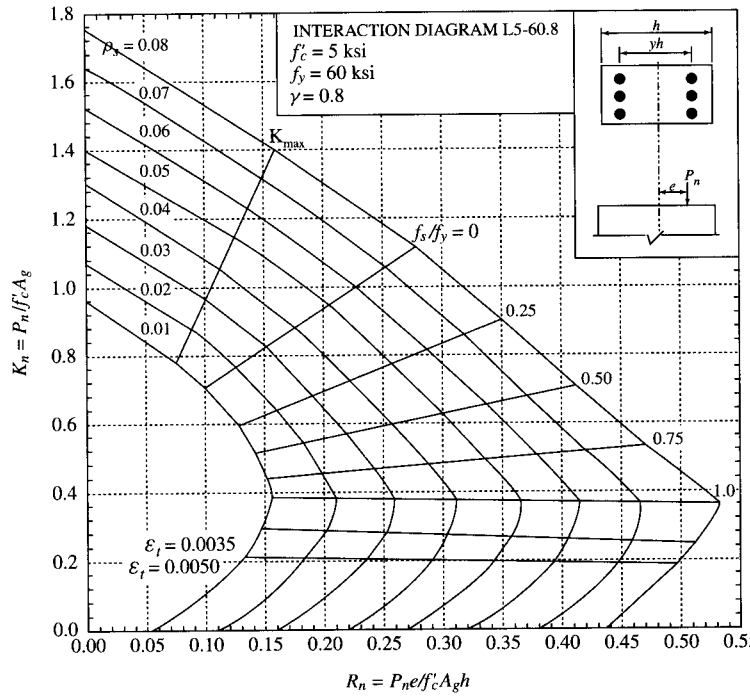


(*b*)

Figure 11.17 Load–moment strength interaction diagram for rectangular columns where $f'_c = 5$ ksi, $f_y = 60$ ksi, and (*a*) $\gamma = 0.60$, (*b*) $\gamma = 0.70$, (*c*) $\gamma = 0.80$, and (*d*) $\gamma = 0.90$. Courtesy of American Concrete Institute [7].



(*c*)



(*d*)

Figure 11.17 (*continued*)

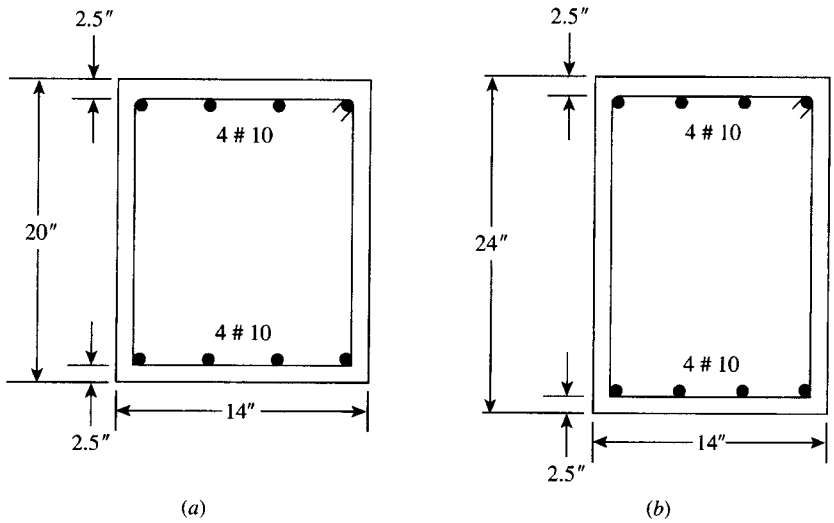


Figure 11.18 Column sections of (*a*) Example 11.11 and (*b*) Example 11.12.

Example 11.12

Use the charts to determine the column strength, ϕP_n, of the short column shown in Fig. 11.18*b* acting at an eccentricity $e = 12$ in. Use $f'_c = 5$ ksi and $f_y = 60$ ksi.

Solution

1. Properties of the section: $H = 24$ in., $\gamma h = 24 - 5 = 19$ in. (distance between tension and compression steel). $\gamma = 19/24 = 0.79$, and $\rho = 8(1.27)/(14 \times 24) = 0.03$.
2. Since $e < d$, assume compression-controlled section. Let $\varepsilon_t = 0.002$, $f_s/f_y = 1.0$ and $\phi = 0.65$. From the charts of Fig. 11.17, get $K_n = 0.36 = P_n/(5 \times 14 \times 24)$. Then $P_n = 605$ K.
3. Check assumption for compression-controlled section: For $K_n = 0.36$, $R_n = K_n\,(e/h) = 0.36\,(12/24) = 0.18$. From charts, get $\rho = 0.018 < 0.03$. Therefore, $P_n > 605$ K (to use $\rho = 0.003$).
4. Second trial: Let $\varepsilon_t = 0.0015$, $f_s = 0.0015\,(29{,}000) = 43.5$ ksi.

$$f_s/f_y = 43.5/60 = 0.725 \quad \rho = 0.03 \quad K_n = 0.44$$

$$0.44 = P_n/(5 \times 14 \times 24) \quad P_n = 740 \text{ K}$$

5. Check assumption: For $K_n = 0.44$, $R_n = 0.44\,(12/24) = 0.22$. From charts, $\rho = 0.03$ as given. Therefore, $P_n = 740$ K.

$$\phi P_n = 0.65(740) = 480 \text{ K} \quad \text{and} \quad \phi M_n = 0.65(740) = 480 \text{ K·ft}$$

By analysis, $\phi P_n = 485$ K(which is close to 480 K·ft).

11.13 DESIGN OF COLUMNS UNDER ECCENTRIC LOADING

In the previous sections, the analysis, behavior, and the load–interaction diagram of columns subjected to an axial load and bending moment were discussed. The design of columns is more complicated, because the external load and moment, P_u and M_u, are given and it is

required to determine many unknowns, such as b, h, A_s, and A'_s, within the ACI Code limitations. It is a common practice to assume a column section first and then determine the amount of reinforcement needed. If the designer needs to change the steel reinforcement calculated, then the cross-section may be adjusted accordingly. The following examples illustrate the design of columns.

11.13.1 Design of Columns for Compression Failure

For compression failure, it is preferable to use $A_s = A'_s$ for rectangular sections. The eccentricity, e, is equal to M_u/P_u. Based on the magnitude of e, two cases may develop.

1. When e is relatively very small (say, $e \leq 4$ in.), a minimum eccentricity case may develop that can be treated by using Eq. 10.8, as explained in the examples of Chapter 10. Alternatively, the designer may proceed as in Case 2. This loading case occurs in the design of the lower-floor columns in a multistory building, where the moment, M_u, develops from one floor system and the load, P_u, develops from all floor loads above the column section.
2. The compression failure zone represents the range from the axial to the balanced load, as shown in Figs. 11.3 and 11.11. In this case, a cross-section (bh) may be assumed and then the steel reinforcement is calculated for the given P_u and M_u. The steps can be summarized as follows:
   a. Assume a square or rectangular section (bh); then determine d, d', and $e = M_u/P_u$.
   b. Assuming $A_s = A'_s$, calculate A'_s from Eq. 11.17 using the dimensions of the assumed section, and $\phi = 0.65$ for tied columns. Let $A_s = A'_s$ and then choose adequate bars. Determine the actual areas used for A_s and A'_s. Alternatively, use the ACI charts.
   c. Check that $\rho_g = (A_s + A'_s)/bh$ is less than or equal to 8% and greater or equal to 1%. If ρ_g is small, reduce the assumed section, but increase the section if less steel is required.
   d. Check the adequacy of the final section by calculating ϕP_n from statics; as explained in the previous examples, ϕP_n should be greater than or equal to P_u.
   e. Determine the necessary ties.

A simple approximate formula for determining the initial size of the column bh or the total steel ratio ρ_g is

$$P_n = K_c bh^2 \quad \text{or} \quad P_u = \phi P_n = \phi K_c bh^2 \tag{11.24}$$

where K_c has the values shown in Table 11.2 and plotted in Fig. 11.19 for $f_y = 60$ ksi and $A_s = A'_s$. Units for K_c are in lb/in.^{3}

The values of K_c shown in Table 11.2 are approximate and easy to use, because K_c increases by 0.02 for each increase of 1 ksi in f'_c. For the same section, as the eccentricity,

Table 11.2 Values of K_c ($f_y = 60$ ksi)

	K_c		
ρ_g (%)	$f'_c = 4$ ksi	$f'_c = 5$ ksi	$f'_c = 6$ ksi
1%	0.090	0.110	0.130
4%	0.137	0.157	0.177
8%	0.200	0.220	0.240

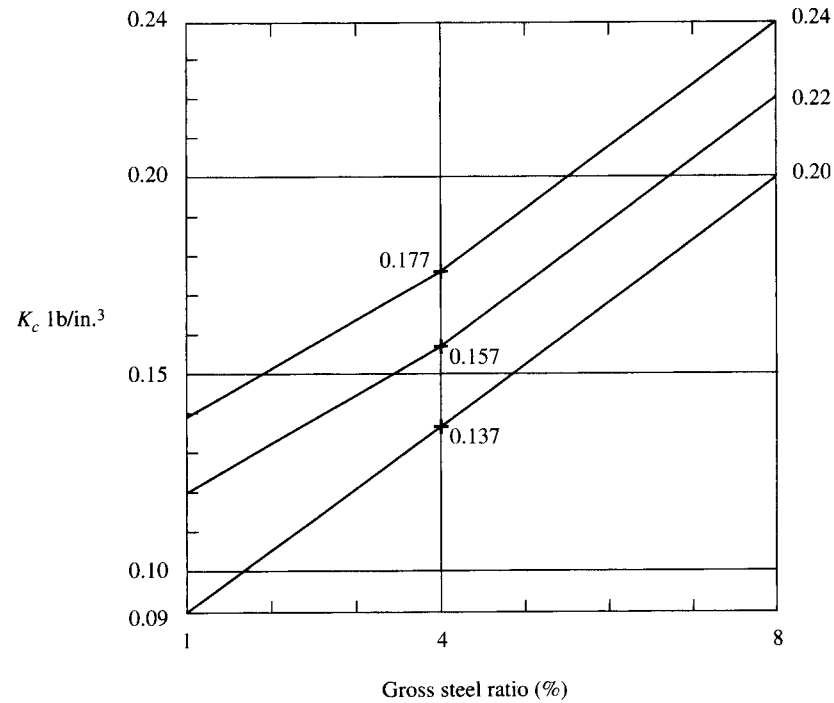


Figure 11.19 Values of K_c versus ρ_g (%).

$e = M_u/P_u$, increases, P_n decreases, and, consequently, K_c decreases. Thus, K_c values represent a load P_n on the interaction diagram between 0.8 P_{n_0} and P_b as shown in Fig. 11.3 or 11.11.

Linear interpolation can be used. For example, $K_c = 0.1685$ for $\rho_g = 6\%$ and $f'_c = 4$ ksi. The steps in designing a column section can be summarized as follows:

1. Assume an initial size of the column section bh.
2. Calculate $K_c = P_u/(\phi bh^2)$.
3. Determine ρ_g from Table 11.2 for the given f'_c.
4. Determine $A_s = A'_s = \rho_g bh/2$ and choose bars and ties.
5. Determine ϕP_n of the final section by statics (accurate solution). The value of ϕP_n should be greater than or equal to P_u. If not, adjust bh or ρ_g.

Alternatively, if a specific steel ratio is desired, say $\rho_g = 6\%$, then proceed as follows:

1. Assume ρ_g as required and then calculate $e = M_u/P_u$.
2. Based on the given f'_c and ρ_g, determine K_c from Table 11.2.
3. Calculate $bh^2 = P_u/\phi K_c$; then choose b and h. Repeat steps 4 and 5. It should be checked that ρ_g is less than or equal to 8% and greater than or equal to 1%. Also, check that c calculated by statics is greater than $c_b = 87d_t/(87 + f_y)$ for compression failure to control.

Example 11.13

Determine the tension and compression reinforcement for a 16 × 24-in. rectangular tied column to support $P_u = 780$ K and $M_u = 390$ K·ft. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. Calculate $e = M_u/P_u = 420(12)/840 = 6.0$ in. We have $h = 24$ in; let $d = 21.5$ in. and $d' = 2.5$ in. Because e is less than $\frac{2}{3}d = 14.38$ in., assume compression failure.
2. Assume $A_s = A'_s$ and use Eq. 11.17 to determine the initial value of $A'_s P_n = P_u/\phi = 780/0.65 = 1200$ K.

$$P_n = \frac{bhf'_c}{\left(\frac{3he}{d^2}\right) + 1.18} + \left[\frac{A'_s f_y}{\left(\frac{e}{d-d'}\right) + 0.5}\right] \tag{11.17}$$

   For $P_n = 1200$ K, $e = 6$ in., $d = 21.5$ in., $d' = 2.5$ in., and $h = 24$ in., calculate $A'_s = 6.44$ in.$^2 = A_s$. Choose five no. 10 bars ($A_s = 6.35$ in.^{2}) for A_s and A'_s (Fig. 11.20).
3. $\rho_g = 2(6.35)/(16 \times 24) = 0.033$, which is less than 0.08 and >0.01.
4. Check the section by statics following the steps of Example 11.4 to get

$$a = 16.64 \text{ in.} \quad c = 19.58 \text{ in.} \quad C_c = 905.2 \text{ K}$$

$$C_s = 6.35(60 - 0.85 \times 4) = 359.4 \text{ K}$$

$$f_s = 87\left(\frac{d-c}{c}\right) = 8.55 \text{ ksi}$$

$$T = A_s f_s = 6.35(8.55) = 54.3 \text{ K}$$

$$P_n = C_c + C_s - T = 1210.3 \text{ K} > 1200 \text{ K}$$

   Note that if $\phi P_n < P_u$, increase A_s and A'_s, for example, to six no. 10 bars, and check the section again.
5. Check P_n based on moments about A_s (Eq. 11.12) to get $P_n = 1210$ K.
6. For a balanced section,

$$c_b = \left(\frac{87}{87 + f_y}\right) d_t = \left(\frac{87}{147}\right) 21.5 = 12.7 \text{ in.}$$

   Because $c = 19.58$ in. $> c_b = 12.7$ in., this is a compression failure case, as assumed.
7. Use no. 3 ties spaced at 16 in. (Refer to Chapter 10.)

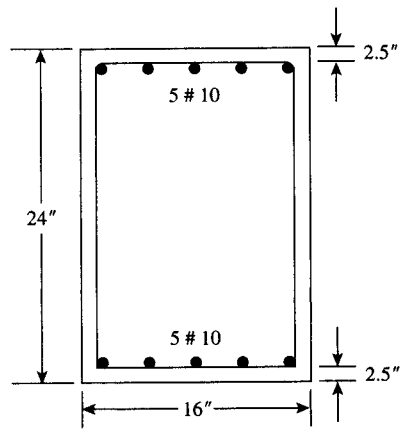


Figure 11.20 Example 11.13.

Example 11.14

Repeat Example 11.13 using Eq. 11.24.

Solution

1. The column section is given: 16 × 24 in.
2. Determine K_c from Eq. 11.24:

$$K_c = \frac{P_u}{\phi b h^2} = \frac{780}{0.65 \times 16 \times 24^2} = 0.13 \text{ lb/in.}^3$$

3. From Table 11.2 or Fig. 11.19, for $K_c = 0.13$, $f'_c = 4$ ksi, by interpolation, get $\rho_g = 3.5\%$.
4. Calculate $A_s = A'_s = \rho bh/2 = 0.035(16)(24)/2 = 6.77$ in.^{2} Choose five no. 10 bars ($A_s = 6.35$ in.^{2}) for the first trial.
5. Determine ϕP_n using steps 4–7 in Example 11.13. $\phi P_n = 1210.3$ K $> P_n = 1200$ K, so the section is adequate.
6. If the section is not adequate, or $\phi P_n < P_u$, increase A_s and A'_s and check again to get closer values.

Example 11.15

Design a rectangular column section to support $P_u = 696$ K and $M_u = 465$ K·ft with a total steel ratio ρ_g of about 4%. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, and $b = 18$ in.

Solution

1. Calculate $e = M_u/P_u = 465(12)/696 = 8$ in. Assume compression failure ($\phi = 0.65$) (to be checked later) and $A_s = A'_s$.
2. For $\rho_s = 4\%$ and $f'_c = 4$ ksi, $K_c = 0.137$ (Table 11.2).
3. Calculate bh^2 from Eq. 11.24: $P_u = \phi K_c bh^2$, or $696 = 0.65(0.137)(18)h^2$. Thus, $h = 20.84$ in. Let $h = 22$ in.
4. Calculate $A_s = A'_s = 0.04(18 \times 22)/2 = 7.92$ in.^{2} Choose five no. 11 bars ($A_s = 7.8$ in.^{2}) in one row for A_s and A'_s (Fig. 11.21). Choose no. 4 ties spaced at 18 in.

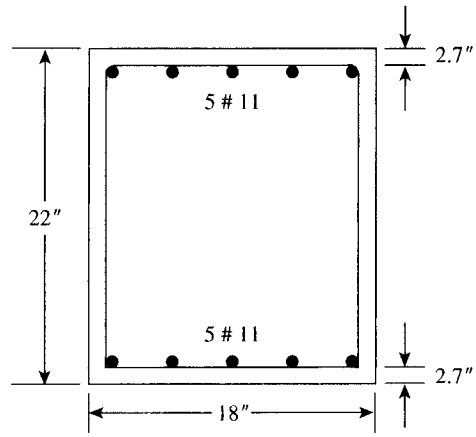


Figure 11.21 Example 11.15.

5. Check the final section by analysis, similar to Example 11.4, to get $a = 13.15$ in., $c = 15.47$ in., $C_c = 0.85\ f'_c ab = 804.8$ K, $f'_s = 60$ ksi, $C_s = A'_s(f_y - 0.85f'_c) = 441.5$ K, $f_s = 87[(d - c)/c] = 21.24$ ksi, and $T = A_s f_s = 168$ K. Also, $P_n = C_c + C_s - T = 1078.3$ K and $\phi P_n = 0.65P_n = 701\,\text{K} > 696\,\text{K}$. The section is adequate.
6. For a balanced section,

$$c_b = \left(\frac{87}{87 + f_y}\right) \qquad d_t = \left(\frac{87}{147}\right) 19.3 = 11.42 \text{ in.} < c = 15.47 \qquad (d = 19.3 \text{ in.})$$

Therefore, this is a compression failure case, as assumed.

11.13.2 Design of Columns for Tension Failure

Tension failure occurs when $P_n < P_b$ or the eccentricity $e > e_b$, as explained in Section 11.7. In the design of columns, P_u and M_u are given, and it is required to determine the column size and its reinforcement. It may be assumed (as a guide) that tension controls when the ratio of M_u (K·ft) to P_u (kips) is greater than 1.75 for sections of $h < 24$ in. and 2.0 for $h \geq 24$ in. In this case, a section may be assumed, and then A_s and A'_s are determined. The ACI charts may be used to determine ρ_g for a given section with $A_s = A'_s$. Note that ϕ varies between 0.65 (0.75) and 0.9, as explained in Section 11.4.

When tension controls, the tension steel yields, whereas the compression steel may or may not yield. Assuming initially $f'_s = f_y$ and $A_s = A'_s$, Eq. 11.16 (Section 11.6) may be used to determine the initial values of A_s and A'_s:

$$A_s = A'_s = \frac{P_n\left(e - \frac{h}{2} + \frac{a}{2}\right)}{f_y(d - d')} \tag{11.16}$$

Because a is not known yet, assume $a = 0.4d$ and $P_u = \phi P_n$; then

$$A_s = A'_s = \frac{P_u(e - 0.5h + 0.2d)}{\phi f_y(d - d')} \tag{11.25}$$

The final column section should be checked by statics to prove that $\phi P_n \geq P_u$. Example 11.16 explains this approach.

When the load P_u is very small relative to M_u, the section dimensions may be determined due to M_u only, assuming $P_u = 0$. The final section should be checked by statics. This case occurs in single- or two-story building frames used mainly for exhibition halls or similar structures. In this case, A'_s may be assumed to be less than A_s. A detailed design of a one-story, two-hinged frame exhibition hall is given in Chapter 16.

Example 11.16

Determine the necessary reinforcement for a 16 × 22-in. rectangular tied column to support a factored load $P_u = 257$ K and a factored moment $M_u = 643$ K·ft. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. Calculate $e = M_u/P_u = 643(12)/257 = 30$ in; let $d = 22 - 2.5 = 19.5$ in. Because $M_u/P_u = 500/200 = 2.5 > 1.75$, or because $e > d$, assume tension failure case, $\phi = 0.9$ (to be checked later).

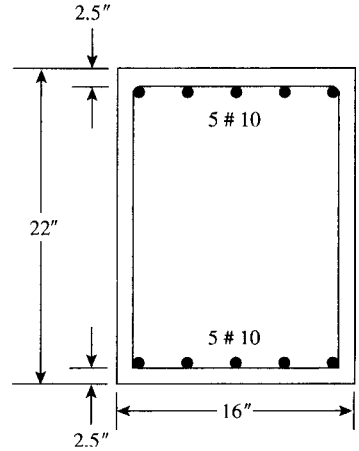


Figure 11.22 Example 11.16.

2. Assume $A_s = A'_s$ and $f'_s = f_y$ and use Eq. 11.25 to determine A_s and A'_s. Let $P_u = 257.0$ K, $e = 30$ in., $h = 22$ in., $d = 19.5$ in., and $d' = 2.5$ in.

$$A_s = A'_s = \frac{257(30 - 0.5 \times 22 + 0.2 \times 19.5)}{0.9(60)(17.0)} = 6.41 \text{ in.}^2$$

Choose five no. 10 bars (6.35 in.^{2}) in one row for each of A_s and A'_s (Fig. 11.22).

3. Check $\rho_g = 2(6.35)(16 \times 22) = 0.036$, which is less than 0.08 and greater than 0.01.
4. Check the chosen section by statics similar to Example 11.3.
   a. Determine the value of a using the general equation $Aa^2 + Ba + C = 0$ with $e' = e + d - h/2 = 38.5$ in., $A = 0.425\, f'_c b = 27.2$, $B = 2A(e' - d) = 1033.6$, $C = A'_s(f_y - 0.85f'_c)(e' - d + d') - A_s f_y e' = -6941.2$. Solve to get $a = 5.82$ in. and $c = a/0.85 = 6.85$.
   b. Check f'_s:

$$f'_s = 87\left(\frac{c - d'}{c}\right) = 87\left(\frac{6.85 - 2.5}{6.85}\right) = 55.26 \text{ ksi}$$

   Let $f'_s = 57$ ksi.
   c. Recalculate a:

$$C = A'_s(f'_s - 0.85f'_c)(e' - d + d') - A_s f_y e' = -7351$$

   Solve now for a to get $a = 6.13$ and $c = 7.21$ in.
   d. Check f'_s:

$$f'_s = 87\left(\frac{c - 2.5}{c}\right) = 56.83 \text{ ksi}$$

   Calculate

$$C_c = 0.85(4)(6.13)(16) = 333.5 \text{ K}, C_s = A'_s(f'_s - 0.85f'_c) = 6.35(57 - 0.85 \times 4)$$
$$= 340.4 \text{ K}, T = A_s f_y = 6.35(60) = 381 \text{ K}.$$

   e. $P_n = C_c + C_s - T = 292.9$ K.
5. Determine ϕ: $\varepsilon_t = [(d_t - c)/c]\ 0.003 = 0.00511$. Because $\varepsilon_t = 0.00511 > 0.005$, $\phi = 0.9$.
6. $\phi P_n = 0.9(292.9) = 263.6\text{ K} > 257\text{ K}$, the section is adequate.

11.14 BIAXIAL BENDING

The analysis and design of columns under eccentric loading was discussed earlier in this chapter, considering a uniaxial case. This means that the load P_n was acting along the y-axis (Fig. 11.23), causing a combination of axial load P_n and a moment about the x-axis equal to $M_{nx} = P_n e_y$ or acting along the x-axis (Fig. 11.24) with an eccentricity e_x, causing a combination of an axial load P_n and a moment $M_{nx} = P_n e_x$.

If the load P_n is acting anywhere such that its distance from the x-axis is e_y and its distance from the y-axis is e_x, then the column section will be subjected to a combination of forces: an axial load P_n a moment about the x-axis $= M_{nx} = P_n e_y$ and a moment about the y-axis $= M_{nx} = P_n e_x$ (Fig. 11.25). The column section in this case is said to be subjected to *biaxial bending*. The analysis and design of columns under this combination of forces is not simple when the principles of statics are used. The neutral axis is at an angle with respect to both axes, and lengthy calculations are needed to determine the location of the neutral axis, strains, concrete compression area, and internal forces and their point of application. Therefore, it was necessary

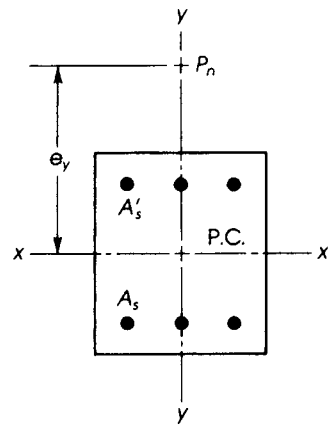


Figure 11.23 Uniaxial bending with load P_n along the y-axis with eccentricity e_y.

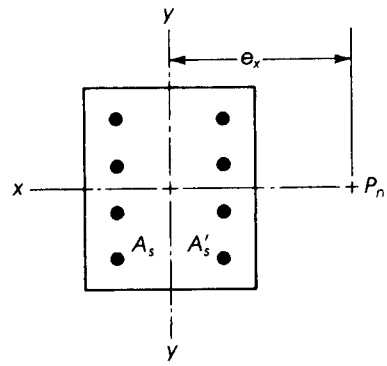


Figure 11.24 Uniaxial bending with load P_n along the x-axis, with eccentricity e_x.

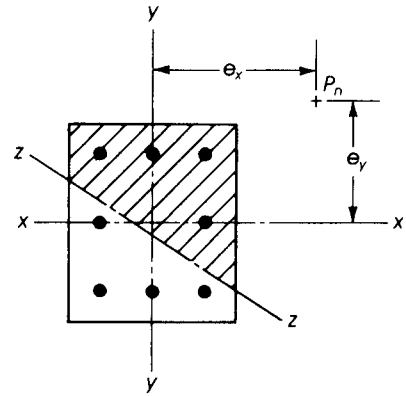


Figure 11.25 Biaxial bending.

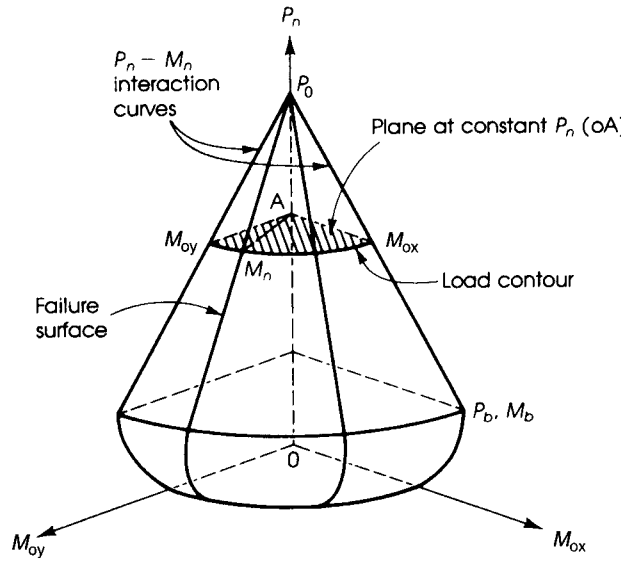


Figure 11.26 Biaxial interaction surface.

to develop practical solutions to estimate the strength of columns under axial load and biaxial bending. The formulas developed relate the response of the column in biaxial bending to its uniaxial strength about each major axis.

The biaxial bending strength of an axially loaded column can be represented by a three-dimensional interaction curve, as shown in Fig. 11.26. The surface is formed by a series of uniaxial interaction curves drawn radially from the P_n-axis. The curve M_{ox} represents the interaction curve in uniaxial bending about the x-axis, and the curve M_{oy} represents the curve in uniaxial bending about the y-axis. The plane at constant axial load P_n shown in Fig. 11.26 represents the contour of the bending moment M_n about any axis.

Different shapes of columns may be used to resist axial loads and biaxial bending. Circular, square, or rectangular column cross-sections may be used with equal or unequal bending capacities in the x- and y-directions.

11.15 CIRCULAR COLUMNS WITH UNIFORM REINFORCEMENT UNDER BIAXIAL BENDING

Circular columns with reinforcement distributed uniformly about the perimeter of the section have almost the same moment capacity in all directions. If a circular column is subjected to biaxial bending about the x- and y-axes, the equivalent uniaxial M_u moment can be calculated using the following equations:

$$M_u = \sqrt{(M_{ux})^2 + (M_{uy})^2} = P_u \cdot e \tag{11.26}$$

and

$$e = \sqrt{(e_x)^2 + (e_y)^2} = \frac{M_u}{P_u} \tag{11.27}$$

where

$M_{ux} = P_u e_y$ = factored moment about the x-axis
$M_{uy} = P_u e_x$ = factored moment about the y-axis
$M_u = P_u e$ = equivalent uniaxial factored moment of the section due to M_{ux} and M_{uy}

In circular columns, a minimum of six bars should be used, and these should be uniformly distributed in the section.

Example 11.17: Circular Column

Determine the load capacity P_n of a 20-in.-diameter column reinforced with 10 no. 10 bars when $e_x = 4$ in. and $e_y = 6$ in. Use $f'_c = 4$ ksi and f_y and 60 ksi.

Solution

1. Calculate the eccentricity that is equivalent to uniaxial loading by using Eq. 11.41.

$$e(\text{for uniaxial loading}) = \sqrt{e_x^2 + e_y^2} = \sqrt{(4)^2 + (6)^2} = 7.211 \text{ in.}$$

2. Determine the load capacity of the column based on $e = 7.211$ in. Proceed as in Example 11.9:

$$d = 17.12 \text{ in.} \qquad a = 9.81 \text{ in.} \qquad c = 11.54 \text{ in. (by trial)}$$

$$C_c = 521.2 \text{ K} \qquad \sum C_s = 269.8 \text{ K} \qquad \sum T = 132.1 \text{ K}$$

$$P_n = C_c + \sum C_s - \sum T = 650 \text{ K}$$

3. For a balanced condition,

$$c_b = \left(\frac{87}{87 + f_y}\right) d_t = \left(\frac{87}{147}\right) 17.12 = 10.13 \text{ in.}$$

$c = 11.54$ in. $> c_b$, which is a compression failure case.

11.16 SQUARE AND RECTANGULAR COLUMNS UNDER BIAXIAL BENDING

11.16.1 Bresler Reciprocal Method

Square or rectangular columns with unequal bending moments about their major axes will require a different amount of reinforcement in each direction. An approximate method of analysis of such sections was developed by Boris Bresler and is called the Bresler reciprocal method [9,12]. According to this method, the load capacity of the column under biaxial bending can be determined by using the following expression:

$$\frac{1}{P_u} = \frac{1}{P_{ux}} + \frac{1}{P_{uy}} - \frac{1}{P_{u0}} \tag{11.28}$$

or

$$\frac{1}{P_n} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_{n0}} \tag{11.29}$$

where

P_u = factored load under biaxial bending
P_{ux} = factored uniaxial load when the load acts at an eccentricity e_y and $e_x = 0$
P_{uy} = factored uniaxial load when the load acts at an eccentricity e_x and $e_y = 0$
P_{u0} = factored axial load when $e_x = e_y = 0$

$$P_n = \frac{P_u}{\phi} \quad P_{nx} = \frac{P_{ux}}{\phi} \quad P_{ny} = \frac{P_{uy}}{\phi} \quad P_{n0} = \frac{P_{u0}}{\phi}$$

The uniaxial load strengths P_{nx}, P_{ny}, and P_{n0} can be calculated according to the equations and method given earlier in this chapter. After that, they are substituted into Eq. 11.29 to calculate P_n.

The Bresler equation is valid for all cases when P_n is equal to or greater than $0.10P_{n0}$. When P_n is less than $0.10P_{n0}$, the axial force may be neglected and the section can be designed as a member subjected to pure biaxial bending according to the following equations:

$$\frac{M_{ux}}{M_x} + \frac{M_{uy}}{M_y} \leq 1.0 \tag{11.30}$$

or

$$\frac{M_{nx}}{M_{ox}} + \frac{M_{ny}}{M_{oy}} \leq 1.0 \tag{11.31}$$

where

$M_{ux} = P_u e_y$ = design moment about the x-axis
$M_{uy} = P_u e_x$ = design moment about the y-axis
M_x and M_y = uniaxial moment strengths about the x-and y-axes

$$M_{nx} = \frac{M_{ux}}{\phi} \qquad M_{ny} = \frac{M_{uy}}{\phi} \qquad M_{ox} = \frac{M_x}{\phi} \qquad M_{oy} = \frac{M_y}{\phi}$$

The Bresler equation is not recommended when the section is subjected to axial tension loads.

11.16.2 Bresler Load Contour Method

In this method, the failure surface shown in Fig. 11.26 is cut at a constant value of P_n, giving the related values of M_{nx} and M_{ny}. The general nondimension expression for the load contour method is

$$\left(\frac{M_{nx}}{M_{ox}}\right)^{\alpha 1} + \left(\frac{M_{ny}}{M_{oy}}\right)^{\alpha 2} = 1.0 \tag{11.32}$$

Bresler indicated that the exponent α can have the same value in both terms of this expression ($\alpha_1 = \alpha_2$). Furthermore, he indicated that the value of α varies between 1.15 and 1.55 and can be assumed to be 1.5 for rectangular sections. For square sections, α varies between 1.5 and 2.0, and an average value of $\alpha = 1.75$ may be used for practical designs. When the reinforcement is uniformly distributed around the four faces in square columns, α may be assumed to be 1.5.

$$\left(\frac{M_{nx}}{M_{ox}}\right)^{1.5} + \left(\frac{M_{ny}}{M_{oy}}\right)^{1.5} = 1.0 \tag{11.33}$$

The British Code assumed $\alpha = 1.0$, 1.33, 1.67, and 2.0 when the ratio $P_u/1.1P_{u_0}$ is equal to 0.2, 0.4, 0.6, and ≥ 0.8, respectively.

11.17 PARME LOAD CONTOUR METHOD

The load contour approach, proposed by the Portland Cement Association (PCA), is an extension of the method developed by Bresler. In this approach, which is also called the *Parme method* [11], a point B on the load contour (of a horizontal plane at a constant P_n shown in Fig. 11.26) is defined such that the biaxial moment capacities M_{nx} and M_{ny} are in the same ratio as the uniaxial moment capacities M_{ox} and M_{oy}; that is,

$$\frac{M_{nx}}{M_{ny}} = \frac{M_{ox}}{M_{oy}} \quad \text{or} \quad \frac{M_{nx}}{M_{ox}} = \frac{M_{ny}}{M_{oy}} = \beta$$

The ratio β is shown in Fig. 11.27 and represents that constant portion of the uniaxial moment capacities that may be permitted to act simultaneously on the column section.

For practical design, the load contour shown in Fig. 11.27 may be approximated by two straight lines, AB and BC. The slope of line AB is $(1 - \beta)/\beta$, and the slope of line BC is $\beta/(1 - \beta)$. Therefore, when

$$\frac{M_{ny}}{M_{oy}} > \frac{M_{nx}}{M_{ox}}$$

then

$$\frac{M_{ny}}{M_{oy}} + \frac{M_{nx}}{M_{ox}}\left(\frac{1-\beta}{\beta}\right) = 1 \tag{11.34}$$

and when

$$\frac{M_{ny}}{M_{oy}} < \frac{M_{nx}}{M_{ox}}$$

then

$$\frac{M_{nx}}{M_{ox}} + \frac{M_{ny}}{M_{oy}}\left(\frac{1-\beta}{\beta}\right) = 1 \tag{11.35}$$

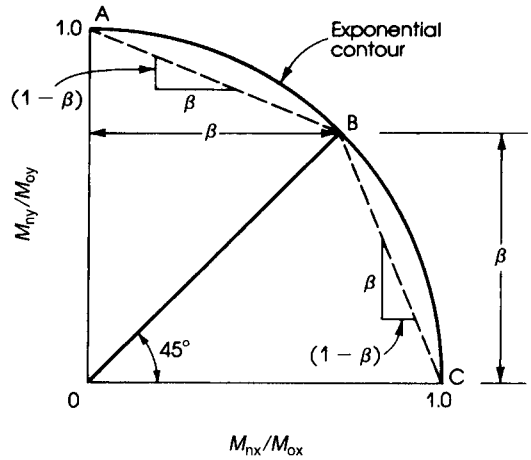


Figure 11.27 Nondimensional load contour at constant P_n (straight-line approximation).

The actual value of β depends on the ratio P_n/P_o as well as the material and properties of the cross-section. For lightly loaded columns, β will vary from 0.55 to 0.7. An average value of $\beta = 0.65$ can be used for design purposes.

When uniformly distributed reinforcement is adopted along all faces of rectangular columns, the ratio M_{oy}/M_{ox} is approximately b/h, where b and h are the width and total depth of the rectangular section, respectively. Substituting this ratio in Eqs. 11.34 and 11.35,

$$M_{ny} + M_{nx}\left(\frac{b}{h}\right)\left(\frac{1-\beta}{\beta}\right) \approx M_{oy} \tag{11.36}$$

and

$$M_{nx} + M_{ny}\left(\frac{h}{b}\right)\left(\frac{1-\beta}{\beta}\right) \approx M_{ox} \tag{11.37}$$

For $\beta = 0.65$ and $h/b = 1.5$,

$$M_{oy} \approx M_{ny} + 0.36M_{nx} \tag{11.38}$$

and

$$M_{ox} \approx M_{nx} + 0.80M_{ny} \tag{11.39}$$

From this presentation, it can be seen that direct explicit equations for the design of columns under axial load and biaxial bending are not available. Therefore, the designer should have enough experience to make an initial estimate of the section using the values of P_n, M_{nx} and M_{ny} and the uniaxial equations and then check the adequacy of the column section using the equations for biaxial bending or by computer.

Example 11.18

The section of a short tied column is 16 × 24 in. and is reinforced with eight no. 10 bars distributed as shown in Fig. 11.28. Determine the design load on the section ϕP_n if it acts at $e_x = 8$ in. and $e_y = 12$ in. Use $f'_c = 5$ ksi, $f_y = 60$ ksi, and the Bresler reciprocal equation.

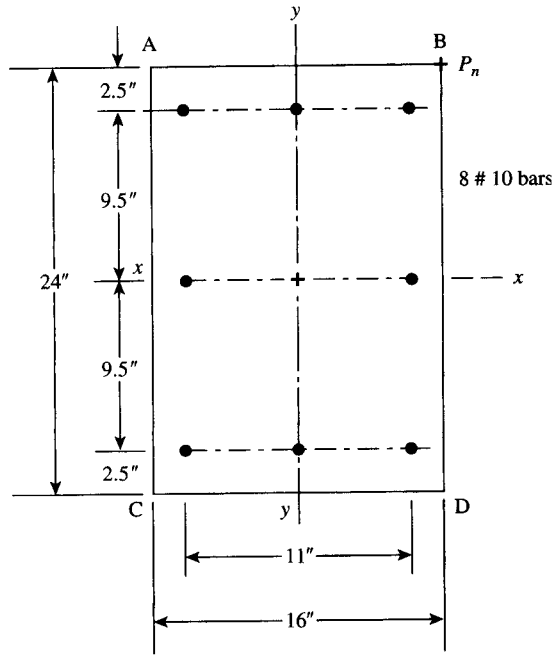


Figure 11.28 Example 11.18: biaxial load, Bresler method: $P_n = 421.5$ K.

Solution

1. Determine the uniaxial load capacity P_{nx} about the x-axis when $e_y = 12$ in. In this case, $b = 16$ in., $h = 24$ in., $d = 21.5$ in., $d' = 2.5$ in., and $A_s = A'_s = 3.81$ in.^{2} The solution will be performed using statics following the steps of Examples 11.2 and 11.4 for balanced and compression-control conditions.

   a. For the balanced condition,

$$c_b = \left(\frac{87}{87 + f_y}\right) \qquad d = \left(\frac{87}{147}\right) 21.5 = 12.72 \text{ in.}$$

$$a_b = 0.80(12.72) = 10.18 \text{ in.} \qquad (\beta_1 = 0.8 \text{ when } f'_c = 5 \text{ ksi})$$

$$C_c = 0.85 f'_c ab = 692.3 \text{ K} \qquad f'_s = 87\left(\frac{c - d'}{c}\right) = 69.9 \text{ ksi}$$

   Then $f'_s = 60$ ksi.

$$C_s = A'_s(f_y - 0.85 f'_c) = 212.4 \text{ K} \qquad T = A_s f_y = 228.6 \text{ K}$$

$$P_{ox} = C_c + C_s - T = 676.1 \text{ K}$$

$$\phi P_{bx} = 0.65 P_{bx} = 439.5 \text{ K} \qquad (\phi = 0.65 \text{ for } \varepsilon_t = 0.002)$$

   b. For $e_y = 12$ in. $< d = 21.5$ in., assume compression failure and follow the steps of Example 11.4 to get $a = 10.65$ in. and $c = a/0.8 = 13.31$ in. $> C_b = 12.72$ in. Thus, compression controls. Check

$$f'_s = 87\left(\frac{c - d'}{c}\right) = 70 \text{ ksi} > f_y$$

Therefore, $f_s' = 60$ ksi. Check

$$f_s = 87\left(\frac{d-c}{c}\right) = 53.53 \text{ ksi} < 60 \text{ ksi}$$

Calculate forces: $C_c = 0.85 f_c' ab = 724.2$ K, $C_s = A_s'(f_y - 0.85 f_c') = 212.4$ K, $T = A_s f_s = 203.95$ K, $P_{nx} = C_c + C_s - T = 732.6$ K. $P_{nx} > P_{bx}$, so this is a compression failure case as assumed.

$$\varepsilon_t = \left(\frac{d-c}{c}\right) 0.003 = 0.00185$$

$$\varepsilon_t < 0.002 \quad \phi = 0.65$$

$$P_{ux} = \phi P_{nx} = 476.2 \text{ K}$$

c. Take moments about A_s using Eq. 11.11,

$$d'' = 9.5 \text{ in.} \qquad e' = 21.5 \text{ in.}$$

$$P_{nx} = \frac{1}{e'}\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right] = 732.5 \text{ K}$$

2. Determine the uniaxial load capacity P_{ny} about the y-axis when $e_x = 8$ in. In this case, $b = 24$ in., $h = 16$ in., $d = 13.5$ in., $d' = 2.5$ in., and $A_s = A_s' = 3.81$ in.^{2} The solution will be performed using statics, as explained in step 1.

a. Balanced condition:

$$c_b = \left(\frac{87}{87 + f_y}\right) d = \left(\frac{87}{147}\right) 13.5 = 7.99 \text{ in.} \qquad a_b = 0.8(7.99) = 6.39 \text{ in.}$$

$$C_c = 0.85 f_c' ab = 651.8 \text{ K} \qquad f_s' = 87\left(\frac{c - d'}{c}\right) = 59.8 \text{ ksi}$$

$$C_s = A_s'(f_s' - 0.85 f_s') = 211.6 \text{ K} \qquad T = A_s f_y = 228.6 \text{ K}$$

In a balanced load, $P_{by} = C_c + C_s - T = 634.8$ K, $\phi P_{by} = 0.65\ P_{by} = 444.4$ K.

b. For $e_x = 8$ in., assume compression failure case and follow the steps of Example 11.4 to get $a = 6.65$ in. and $c = a/0.8 = 8.31$ in. $> c_b$ (compression failure). Check

$$f_s' = 87\left(\frac{c - d'}{c}\right) = 60.8 \text{ ksi}$$

Therefore, $f_s' = 60$ ksi. Check

$$f_s = 87\left(\frac{d-c}{c}\right) = 54.3 \text{ ksi}$$

Calculate forces: $C_c = 0.85 f_c' ab = 678.3$ K, $C_s = A_s'(60 - 0.85 f_c') = 212.4$ K, $T = A_s f_s = 206.9$ K, $P_{ny} = C_c + C_s - T = 683.3$ K, and $\phi P_{ny} = P_{uy} = 0.65\ P_{ny} = 444.5$ K. Because $P_{ny} > P_{by}$, compression failure occurs, as assumed.

$$\varepsilon_t = \left(\frac{d-c}{c}\right) 0.003 = 0.00187$$

$$\varepsilon_t < 0.002 \quad \phi = 0.65$$

$$P_{uy} = \phi P_{ny} = 444.5 \text{ K}$$

c. Take moments about A_s using Eq. 11.11:

$$d'' = 5.5 \text{ in.} \quad e' = 13.5 \text{ in.}$$

$$P_{ny} = \frac{1}{e'}\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right] = 684 \text{ K}$$

3. Determine the theoretical axial load P_{n_0}:

$$P_{n_0} = 0.85 f'_c A_g + A_{st}(f_y - 0.85 f'_c)$$

$$= 0.85(5)(16 \times 24) + 10.16(60 - 0.85 \times 5) = 2198.4 \text{ K} \quad \phi P_{n0} = 0.65 P_{n0} = 1429 \text{ K}$$

4. Using the Bresler equation (Eq. 11.42), multiply by 100:

$$\frac{100}{P_u} = \frac{100}{476.2} + \frac{100}{444.5} - \frac{100}{1429} = 0.365$$

$$P_u = 274 \text{ K} \quad \text{and} \quad P_n = \frac{P_u}{0.65} = 421.5 \text{ K}$$

Notes:

1. Approximate equations or the ACI charts may be used to calculate P_{nx} and P_{ny}. However, since the Bresler equation is an approximate solution, it is preferable to use accurate procedures, as was done in this example, to calculate P_{nx} and P_{ny}. Many approximations in the solution will produce inaccurate results. Computer programs based on statics are available and may be used with proper checking of the output.
2. In Example 11.18, the areas of the corner bars were used twice, once to calculate P_{nx} and once to calculate P_{ny}. The results obtained are consistent with similar solutions. A conservative solution is to use half of the corner bars in each direction, giving $A_s = A'_s = 2(1.27) = 2.54$ in.^{2}, which will reduce the values of P_{nx} and P_{ny}.

Example 11.19

Determine the nominal design load, P_n, for the column section of the previous example using the Parme load contour method; see Fig. 11.29.

Solution

1. Assume $\beta = 0.65$. The uniaxial load capacities in the direction of x- and y-axes were calculated in Example 11.18:

$$P_{ux} = 476.2 \text{ K} \quad P_{uy} = 444.5 \text{ K} \quad P_{nx} = 732.6 \text{ K} \quad P_{ny} = 683.8 \text{ K}$$

2. The moment capacity of the section about the x-axis is

$$M_{ox} = P_{nx} \cdot e_y = 732.6 \times 12$$

The moment capacity of the section about the y-axis is

$$M_{oy} = P_{ny} e_x = 683.8 \times 8 \text{ K·in.}$$

3. Let the nominal load capacity be P_n. The nominal design moment on the section about the x-axis is

$$M_{nx} = P_n e_y = P_n \times 12 \text{ K·in.}$$

and that about the y-axis is

$$M_{ny} = P_n e_x = 8 P_n$$

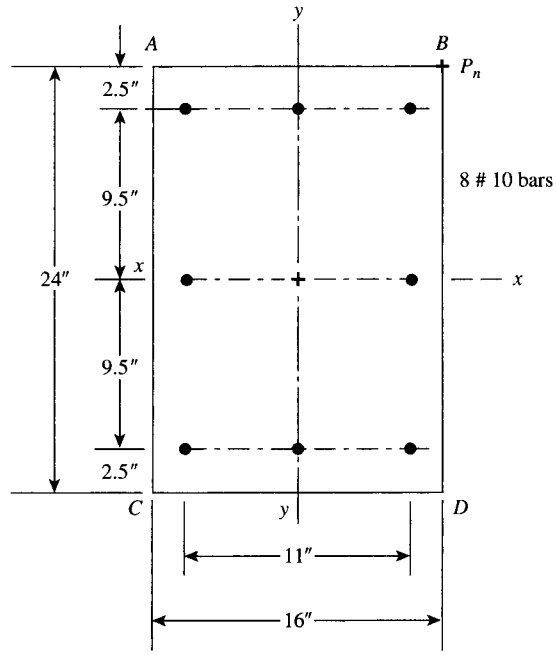


Figure 11.29 Example 11.19: biaxial load, PCA method: $P_n = 455$ K.

4. Check if $M_{ny}/M_{oy} > M_{nx}/M_{ox}$:

$$\frac{8P_n}{683.8 \times 8} > \frac{12P_n}{732.6 \times 12} \quad \text{or} \quad 1.463 \times 10^{-3}P_n > 1.365 \times 10^{-3}P_n$$

Then $M_{ny}/M_{oy} > M_{nx}/M_{ox}$. Therefore, use Eq. 11.48.

5. $$\frac{8P_n}{683.8 \times 8} + \frac{12P_n}{732.6 \times 12}\left(\frac{1-0.65}{0.65}\right) = 1$$

Multiply by 1000 to simplify calculations.

$$1.463P_n + 0.735P_n = 1000 \qquad P_n = 455 \text{ K}$$

$$P_u = \phi P_n = 295.75 \text{ K} \quad (\phi = 0.65)$$

Note that P_u is greater than the value of 274 K obtained by the Bresler reciprocal method (Eq. 11.42) in the previous example by about 8%.

11.18 EQUATION OF FAILURE SURFACE

A general equation for the analysis and design of reinforced concrete short and tied rectangular columns was suggested by Hsu [16]. The equation is supposed to represent the failure surface and interaction diagrams of columns subjected to combined biaxial bending and axial load, as

shown in Fig. 11.26. The axial load can be compressive or a tensile force. The equation is presented as follows:

$$\left(\frac{P_n - P_b}{P_o - P_b}\right) + \left(\frac{M_{nx}}{M_{bx}}\right)^{1.5} + \left(\frac{M_{ny}}{M_{by}}\right)^{1.5} = 1.0 \tag{11.40}$$

where

P_n = nominal axial strength (positive if compression and negative if tension) for a given eccentricity
P_o = nominal axial load (positive if compression and negative if tension) at zero eccentricity
P_b = nominal axial compressive load at balanced strain condition
M_{nx}, M_{ny} = nominal bending moments about the x- and y-axes, respectively
M_{bx}, M_{by} = nominal balanced bending moments about the x- and y-axes, respectively, at balanced strain conditions

To use Eq. 11.4, all terms must have a positive sign. The value of P_o was given earlier (Eq. 10.1):

$$P_o = 0.85 f'_c (A_g - A_{st}) + A_{st} \cdot f_y \tag{11.41}$$

The nominal balanced load, P_b, and the nominal balanced moment, $M_b = P_b e_b$, were given in Eq. 11.6 and 11.7, respectively, for sections with tension and compression reinforcement only. For other sections, these values can be obtained by using the principles of statics.

Note that the equation of failure surface can also be used for uniaxial bending representing the interaction diagram. In this case, the third term will be omitted when $e_x = 0$, and the second term will be omitted when $e_y = 0$.

When $e_x = 0$ (moment about the x-axis only),

$$\left(\frac{P_n - P_b}{P_o - P_b}\right) + \left(\frac{M_{nx}}{M_{bx}}\right)^{1.5} = 1.0 \tag{11.42}$$

(This is Eq. 11.18, given earlier.) When $e_y = 0$ (moment about the y-axis only),

$$\left(\frac{P_n - P_b}{P_o - P_b}\right) + \left(\frac{M_{ny}}{M_{by}}\right)^{1.5} = 1.0 \tag{11.43}$$

Applying Eq. 11.4 to Examples 11.2 and 11.4, $P_b = 453.4$ K, $M_{bx} = 6810.8$ K·in., $e_y = 10$ in., and $P_o = 0.85(4)(14 \times 22 - 8) + 8(60) = 1500$ K.

$$\frac{P_n - 453.4}{1500 - 453.4} + \left(\frac{10P_n}{6810.8}\right)^{1.5} = 1.0$$

Multiply by 1000 and solve for P_n:

$$(0.9555P_n - 433.2) + 0.05626P_n^{1.5} = 1000$$

$$0.9555P_n + 0.05626P_n^{1.5} = 1433.2$$

$P_n = 611$ K, which is close to that obtained by analysis.

Example 11.20

Determine the nominal design load, P_n, for the column section of Example 11.18 using the equation of failure surface.

Solution

1. Compute

$$\begin{aligned} P_o &= 0.85 f_c'(A_g - A_{st}) + A_{st} f_y \\ &= 0.85(5)(16 \times 24 - 10.16) + (10.16 \times 60) \\ &= 2198.4 \text{ K} \end{aligned}$$

2. Compute P_b and M_b using Eqs. 11.6 and 11.8 about the x- and y-axes, respectively.

a. About the x-axis,

$$a_{bx} = \frac{87 d_t}{87 + f_y} = \frac{87(21.5)}{87 + 60} = 12.72 \text{ in.}$$

$$a_{bx} = 0.8(12.72) = 10.18 \text{ in.}$$

$$f_s' = 87\left(\frac{c - d'}{c}\right) = 69.9 \text{ ksi} \quad f_s' = 60 \text{ ksi}$$

$$d_x'' = 9.5 \text{ in} \quad A_s = A_s' = 3.81 \text{ in.}^2$$

$$\begin{aligned} P_{bx} &= 0.85 f_c' a_x b + A_s'(f_y - 0.85 f_c') - A_s f_y \\ &= 0.85(5)(10.18)(16) + 3.81(60 - 0.85 \times 5) - 3.81(60) \\ &= 676.1 \text{ K} \end{aligned}$$

$$\begin{aligned} M_{bx} &= 0.85(5)(10.18)(16)\left(21.5 - \frac{10.18}{2} - 9.5\right) \\ &\quad + 3.81(60 - 0.85 \times 5) \times (21.5 - 2.5 - 9.5) + 3.81(60)(9.5) \\ &= 8973 \text{ K·in.} = 747.8 \text{ K·ft} \end{aligned}$$

b. About the y-axis: $d_t = 13.5$ in., $d_y'' = 5.5$ in., $A_s = A_s' = 3.81$ in.^{2}

$$c_{by} = \frac{87(13.5)}{87 + 60} = 7.99 \text{ in.}$$

$$a_{by} = 0.8(7.99) = 6.39 \text{ in.} \quad f_s' = 87\left(\frac{c - d'}{c}\right) = 59.8 \text{ ksi}$$

$$\begin{aligned} P_{by} &= 0.85(5)(6.39)(24) + 3.81(59.8 - 0.85 \times 5) - 3.81(60) \\ &= 634.8 \text{ K} \end{aligned}$$

$$\begin{aligned} M_{by} &= 0.85(5)(6.39)(24)\left(13.5 - \frac{6.39}{2} 5.5\right) \\ &\quad + 3.81(59.8 - 0.85 \times 5)(13.5 - 2.5 - 5.5) + 3.81(60)(5.5) \\ &= 5557.3 \text{ K} \cdot \text{in.} = 463 \text{ K} \cdot \text{ft} \end{aligned}$$

3. Compute the nominal balanced load for biaxial bending, P_{bb}:

$$\tan\alpha = \frac{M_{ny}}{M_{nx}} = \frac{P_n \cdot e_x}{P_n \cdot e_y} = \frac{e_x}{e_y} = \frac{8}{12} \quad \alpha = 33.7^\circ$$

$$\frac{P_{bx} - P_{by}}{90^\circ} = \frac{\Delta P_b}{90^\circ - \alpha^\circ} \quad \text{or} \quad \frac{676.1 - 634.8}{90} = \frac{\Delta P_b}{90 - 33.7}$$

$$\Delta P_b = 25.8 \text{ K}$$

$$P_{bb} = P_{by} + \Delta P_b = 634.8 + 25.8 = 660.6 \text{ K}$$

4. Compute P_n from the equation of failure surface:

$$\frac{P_n - 660.6}{2198.4 - 660.6} + \left(\frac{P_n \times 12}{8973}\right)^{1.5} + \left(\frac{P_n \times 8}{5557.3}\right)^{1.5} = 1.0$$

Multiply by 1000 and solve for P_n:

$$(0.65P_n - 429.85) + 0.0489P_n^{1.5} + 0.0546P_n^{1.5} = 1000$$

$$0.65P_n + 0.1035P_n^{1.5} = 1429.85$$

By trial, $P_n = 487$ K. Because $P_n < P_{bb}$, it is a tension failure case for biaxial bending, and thus $P_o = -2198.4$ K (to keep the first term positive).

$$1000\left(\frac{P_n - 660.9}{-2198.4 - 660.9}\right) + 0.0489P_n^{1.5} + 0.0546P_n^{1.5} = 1000$$

$$0.35P_n + 0.1035P_n^{1.5} = 769.1$$

$$P_n = 429 \text{ K} \quad \text{and} \quad P_u = 0.65P_n = 278.8 \text{ K}$$

Note: The strength capacity, ϕP_n, of the same rectangular section was calculated using the Bresler reciprocal equation (Example 11.18), Parme method (Example 11.19), and Hsu method (Example 11.20) to get $\phi P_n = 421.5$ K, 455 K, and 429 K, respectively. The Parme method gave the highest value for this example.

11.19 SI EXAMPLE

Example 11.21

Determine the balanced compressive forces P_b, e_b, and M_b for the section shown in Fig. 11.30. Use $f'_c = 30$ MPa and $f_y = 400$ MPA ($b = 350$ mm, $d = 490$ mm).

Solution

1. For a balanced condition, the strain in the concrete is 0.003 and the strain in the tension steel is $\varepsilon_y = f_y/E_s = 400/200{,}000 = 0.002$, where $E_s = 200{,}000$ MPa.

$$A_s = A'_s = 4(700) = 2800 \text{ mm}^2$$

2. Locate the neutral axis depth, c_b:

$$c_b = \left(\frac{600}{600 + f_y}\right) d_t \quad (\text{where } f_y \text{ is in MPa})$$

$$= \left(\frac{600}{600 + 420}\right)(490) = 288 \text{ mm}$$

$$a_b = 0.85c_b = 0.85 \times 288 = 245 \text{ mm}$$

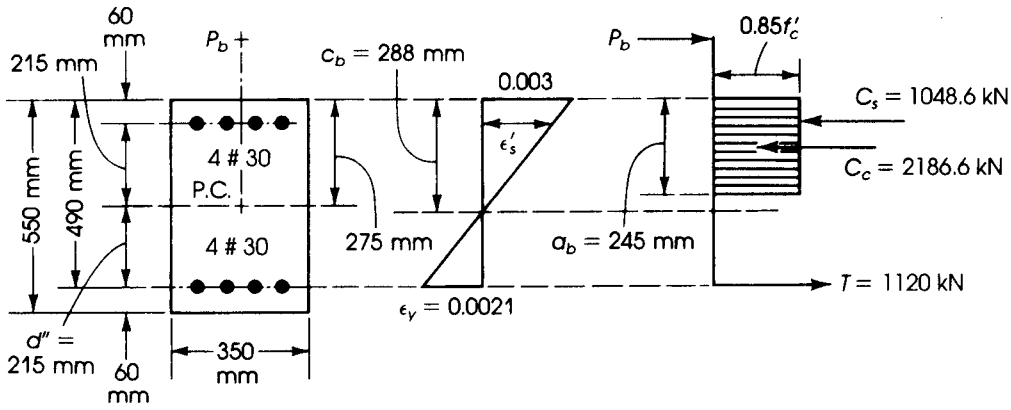


Figure 11.30 Example 11.21.

3. Check if compression steel yields. From the strain diagram,

$$\frac{\varepsilon'_s}{0.003} = \frac{c - d'}{c} = \frac{288 - 60}{288}$$

$$\varepsilon'_s = 0.00238 > \varepsilon_y$$

Therefore, compression steel yields.

4. Calculate the forces acting on the section:

$$C_c = 0.85 f'_c ab = \frac{0.85}{1000} \times 30 \times 245 \times 350 = 2186.6 \text{ kN}$$

$$T = A_s f_y = 2800 \times 0.400 \times 1120 \text{ kN}$$

$$C_s = A'_s(f_y - 0.85 f'_c) = \frac{2800 \text{ mm}^2}{1000}(400 - 0.85 \times 30) = 1048.6 \text{ kN}$$

5. Calculate P_b and M_b:

$$P_b = C_c + C_s - T = 2115.2 \text{ kN}$$

From Eq. 11.10,

$$M_b = P_b e_b = C_c\left(d - \frac{a}{2} - d''\right) + C_s(d - d' - d'') + Td''$$

The plastic centroid is at the centroid of the section and $d'' = 215$ mm.

$$M_b = 2186.6\left(490 - \frac{245}{2} - 215\right) + 1048.6(490 - 60 - 215)$$

$$+ 1120 \times 215 - 799.7 \text{ kN} \cdot \text{m}$$

$$e_b = \frac{M_b}{P_b} = \frac{799.7}{2115.2} = 0.378 \text{ m} = 378 \text{ mm}$$

SUMMARY

Sections 11.1–11.3

1. The plastic centroid can be obtained by determining the location of the resultant force produced by the steel and the concrete, assuming both are stressed in compression to f_y and $0.85f'_c$, respectively.
2. On a load–moment interaction diagram the following cases of analysis are developed:
   a. Axial compression, P_o
   b. Maximum nominal axial load, $P_{n\,\max} = 0.8\ P_o$ (for tied columns) and $P_{n\,\max} = 0.85P_o$ (for spiral columns)
   c. Compression failure occurs when $P_n > P_b$ or $e < e_b$
   d. Balanced condition, P_b and M_b
   e. Tension failure occurs when $P_n < P_b$ or $e > e_b$
   f. Pure flexure

Section 11.4

1. For compression-controlled sections, $\phi = 0.65$, while for tension-controlled section, $\phi = 0.9$.
2. For the transition region,

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) \quad \text{(for tied columns)}$$

$$\phi = 0.75 + (\varepsilon_t - 0.002)(50) \quad \text{(for spiral columns)}$$

Section 11.5

For a balanced section,

$$c_b = \frac{87d_t}{87 + f_y} \quad \text{and} \quad a_b = \beta_1 c_b$$

$$\beta_1 = 0.85 \text{ for} f'_c \leq 4 \text{ ksi}$$

$$P_b = C_c + C_s - T = 0.85f'_c ab + A'_s(f_y - 0.85f'_c) - A_s f_y$$

$$M_b = P_b e_b = C_c\left(d - \frac{a}{2} - d''\right) + Td'' + C_s(d - d' - d'')$$

$$e_b = \frac{M_b}{P_b}$$

Section 11.6

The equations for the general analysis of rectangular sections under eccentric forces are summarized.

Sections 11.7–11.8

Examples for the cases when tension and compression controls are given.

Sections 11.9–11.10

Examples are given for the interaction diagram and for the case when side bars are used.

Section 11.11

This section gives the load capacity of circular columns. The cases of a balanced section when compression controls are explained by examples.

Section 11.12

This section gives examples of the analysis and design of columns using charts.

Section 11.13

This section gives examples of the design of column sections.

Sections 11.14–11.18

Biaxial bending:

1. For circular columns with uniform reinforcement,

$$M_u = \sqrt{(M_{ux})^2 + (M_{uy})^2} \quad e = \sqrt{(e_x)^2 + (e_y)^2}$$

2. For square and rectangular sections,

$$\frac{1}{P_n} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_{n_0}}$$

$$\frac{M_{nx}}{M_{ox}} + \frac{M_{ny}}{M_{oy}} \leq 1.0$$

3. In the Bresler load contour method,

$$\left(\frac{M_{nx}}{M_{ox}}\right)^{1.5} + \left(\frac{M_{ny}}{M_{oy}}\right)^{1.5} = 1.0$$

4. In the PCA load contour method,

$$M_{ny} + M_{nx}\left(\frac{b}{h}\right)\left(\frac{1-\beta}{\beta}\right) = M_{oy}$$

$$M_{nx} + M_{ny}\left(\frac{h}{b}\right)\left(\frac{1-\beta}{\beta}\right) = M_{ox}$$

5. Equations of failure surface method are given with applications.

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PROBLEMS

Note: For all problems, use $f_y = 60$ ksi, $d' = 2.5$ in., and $A_s = A'_s$ where applicable. Slight variation in answers are expected.

11.1 (Rectangular sections: balanced condition) For the rectangular column sections given in Table 11.3, determine the balanced compressive load, P_b, the balanced moment, M_b, and the balanced eccentricity, e_b, for each assigned problem. (Answers are given in Table 11.3.) ($\phi = 0.65$.)

11.2 (Rectangular sections: compression failure) For the rectangular column sections given in Table 11.3, determine the load capacity, P_n, for each assigned problem when the eccentricity is $e = 6$ in. (Answers are given in Table 11.3.)

11.3 (Rectangular sections: tension failure) For the rectangular column sections given in Table 11.3, determine the load capacity, P_n, for each assigned problem when the eccentricity is $e = 24$ in. (Answers are given in Table 11.3.)

11.4 (Rectangular sections with side bars) Determine the load capacity, ϕP_n, for the column section shown in Fig. 11.31 considering all side bars when the eccentricity is $e_y = 8$ in. Use $f'_c = 4$ ksi and $f_y = 60$ ksi. (Answer: 658 K.)

11.5 Repeat Problem 11.4 with Fig. 11.32. (Answer: 660 K.)

11.6 Repeat Problem 11.4 with Fig. 11.33. (Answer: 368 K.)

11.7 Repeat Problem 11.4 with Fig. 11.34. (Answer: 822 K.)

Table 11.3 Answers for Problems 11.1–11.3

					Answers to Problems			
					11.1		11.2	11.3
Number	f'_c(ksi)	b (in.)	h (in.)	$A_s = A'_s$	P_b	e_b	P_n ($e = 6$ in.)	P_n ($e = 24$ in.)
(a)	4	20	20	6 no. 10	572	17.4	1193	395
(b)	4	14	14	4 no. 8	249	10.9	407	93
(c)	4	24	24	8 no. 10	848	20.1	1860	696
(d)	4	18	26	6 no. 10	698	20.6	1528	591
(e)	4	12	18	4 no. 9	305	15.2	592	176
(f)	4	14	18	4 no. 10	354	16.2	715	221
(g)	5	16	16	5 no. 10	406	15.3	807	228
(h)	5	18	18	5 no. 9	540	12.5	930	230
(i)	5	14	20	4 no. 9	476	13.4	847	221
(j)	5	16	22	4 no. 10	606	14.8	1140	327
(k)	6	16	24	5 no. 10	746	16.8	1532	476
(l)	6	14	20	4 no. 9	534	12.8	944	226

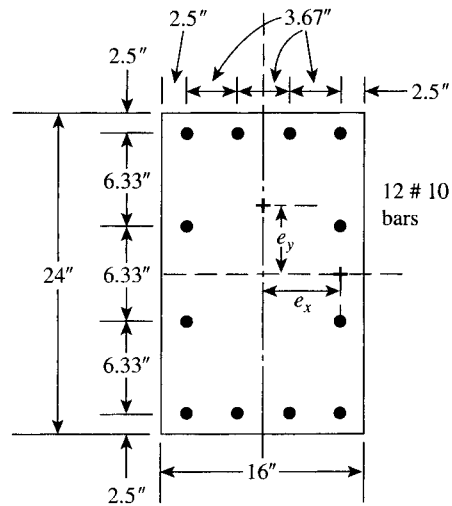


Figure 11.31 Problem 11.4.

11.8 (Design of rectangular column sections) For each assigned problem in Table 11.4, design a rectangular column section to support the factored load and moment shown. Determine A_s, A'_s, and h if not given; then choose adequate bars considering that $A_s = A'_s$. The final total steel ratio, ρ_g, should be close to the given values where applicable. Check the load capacity, ϕP_n, of the final section using statics and equilibrium equations. One solution for each problem is given in Table 11.4.

11.9 (ACI charts) Repeat Problems 11.2*b*, 11.2*d*, 11.2*f*, 11.8*a*, 11.8*c*, and 11.8*e* using the ACI charts.

11.10 (Circular columns: balanced condition) Determine the balanced load capacity, ϕP_b, the balanced moment, ϕM_b, and the balanced eccentricity, e_b, for the circular tied sections shown in Fig. 11.35. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

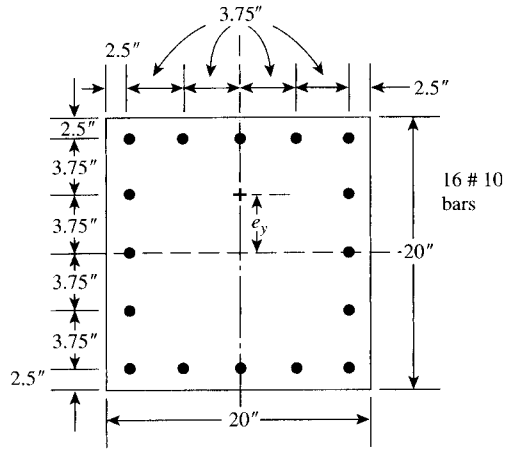


Figure 11.32 Problem 11.5.

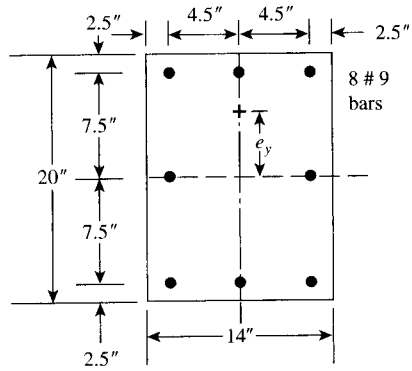


Figure 11.33 Problem 11.6.

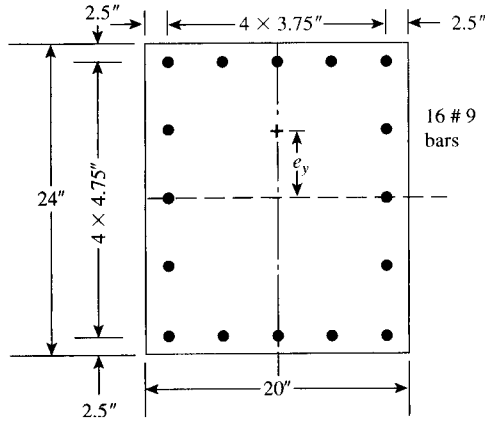


Figure 11.34 Problem 11.7.

Table 11.4 Problem 11.8

Number	f'_c(ksi)	P_u (K)	M_u (K·ft)	b (in.)	h (in.)	ρ_g %	One Solution	
							h (in.)	$A_s = A'_s$
(a)	4	530	353	16	—	4.0	20	5 no. 10
(b)	4	410	205	14	18	—	18	5 no. 8
(c)	4	480	640	18	—	3.5	24	6 no. 10
(d)	4	440	440	20	20	—	20	6 no. 9
(e)	4	1125	375	20	24	—	24	6 no. 10
(f)	4	710	473	18	—	3.0	24	5 no. 10
(g)	5	300	300	14	—	2.0	20	3 no. 9
(h)	5	1000	665	20	26	—	26	6 no. 10
(i)	6	590	197	14	—	2.0	18	2 no. 10
(j)	6	664	332	16	20	—	20	4 no. 9

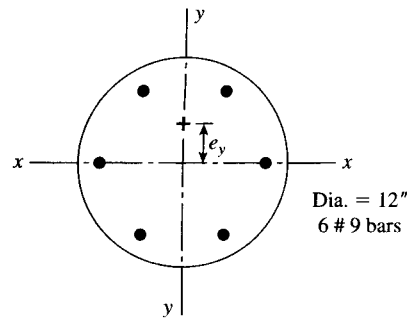


Figure 11.35 Problem 11.10.

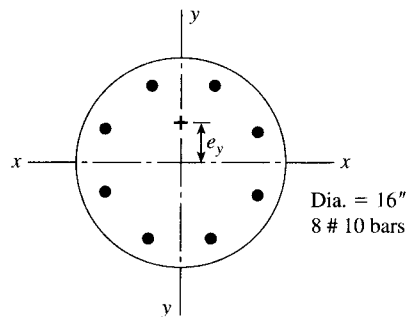


Figure 11.36 Problem 11.11.

11.11 Repeat Problem 11.10 for Fig. 11.36.

11.12 Repeat Problem 11.10 for Fig. 11.37.

11.13 Repeat Problem 11.11 for Fig. 11.38.

11.14 (Circular columns) Determine the load capacity, ϕP_n, for the circular tied column sections shown in Figs. 11.35 through 11.38 when the eccentricity is $e_y = 6$ in. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

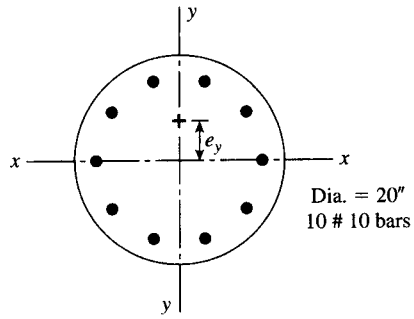


Figure 11.37 Problem 11.12.

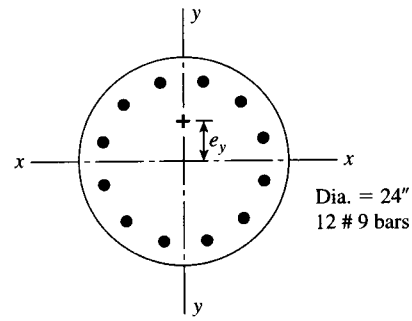


Figure 11.38 Problem 11.13.

11.15 (Biaxial bending) Determine the load capacity, P_n, for the column sections shown in Figs. 11.31 through 11.34 if $e_y = 8$ in. and $e_x = 6$ ini. using the Bresler reciprocal method. Use f'_c(4 ksi) and $f_y = 60$ ksi. For each problem the values of P_{nx}, P_{ny}, P_{n0} (P_{bx}, M_{bx}), and (P_{by}, M_{by}) are as follows:

- **a.** Figure 11.31: 952 K, 835 K, 2168 K (571 K, 792 K·ft), (536 K, 483 K·ft)
- **b.** Figure 11.32: 930 K, 1108 K, 2505 K (577 K, 742 K·ft), (577 K, 742 K·ft)
- **c.** Figure 11.33: 558 K, 495 K, 1408 K (408 K, 414 K·ft), (368 K, 260 K·ft)
- **d.** Figure 11.34: 1093 K, 1145 K, 2538 K (718 K, 865 K·ft), (701 K, 699 K·ft)

11.16 Repeat Problem 11.15 using the Parme method.

11.17 Repeat Problem 11.15 using the Hsu method.

11.18 For the column sections shown in Fig. 11.31, determine

- **a.** The uniaxial load capacities about the x- and y-axes, P_{nx} and P_{ny} using $e_y = 6$ in. and $e_x = 6$ in.
- **b.** The uniaxial balanced load and moment capacities about the x- and y-axes, P_{bx}, P_{by}, M_{bx}, and M_{by}.
- **c.** The axial load, P_{n0}.
- **d.** The biaxial load capacity P_n when $e_y = e_x = 6$ in., using the Bresler reciprocal method, the Hsu method, or both.

11.19 Repeat Problem 11.18 for Fig. 11.32.

11.20 Repeat Problem 11.18 for Fig. 11.33.

11.21 Repeat Problem 11.18 for Fig. 11.34.

CHAPTER 12

SLENDER COLUMNS



Columns in a high-rise building, Toronto, Canada.

12.1 INTRODUCTION

In the analysis and design of short columns discussed in the previous two chapters, it was assumed that buckling, elastic shortening, and secondary moment due to lateral deflection had minimal effect on the ultimate strength of the column; thus, these factors were not included in the design procedure. However, when the column is long, these factors must be considered. The extra length will cause a reduction in the column strength that varies with the column effective height, width of the section, the slenderness ratio, and the column end conditions.

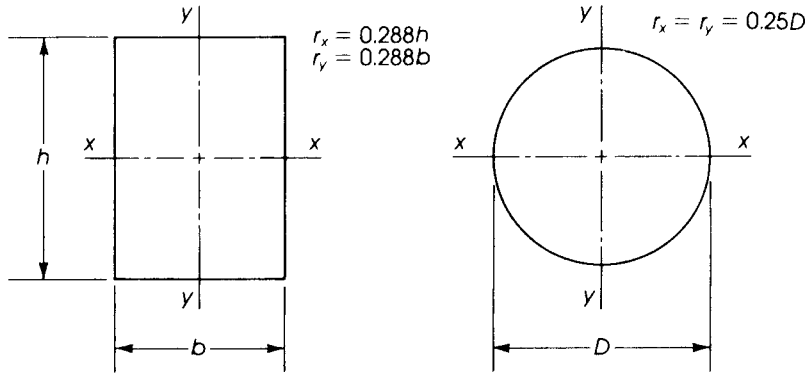


Figure 12.1 Rectangular and circular sections of columns, with radius of gyration r.

A column with a high slenderness ratio will have a considerable reduction in strength, whereas a low slenderness ratio means that the column is relatively short and the reduction in strength may not be significant. The slenderness ratio is the ratio of the column height, l, to the radius of gyration, r, where $r = I/A$, I being the moment of inertia of the section and A the sectional area.

For a rectangular section of width b and depth h (Fig. 12.1), $I_x = bh^3/12$ and $A = bh$. Therefore, $r_x = I/A = 0.288h$ (or, approximately, $r_x = 0.3h$). Similarly, $I_y = hb^3/12$ and $r_y = 0.288b$ (or, approximately, $0.3b$). For a circular column with diameter D, $I_x = I_y = \pi D^2/64$ and $A = \pi D^2/4$; therefore, $r_x = r_y = 0.25D$.

In general, columns may be considered as follows:

1. Long with a relatively high slenderness ratio, where lateral bracing or shear walls are required.
2. Long with a medium slenderness ratio that causes a reduction in the column strength. Lateral bracing may not be required, but strength reduction must be considered.
3. Short where the slenderness ratio is relatively small, causing a slight reduction in strength. This reduction may be neglected, as discussed in previous chapters.

12.2 EFFECTIVE COLUMN LENGTH (Kl_u)

The slenderness ratio l/r can be calculated accurately when the effective length of the column (Kl_u) is used. This effective length is a function of two main factors:

1. The unsupported length, l_u, represents the unsupported height of the column between two floors. It is measured as the clear distance between slabs, beams, or any structural member providing lateral support to the column. In a flat slab system with column capitals, the unsupported height of the column is measured from the top of the lower floor slab to the bottom of the column capital. If the column is supported with a deeper beam in one

direction than in the other direction, l_u should be calculated in both directions (about the x- and y-axes) of the column section. The critical (greater) value must be considered in the design.

2. The effective length factor, K, represents the ratio of the distance between points of zero moment in the column and the unsupported height of the column in one direction. For example, if the unsupported length of a column hinged at both ends, on which sidesway is prevented, is l_u, the points of zero moment will be at the top and bottom of the column—that is, at the two hinged ends. Therefore, the factor $K = l_u/l_u$ is 1.0. If a column is fixed at both ends and sidesway is prevented, the points of inflection (points of 0 moment) are at $l_u/4$ from each end. Therefore, $K = 0.5l_u/l_u = 0.5$ (Fig. 12.2). To evaluate the proper value of K, two main cases are considered.

   When structural frames are braced, the frame, which consists of beams and columns, is braced against sidesway by shear walls, rigid bracing, or lateral support from an adjoining structure. The ends of the columns will stay in position, and lateral translation of joints is prevented. The range of K in braced frames is always equal to or less than 1.0. The ACI Code, Section 10.10, recommends the use of $K = 1.0$ for braced frames.

   When the structural frames are unbraced, the frame is not supported against sidesway, and it depends on the stiffness of the beams and columns to prevent lateral deflection. Joint translations are not prevented, and the frame sways in the direction of lateral loads. The range of K for different columns and frames is given in Fig. 12.2, considering the two cases when sidesway is prevented or not prevented.

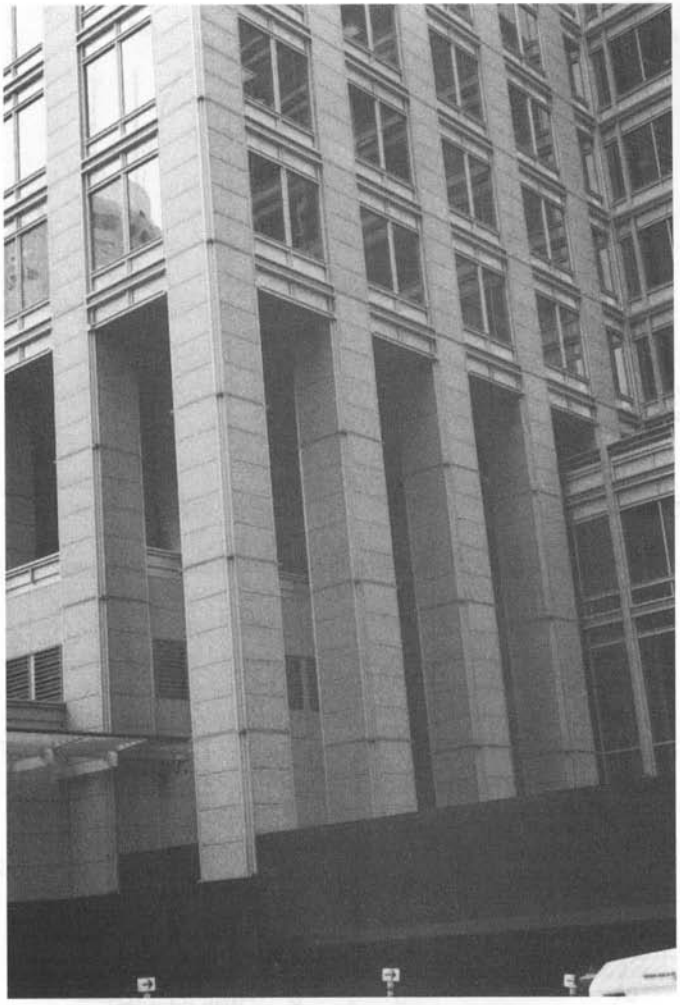
12.3 EFFECTIVE LENGTH FACTOR (K)

The effective length of columns can be estimated by using the alignment chart shown in Fig. 12.3 [10]. To find the effective length factor K, it is necessary first to calculate the end restraint factors Ψ_A and ψ_B at the top and bottom of the column, respectively, where

$$\psi = \frac{\Sigma EI/l_c \text{ of columns}}{\Sigma EI/l \text{ of beams}} \tag{12.1}$$

(both in the plane of bending) where l_c = length center to center of joints in a frame and l = span length, center to center of joints. The ψ factor at one end shall include all columns and beams meeting at the joint. For a hinged end, ψ is infinite and may be assumed to be 10.0. For a fixed end, ψ is zero and may be assumed to be 1.0. Those assumed values may be used because neither a perfect frictionless hinge nor perfectly fixed ends can exist in reinforced concrete frames.

The procedure for estimating K is to calculate ψ_A for the top end of the column and ψ_B for the bottom end of the column. Plot ψ_A and ψ_B on the alignment chart of Fig. 12.3 and connect the two points to intersect the middle line, which indicates the K-value. Two nomograms are shown, one for braced frames where sidesway is prevented, and the second for unbraced frames, where sidesway is not prevented. The development of the charts is based on the assumptions that (1) the structure consists of symmetrical rectangular frames, (2) the girder moment at a joint is distributed to columns according to their relative stiffnesses, and (3) all columns reach their critical loads at the same time.



Long columns in an office building.

12.4 MEMBER STIFFNESS (*EI*)

The stiffness of a structural member is equal to the modulus of elasticity E times the moment of inertia I of the section. The values of E and I for reinforced concrete members can be estimated as follows:

1. The modulus of elasticity of concrete was discussed in Chapter 2; the ACI Code gives the following expression:

$$E_c = 33w^{1.5}\sqrt{f'_c} \quad \text{or} \quad E_c = 57{,}000\sqrt{f'_c} \text{ (psi)}$$

for normal-weight concrete. The modulus of elasticity of steel is $E_s = 29 \times 10^6$ psi.

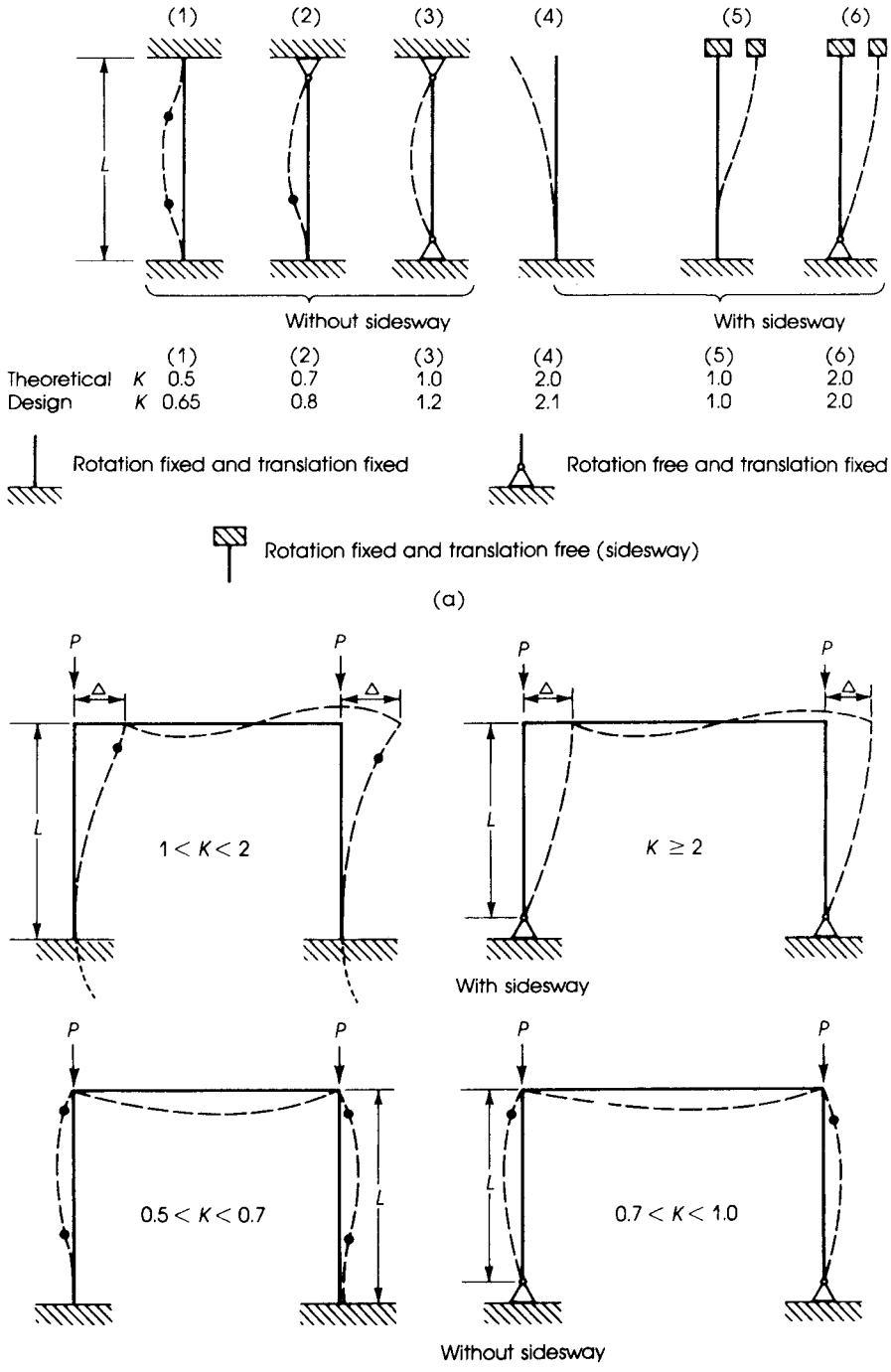


Figure 12.2 (*a*) Effective lengths of columns and length factor K and (*b*) effective lengths and K for portal columns.

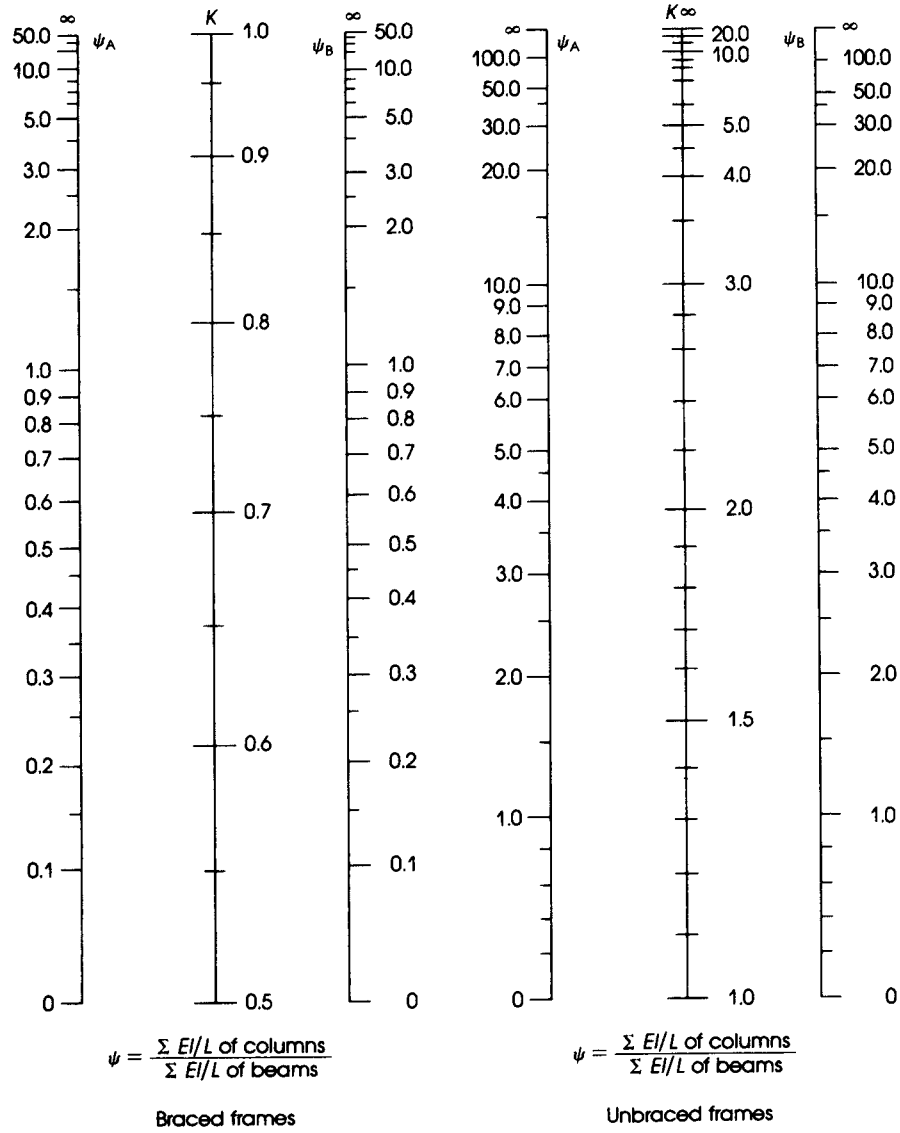


Figure 12.3 Alignment chart.

2. For reinforced concrete members, the moment of inertia I varies along the member, depending on the degree of cracking and the percentage of reinforcement in the section considered.

   To evaluate the factor ψ, EI must be calculated for beams and columns. For this purpose, I can be estimated as follows (ACI Code, Section 10.4.4.1):

   a. Compression members:

   Columns $I = 0.70I_g$

   Walls—Uncracked $I = 0.70I_g$

   —(Cracked) $I = 0.35I_g$

b. Flexural members:

Beams $I = 0.35I_g$

Flat plates and flat slabs $I = 0.25I_g$

Alternatively, the moments of inertia of compression and flexural members, I shall be permitted to be computed as follows:

c. Compression members:

$$I = \left(0.80 + 25\frac{A_{st}}{A_g}\right)\left(1 - \frac{M_u}{P_u h} - 0.5\frac{P_u}{P_o}\right) I_g \leq 0.875I_g \tag{12.2}$$

where P_u and M_u shall be determined from the particular load combination under consideration, or the combination of P_u and M_u determined in the smallest value of I, I need not be taken less than $0.35I_g$.

d. Flexural members:

$$I = (0.10 + 25\rho)\left(1.2 - 0.2\frac{b_w}{d}\right) I_g \leq 0.5I_g \tag{12.3}$$

where I_g = the moment of inertia of the gross concrete section about the centroidal axis, neglecting reinforcement.

$$\rho = \text{ratio of } A_s/bd \text{ in cross section}$$

The moment of inertia of T-beams should be based on the effective flange width defined in Section 3.15.2. It is generally sufficiently accurate to take I_g of a T-beam as two times the I_g of the web, $2(b_w h^3/12)$.

If the factored moments and shears from an analysis based on the moment of inertia of a wall, taken equal to $0.70I_g$, indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with $I = 0.35I_g$ in those stories where cracking is predicted using factored loads.

The values of the moments of inertia were dervied for nonprestressed members. For prestressed members, the moments of inertia may differ depending on the amount, location, and type of the reinforcement and the degree of cracking prior to ultimate. The stifness value for prestressed concrete members should include an allowance for the variabilty of the stifnesses.

For continuous flexural members, I shall be permitted to be taken as the average of values obtained from Eq. (12.3) for the critical positive and negative moment sections. I need not be taken less than $0.25I_g$.

The cross-sectional dimensions and reinforcement ratio used in the above formulas shall be within 10 percent of the dimensions and reinforcement ratio shown on the design drawings or the stiffness evaluation shall be repeated.

3. Area, $A = 1.0A_g$ (gross-sectional area).
4. The moments of inertia shall be divided by $(1 + \beta_{dns})$ when, sustained lateral loads act on the structure or for stability check, where

$$\beta_{dns} = \frac{\text{maximum factored axial sustained load}}{\text{maximum factored axial load}} = \frac{1.2D \text{ (sustained)}}{1.2D + 1.6L} \leq 1.0 \tag{12.4}$$

12.5 LIMITATION OF THE SLENDERNESS RATIO (Kl_u/r)

12.5.1 Nonsway Frames

The ACI Code, Section 10.10.1 recommends the following limitations between short and long columns in braced (nonsway) frames:

1. The effect of slenderness may be neglected and the column may be designed as a short column when

$$\frac{Kl_u}{r} \leq 34 - \frac{12M_1}{M_2} \tag{12.5}$$

where M_1 and M_2 are the factored end moments of the column and M_2 is greater than M_1.

2. The ratio M_1/M_2 is considered positive if the member is bent in single curvature and negative for double curvature (Fig. 12.4).
3. The term $(34 - 12M_1/M_2)$ shall not be taken greater than 40.
4. If the factored column moments are zero or $e = M_u/P_u < e_{\min}$, the value of M_2 should be calculated using the minimum eccentricity given by ACI Code Section 10.10.6.5:

$$e_{\min} = (0.6 + 0.03h) \quad \text{(inch)} \tag{12.6}$$

$$M_2 = P_u(0.6 + 0.03h) \tag{12.7}$$

where M_2 is the minimum moment. The moment M_2 shall be considered about each axis of the column separately. The value of K may be assumed to be equal to 1.0 for a braced frame unless it is calculated on the basis of EI analysis.

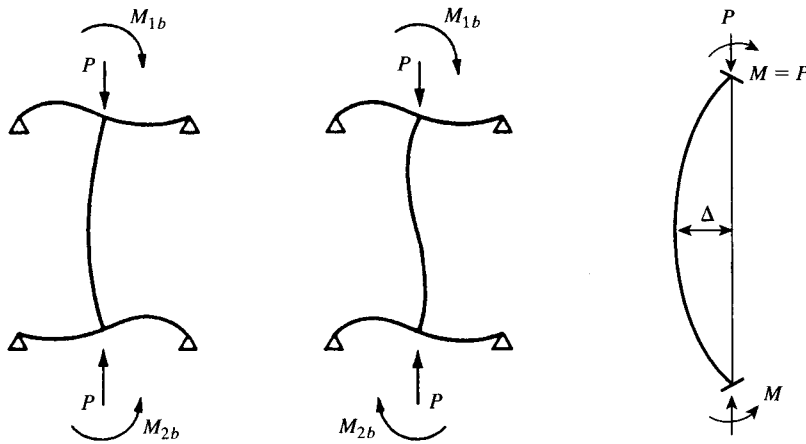


Figure 12.4 Single and double curvatures.

5. It shall be permitted to consider compression members braced against sidesway when bracing elements have a total stiffness, resisting lateral movement of that story, of at least 12 times the gross stiffness of the columns within the story.

12.5.2 Sway Frames

In compression members not braced (sway) against sidesway, the effect of the slenderness ratio may be neglected when

$$\frac{Kl_u}{r} < 22 \qquad \text{(ACI Code Section 10.10.1)} \tag{12.8}$$

12.6 MOMENT-MAGNIFIER DESIGN METHOD

12.6.1 Introduction

The first step in determining the design moments in a long column is to determine whether the frame is braced or unbraced against sidesway. If lateral bracing elements, such as shear walls and shear trusses, are provided or the columns have substantial lateral stiffness, then the lateral deflections produced are relatively small and their effect on the column strength is substantially low. It can be assumed that a story within a structure is nonsway if

$$Q = \frac{\Sigma P_u \Delta_0}{V_{us} l_c} \leq 0.05 \tag{12.9}$$

where ΣP_u and V_{us} are the story total vertical load and story shear, respectively, and Δ_0 is the first-order relative deflection between the top and bottom of the story due to V_{us}. The length l_c is that of the compression member in a frame, measured from center to center of the joints in the frame.

In general, compression members may be subjected to lateral deflections that cause secondary moments. If the secondary moment, M', is added to the applied moment on the column, M_a, the final moment is $M = M_a + M'$. An approximate method for estimating the final moment M is to multiply the applied moment M_a by a factor called the *magnifying moment factor* δ, which must be equal to or greater than 1.0, or $M_{max} = \delta M_a$ and $\delta \geq 1.0$. The moment M_a is obtained from the elastic structural analysis using factored loads, and it is the maximum moment that acts on the column at either end or within the column if transverse loadings are present.

If the P-Δ effect is taken into consideration, it becomes necessary to use a second-order analysis to account for the nonlinear relationship between the load, lateral displacement, and the moment. This is normally performed using computer programs. The ACI Code permits the use of first-order analysis of columns. The ACI Code *moment-magnifier design method* is a simplified approach for calculating the moment-magnifier factor in both braced and unbraced frames.

12.6.2 Magnified Moments in Nonsway Frames

The effect of slenderness ratio Kl_u/r in a compression member of a braced frame may be ignored if $Kl_u/r \leq 34 - 12M_1/M_2$, as given in Section 12.5.1. If Kl_u/r is greater than $(34 - 12M_1/M_2)$, then slenderness effect must be considered. The procedure for determining the magnification factor δ_{ns} in nonsway frames can be summarized as follows (ACI Code, Section 10.10.6):

1. Determine if the frame is braced against sidesway and find the unsupported length, l_u, and the effective length factor, K (K may be assumed to be 1.0).

2. Calculate the member stiffness, EI, using the reasonably approximate equation

$$EI = \frac{0.2E_cI_g + E_sI_{se}}{1 + \beta_{dns}} \tag{12.10}$$

or the more simplified approximate equation

$$EI = \frac{0.4E_cI_g}{1 + \beta_{dns}} \tag{12.11}$$

$$EI = 0.25E_cI_g \quad (\text{for } \beta_{dns} = 0.6) \tag{12.12}$$

where

$E_c = 57{,}000\sqrt{f'_c}$
$E_s = 29 \times 10^6$ psi
I_g = gross moment of inertia of the section about the axis considered, neglecting A_s
I_{se} = moment of inertia of the reinforcing steel

$$\beta_{dns} = \frac{\text{maximum factored axial sustained load}}{\text{maximum factored axial load}} = \frac{1.2D \text{ (sustained)}}{1.2D + 1.6L}$$

Note: The above β_{dns} is the ratio used to compute magnified moments in columns due to sustained loads.

Equations 12.11 and 12.12 are less accurate than Eq. 12.10. Moreover, Eq. 12.12 is obtained by assuming $\beta_d = 0.6$ in Eq. 12.11.
For improved accuracy EI can be approximated using suggested E and I values provided by:

$$I = \left(0.80 + 25\frac{A_{st}}{A_g}\right)\left(1 - \frac{M_u}{P_u h} - 0.5\frac{P_u}{p_o}\right) I_g \leq 0.875\, I_g$$

I need not be taken less than $0.35I_g$
where

A_{st} = Total area of longitudinal reinforcement (in.^{2})
P_o = nominal axial strenght at zero eccentricity (lb)
P_u = Factored axial force (+ve for compression) (lb)
M_u = Factored moment at section (lb.in.)
h = thikness of member (in.)

3. Determine the Euler buckling load, P_c:

$$P_c = \frac{\pi^2 EI}{(Kl_u)^2} \tag{12.13}$$

Use the values of EI, K, and l_u as calculated from steps 1 and 2.

4. Calculate the value of the factor C_m to be used in the equation of the moment-magnifier factor. For braced members without transverse loads,

$$C_m = 0.6 + \frac{0.4M_1}{M_2} \geq 0.4 \tag{12.14}$$

where M_1/M_2 is positive if the column is bent in single curvature. For members with transverse loads between supports, C_m shall be taken as 1.0.

5. Calculate the moment magnifier factor δ_{ns}:

$$\delta_{ns} = \frac{C_m}{1 - (P_u/0.75P_c)} \geq 1.0 \tag{12.15}$$

where P_u is the applied factored load and P_c and C_m are as calculated previously.

6. Design the compression member using the axial factored load, P_u, from the conventional frame analysis and a magnified moment, M_c, computed as follows:

$$M_c = \delta_{ns} M_2 \tag{12.16}$$

where M_2 is the larger factored end moment due to loads that result in no sidesway and should be $\geq P_u(0.6 + 0.03\,h)$. For frames braced against sidesway, the sway factor is $\delta_s = 0$. In nonsway frames, the lateral deflection is expected to be less than or equal to $H/1500$, where H is the total height of the frame.

12.6.3 Magnified Moments in Sway Frames

The effect of slenderness may be ignored in sway (unbraced) frames when $Kl_u/r < 22$. The procedure for determining the magnification factor, δ_s, in sway (unbraced) frames may be summarized as follows (ACI Code, Section 10.10.7):

1. Determine if the frame is unbraced against sidesway and find the unsupported length l_u and K, which can be obtained from the alignment charts (Fig. 12.3).

2–4. Calculate EI, P_c, and C_m as given by Eqs. 12.2, 12.10 through 12.14. Note that β_{dns} (to calculate I) is the ratio of maximum factored sustained shear within a story to the total factored shear in that story.

5. Calculate the moment-magnifier factor, δ_s using one of the following methods:

a. Magnifier method

$$\delta_s = \frac{1}{1 - (\Sigma P_u/0.75\Sigma P_c)} \geq 1.0 \tag{12.17}$$

where $\delta_s \leq 2.5$ and ΣP_u is the summation for all the factored vertical loads in a story and ΣP_c is the summation for all sway-resisting columns in a story. Also,

$$\delta_s M_s = \frac{M_s}{1 - (\Sigma P_u/0.75\Sigma P_c)} \geq M_s \tag{12.18}$$

where M_s is the factored end moment due to loads causing appreciable sway.

b. Approximate second order analysis

$$\delta_s = \frac{1}{1 - Q} \geq 1 \quad \text{or} \quad \delta_s M_s = \frac{M_s}{1 - Q} \geq M_s \tag{12.19}$$

where

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us} l_c} \tag{12.20}$$

where

P_u = Factored axial load (lb)

Δ_o = Revative lateral deflection between the top and bottom of a story due to lateral forces using a first order elastic frame analysis

V_{us} = Factored horizontal shear in a story (lb)
l_c = Length of compression member in a frame (m.)

If δ_s exceeds 1.5, δ_s shall be calculated using second order elastic analysis or the magnifier method described in a.

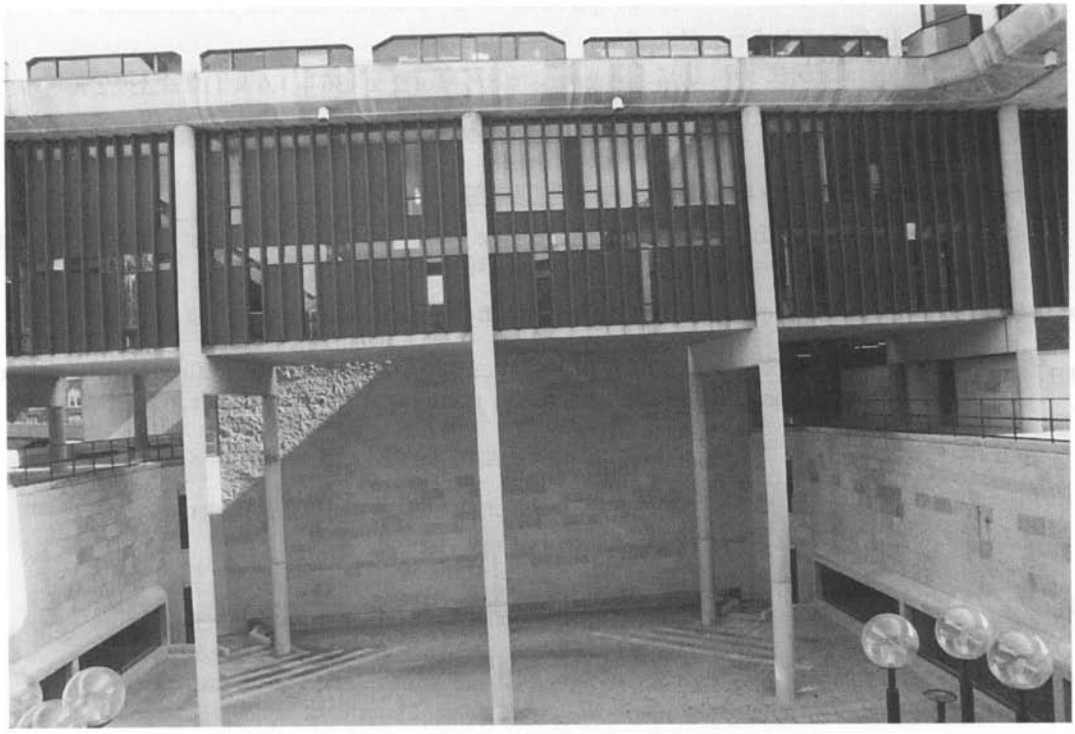
6. Calculate the magnified end moments M_1 and M_2 at the ends of an individual compression member, as follows:

$$M_1 = M_{1\,ns} + \delta_s M_{1s} \tag{12.21}$$

$$M_2 = M_{2\,ns} + \delta_s M_{2s} \tag{12.22}$$

where $M_{1\,ns}$ and $M_{2\,ns}$ are the moments obtained from the no-sway condition, whereas M_{1s} and M_{2s} are the moments obtained from the sway condition. If M_2 is greater than M_1 from structural analysis, then the design magnified moment is

$$M_c = M_{2\,ns} + \delta_s M_{2s} \tag{12.23}$$



Columns, University of Wisconsin, Madison, Wisconsin.

Example 12.1

The column section shown in Fig. 12.5 carries an axial load $P_D = 136\,\text{K}$ and a moment $M_D = 116\,\text{K·ft}$ due to dead load and an axial load $P_L = 110\,\text{K}$ and a moment $M_L = 93\,\text{K·ft}$ due to live load. The column is part of a frame that is braced against sidesway and bent in single curvature about its major axis. The unsupported length of the column is $l_c = 19\,\text{ft}$, and the moments at both ends of the column are equal. Check the adequacy of the column using $f'_c = 4$ ksi and $f_y = 60\,\text{ksi}$.

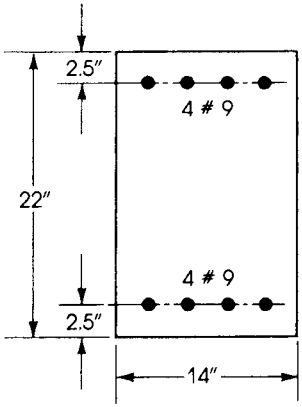


Figure 12.5 Example 12.1.

Solution

1. Calculate factored loads:

$$P_u = 1.2P_D + 1.6P_L = 1.2 \times 136 + 1.6 \times 110 = 339.2 \text{ K}$$

$$M_u = 1.2M_D + 1.6M_L = 1.2 \times 116 + 1.6 \times 93 = 288 \text{ K·ft}$$

$$e = \frac{M_u}{P_u} = \frac{288 \times 12}{339.2} = 10.2 \text{ in.}$$

2. Check if the column is long. Because the frame is braced against sidesway, assume $K = 1.0$, $r = 0.3h = 0.3 \times 22 = 6.6$ in., and $l_u = 19$ ft.

$$\frac{Kl_u}{r} = \frac{1 \times 19 \times 12}{6.6} = 34.5$$

For braced columns, if $Kl_u/r \leq 34 - 12M_1/M_2$, slenderness effect may be neglected. Given end moments $M_1 = M_2$ and M_1/M_2 positive for single curvature,

$$\text{Right-hand side} = 34 - 12\frac{M_1}{M_2} = 34 - 12 \times 1 = 22$$

Because $Kl_u/r = 34.5 > 22$, slenderness effect must be considered.

3. Calculate EI from Eq. 12.10:

   a. Calculate E_c:

$$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4000} = 3605 \text{ ksi}$$

$$E_s = 29{,}000 \text{ ksi}$$

   b. The moment of inertia is

$$I_g = \frac{14(22)^3}{12} = 12{,}422 \text{ in.}^4 \quad A_s = A'_s = 4.0 \text{ in.}^2$$

$$I_{se} = 2 \times 4.0\left(\frac{22 - 5}{2}\right)^2 = 578 \text{ in.}^4$$

   The dead-load moment ratio is

$$\beta_{dns} = \frac{1.2 \times 136}{339.2} = 0.48$$

c. The stiffness is

$$EI = \frac{0.2E_cI_g + E_sI_{se}}{1 + \beta_{dns}}$$
$$= \frac{(0.2 \times 3605 \times 12{,}422) + (29{,}000 \times 578)}{1 + 0.48}$$
$$= 17.40 \times 10^6 \text{ K·in.}^2$$

4. Calculate P_c:

$$P_c = \frac{\pi^2 EI}{(Kl_u)^2} = \frac{\pi^2(17.40 \times 10^6)}{(12 \times 19)^2} = 3303 \text{ K}$$

5. Calculate C_m from Eq. 12.14:

$$C_m = 0.6 + \frac{0.4M_1}{M_2} \geq 0.4$$
$$= 0.6 + 0.4(1) = 1.0$$

6. Calculate the moment-magnifier factor from Eq. 12.15:

$$\delta_{ns} = \frac{C_m}{1 - (P_u/0.75P_c)} = \frac{1}{1 - 339.2/(0.75 \times 3303)} = 1.16$$

7. Calculate the design moment and load: Assume ($\phi = 0.65$),

$$P_n = \frac{339.2}{0.65} = 522 \text{ K}$$
$$M_n = \frac{288}{0.65} = 443.1 \text{ K·ft}$$

Design $M_c = 443.1(1.16) = 514$ K·ft. Design eccentricity = 514/522 = 0.98 ft = 11.82 in., or 12 in.

8. Determine the nominal load strength of the section using $e = 12$ in. according to Example 11.4:

$$P_n = 47.6a + 226.4 - 4f_s \quad \text{(I)}$$

$$e' = e + d - \frac{h}{2} = 12 + 19.5 - \frac{22}{2} = 20.50 \text{ in.}$$

$$P_n = \frac{1}{20.50}\left[47.6a\left(19.5 - \frac{a}{2}\right) + 226.4(19.5 - 2.5)\right]$$
$$= 45a - 1.15a^2 + 186.6 \quad \text{(II)}$$

Solving for a from Eqs. I and II, $a = 10.6$ in. and $P_n = 535$ K. The load strength, P_n, is greater than the required load of 522 K; therefore, the section is adequate. If the section is not adequate, increase steel reinforcement.

9. Check the assumed ϕ:

$$a = 10.6 \text{ in.} \quad c = 12.47 \text{ in.} \quad d_t = 19.5 \text{ in.}$$
$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) 0.003$$
$$= 0.00169 < 0.002$$
$$\phi = 0.65$$

Example 12.2

Check the adequacy of the column in Example 12.1 if the unsupported length is $l_u = 10$ ft. Determine the maximum nominal load on the column.

Solution

1. Applied loads are $P_n = 522$ K and $M_n = 443.1$ K.
2. Check if the column is long: $l_u = 10$ ft, $r = 0.3h = 0.3 \times 22 = 6.6$ in., and $K = 1.0$ (frame is braced against sidesway).

$$\frac{Kl_u}{r} = \frac{1 \times (10 \times 12)}{6.6} = 18.2$$

Check if $Kl_u/r = 34 - 12M_{1b}/M_{2b}$:

$$\text{Right-hand side} = 34 - 12 \times 1 = 22$$

$$\frac{Kl_u}{r} = 18.2 < 22$$

Therefore, the slenderness effect can be neglected.

3. Determine the nominal load capacity of the short column, as explained in Example 11.4. From Example 11.4, the nominal compressive strength is $P_n = 612.1$ K (for $e = 10$ in.), which is greater than the required load of 522 K, because the column is short with $e = 10.2$ in. (Example 12.1).

Example 12.3

Check the adequacy of the column in Example 12.1 if the frame is unbraced (sway) against sidesway, the end-restraint factors are $\psi_A = 0.8$ and $\psi_B = 2.0$, and the unsupported length is $l_u = 16$ ft.

Solution

1. Determine the value of K from the alignment chart (Fig. 12.3) for unbraced frames. Connect the values of $\psi_A = 0.8$ and $\psi_B = 2.0$, to intersect the K-line at $K = 1.4$.

$$\frac{Kl_u}{r} = \frac{1.4 \times (16 \times 12)}{6.6} = 40.7$$

2. For unbraced frames, if $Kl_u/r \leq 22$, the column can be designed as a short column. Because actual $Kl_u r = 40.7 > 22$, the slenderness effect must be considered.
3. Calculate the moment magnifier δ_{ns}, given $C_m = 1.0$, $K = 1.4$, $EI = 17.40 \times 10^6$ K·in.^{2} (from Example 12.1), and

$$P_c = \frac{\pi^2 EI}{(Kl_u)^2} = \frac{\pi^2 \times 17.40 \times 10^6}{(1.4 \times 16 \times 12)^2} = 2377 \text{ K}$$

$$\delta_{ns} = \frac{C_m}{1 - \left(\dfrac{P_u}{0.75 \times P_c}\right)} = \frac{1.0}{1 - \left(\dfrac{339.2}{0.75 \times 2377}\right)} = 1.24$$

4. From Example 12.1, the applied loads are $P_u = 339.2$ K and $M_u = 288$ K·ft, or

$$P_n = 522 \text{ K} \quad \text{and} \quad M_n = 443.1 \text{ K·ft}$$

The design moment $M_c = 1.24(443.1) = 549.4$ K·ft; hence,

$$e = \frac{\delta_{ns} M_n}{P_n} = 549.4 \times \frac{12}{522} = 12.63 \text{ in.} \qquad \text{say, } 13 \text{ in.}$$

5. The requirement now is to check the adequacy of a short column for $P_n = 522\,\text{K}$, $M_c = 549.4\,\text{K·ft}$, and $e = 13\,\text{in}$. The procedure is explained in Example 11.4.
6. From Example 11.4,

$$P_n = 47.6a + 226.4 - 4f_s$$

$$e' = e + d - \frac{h}{2} = 13 + 19.5 - \frac{22}{2} = 21.5 \text{ in.}$$

$$P_n = \frac{1}{21.5}\left[47.6a\left(19.5 - \frac{a}{2}\right) + 226.4(19.5 - 2.5)\right]$$

$$= 43.16a - 1.1a^2 + 179 \quad a = 10.4 \text{ in.}$$

Thus, $c = 12.24\,\text{in.}$ and $P_n = 508\,\text{K}$. This load capacity of the column is less than the required P_n of 522 K. Therefore, the section is not adequate.

7. Increase steel reinforcement to four no. 10 bars on each side and repeat the calculations to get $P_n = 568\,\text{K}$, $\varepsilon_t < 0.002$, and $\phi = 0.65$.

Example 12.4

Design an interior square column for the first story of an eight-story office building. The clear height of the first floor is 16 ft, and the height of all other floors is 11 ft. The building layout is in 24 bays (Fig. 12.6), and the columns are not braced against sidesway. The loads acting on a first-floor interior column due to gravity and wind are as follows:

Axial dead load = 380 K

Axial live load = 140 K

Axial wind load = 0 K

Dead-load moments = 32 K·ft (top) and 54 K·ft (bottom)

Live-load moments = 20 K·ft (top) and 36 K·ft (bottom)

Wind-load moments = 50 K·ft (top) and 50 K·ft (bottom)

EI/l for beams = 360×10^3 K·in.

Use $f'_c = 5$ ksi, $f_y = 60$ ksi, and the ACI Code requirements. Assume an exterior column load of two-thirds the interior column load, a corner column load of one-third the interior column load, and $\beta_{\text{dns}} = 0.55$.

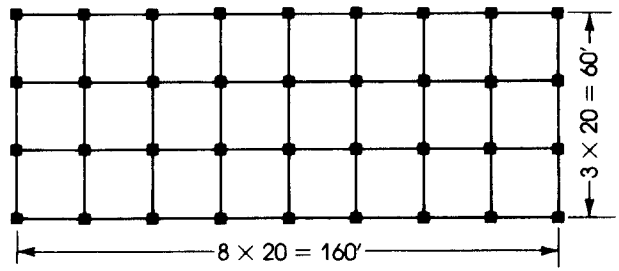


Figure 12.6 Example 12.4.

Solution

1. Calculate the factored forces using load combinations. For gravity loads,

$$P_u = 1.2D + 1.6L = 1.2(380) + 1.6(140) = 680 \text{ K}$$
$$M_u = M_{2\text{ns}} = 1.2M_D + 1.6M_L = 1.2(54) + 1.6(36) = 122.4 \text{ K·ft}$$

For gravity plus wind load,

$$P_u = (1.2D + 0.5L + 1.6W)$$
$$= [1.2(380) + 0.5(140 + 0)] = 526 \text{ K}$$
$$M_{\text{uns}} = M_{2\text{ns}} = (1.2 \times 54 + 1.6 \times 36) = 122.4 \text{ K·ft}$$
$$M_{\text{us}} = M_{2s} = (1.6M_w) = (1.6 \times 50) = 80 \text{ K·ft}$$

Other combinations are not critical:

$$P_u = 0.9D + 1.6W = 0.9(380) + 1.6(0) = 342 \text{ K}$$
$$M_2 = M_{\text{uns}} = 0.9M_D = 0.9(54) = 48.6 \text{ K·ft}$$
$$M_{2s} = 1.6M_w = 1.6(50) = 80 \text{ K·ft}$$
$$e = \frac{M_u}{P_u} = \frac{M_{2\text{ns}}}{P_u} = 122.4 \times \frac{12}{680} = 2.16 \text{ in.}$$
$$\min e = 0.6 + 0.03(18) = 1.14 \text{ in.} < 2.16$$

2. Select a preliminary section of column based on gravity load combination using tables or charts. Select a section 18 by 18 in. reinforced by four no. 10 bars (Fig. 12.7).
3. Check Kl_u/r:

$$I_g = \frac{(18)^4}{12} = 8748 \text{ in.}^4 \quad E_c = 4.03 \times 10^6 \text{ psi}$$

for columns, $I = 0.7\ I_g$.
For a 16-ft column,

$$\frac{EI}{l_c} = \frac{(0.7)(8748)(4.03 \times 10^6)}{16 \times 12} = 128.5 \times 10^6$$

For an 11-ft column,

$$\frac{EI}{l_c} = \frac{(0.7)(8748)(4.03 \times 10^6)}{11 \times 12} = 187 \times 10^6$$

For beams, and $EI_g/l_b = 360 \times 10^6$, $I = 0.35\ I_g$, and $EI/l_b = 0.35EI_g/l_b = 126 \times 10^6$

$$\psi(\text{top}) = \psi(\text{bottom}) = \frac{\Sigma(EI/l_c)}{\Sigma(EI/l_b)} = \frac{(128.5 + 187)}{2(126)} = 1.25$$

From the chart (Fig. 12.3), K is 1.37 for an unbraced frame and 0.8 for a braced frame.

$$\frac{Kl_u}{r} = \frac{1.37(16 \times 12)}{0.3 \times 18} = 48.7$$

which is more than 22. Therefore, the slenderness ratio must be considered.

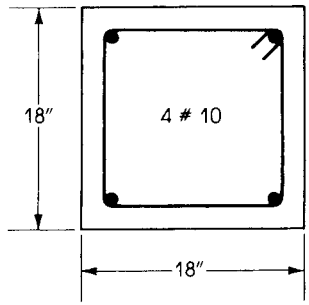


Figure 12.7 Column cross section, Example 12.4.

4. Compute P_c:

$$E_c = 4.03 \times 10^3 \text{ ksi} \quad E_s = 29 \times 10^3 \text{ ksi}$$

$$I_g = 8748 \text{ in.}^4 \qquad I_{se} = 5.06\left(\frac{13}{2}\right)^2 = 214 \text{ in.}^4$$

$$\beta_{dns} = 0.55$$

$$EI = \frac{0.2E_cI_g + E_sI_{se}}{1 + \beta_{dns}}$$

$$EI = \frac{0.2(4.03 \times 10^3 \times 8748) + 29 \times 10^3(214)}{1 + 0.55} = 8.55 \times 10^6 \text{ K·in.}^2$$

For calculation of δ_s, $\beta_{dns} = 0$ and $E = 8.55 \times 10^6(1.55) = 13.25 \times 10^6$ K·in.^{2}

$$P_c = \frac{\pi^2 EI}{(Kl_u)^2} = \frac{\pi^2(8.55 \times 10^6)}{(0.8 \times 16 \times 12)^2} = 3577 \text{ K} \quad \text{(braced)}$$

$$P_c = \frac{\pi^2(13.25 \times 10^6)}{(1.37 \times 16 \times 12)^2} = 1890 \text{ K} \quad \text{(unbraced)}$$

For one floor in the building, there are 14 interior columns, 18 exterior columns, and four corner columns.

$$\Sigma P_u = 14(526) + 18\left(\frac{2}{3} \times 526\right) + 4\left(\frac{1}{3} \times 526\right) = 14{,}377 \text{ K}$$

$$\Sigma P_c = 14(1890) + 22\left(\frac{2}{3} \times 1890\right) = 54{,}180 \text{ K}$$

$$\delta_s = \frac{1.0}{1 - \left(\dfrac{14{,}377}{0.75 \times 54{,}180}\right)} = 1.54$$

which is greater than 1.0 (Eq. 12.17).

$$M_c = M_{2ns} + \delta_s M_{2s} = (122.4) + 1.54(80) = 245.6 \text{ K·ft}$$

5. Design loads are $P_u = 526$ K and $M_c = 245.6$ K·ft.

$$e = \frac{245.6(12)}{526} = 5.6 \text{ in.}$$

$$e_{\min} = 0.6 + 0.03(18) = 1.14 \text{ in.} < e$$

By analysis, for $e = 5.6$ in. and $A_s = A'_s = 2.53$ in.^{2}, ($\phi = 0.65$ in.) the load capacity of the 18 × 18-in. column is $\phi P_n = 556$ K and $\phi M_n = 259$ K·ft, so the section is adequate. (Solution steps are similar to Example 11.4. Values are $a = 10.37$ in., $c = 13$ in., $f_s = 17$ ksi, $f'_s = 60$ ksi, $\phi P_b = 385$ K, and $e_b = 8.9$ in.).

$$\varepsilon_t = 0.003\frac{(15.5 - 13)}{13} = 0.00058 < 0.002, \quad \phi = 0.65.$$

SUMMARY

Sections 12.1–12.3

1. The radius of gyration is $r = \sqrt{I/A}$, where $r = 0.3h$ for rectangular sections and $0.25D$ for circular sections.
2. The effective column length is Kl_u. For braced frames, $K = 1.0$; for unbraced frames, K varies as shown in Fig. 12.2.
3. K can be determined from the alignment chart (Fig. 12.3) or Eqs. 12.2 through 12.6.

Section 12.4

Member stiffness is EI:

$$E_c = 33w^{1.5}\sqrt{f'_c}$$

The moment of inertia, I, may be taken as $I = 0.35I_g$ for beams, $0.70I_g$ for columns, $0.70I_g$ for uncracked walls, $0.35I_g$ for cracked walls, and $0.25I_g$ for plates and flat slabs.

Alternatively, the moments of inertia of compression and flexural members, I, shall be permitted to be computed as follows:

1. Compression members:

$$I = \left(0.80 + 25\frac{A_{st}}{A_g}\right)\left(1 - \frac{M_u}{P_u h} - 0.5\frac{P_u}{P_o}\right) I_g \le 0.875I_g \tag{12.2}$$

2. Flexural members:

$$I = (0.10 + 25\rho)\left(1.2 - 0.2\frac{b_w}{d}\right) I_g \le 0.5I_g \tag{12.3}$$

Section 12.5

The effect of slenderness may be neglected when

$$\frac{Kl_u}{r} \le 22 \quad \text{(for unbraced frames)} \tag{12.8}$$

$$\frac{Kl_u}{r} \le 34 - \frac{12M_1}{M_2} \quad \text{(for braced columns)} \tag{12.5}$$

where M_1 and M_2 are the end moments and $M_2 > M_1$.

Section 12.6

1. For nonsway frames,

$$EI = \frac{0.2E_cI_g + E_sI_{se}}{1 + \beta_{dns}} \tag{12.10}$$

or the more simplified equation

$$EI = \frac{0.4E_cI_g}{1 + \beta_{dns}} \tag{12.11}$$

$$\beta_{dns} = \frac{1.2D}{1.2D + 1.6L} \tag{12.4}$$

More simply,

$$EI = 0.25E_cI_g \ (\beta_{dns} = 0.6) \tag{12.12}$$

The Euler buckling load is

$$P_c = \frac{\pi^2 EI}{(Kl_u)^2} \tag{12.13}$$

$$C_m = 0.6 + \frac{0.4M_1}{M_2} \geq 0.4 \tag{12.14}$$

The moment-magnifier factor (nonsway frames) is

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75P_c}} \tag{12.15}$$

The design moment is

$$M_c = \delta_{ns} M_2 \tag{12.16}$$

2. For sway (unbraced) frames, the moment-magnifier factor is calculated either from

a. Magnifier method

$$\delta_s = \frac{1.0}{1 - \frac{\Sigma P_u}{0.75\Sigma P_c}} \geq 1.0 \tag{12.17}$$

b. Approximate second order analysis

$$\delta_s = \frac{1}{1 - Q} \tag{12.19}$$

$$Q = \frac{\Sigma P_u \Delta_0}{V_{us} l_c} \tag{12.20}$$

the design moment is

$$M_1 = M_{1ns} + \delta_s M_{1s} \tag{12.21}$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \tag{12.22}$$

If $M_2 > M_1$ then:

$$M_c = M_{2ns} + \delta_s M_{2s} \tag{12.23}$$

where M_{2ns} is the unmagnified moment due to gravity loads (nonsway moment) and $\delta_s M_{2s}$ is the magnified moment due to sway frame loads.

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PROBLEMS

12.1 The column section in Fig. 12.8 carries an axial load $P_D = 128\,\text{K}$ and a moment $M_D = 117\,\text{K·ft}$ due to dead load and an axial load $P_L = 95\,\text{K}$ and a moment $M_1 = 100\,\text{K·ft}$ due to live load. The column is part of a frame, braced against sidesway, and bent in single curvature about its major axis. The unsupported length of the column is $l_u = 18\,\text{ft}$, and the moments at both ends are equal. Check the adequacy of the section using $f'_c = 4$ ksi and $f_y = 60\,\text{ksi}$.

12.2 Repeat Problem 12.1 if $l_u = 12\,\text{ft}$.

12.3 Repeat Problem 12.1 if the frame is unbraced against sidesway and the end-restraint factors are ψ (top) = 0.7 and ψ (bottom) = 1.8 and the unsupported height is $l_u = 14\,\text{ft}$.

12.4 The column section shown in Fig. 12.9 is part of a frame unbraced against sidesway and supports an axial load $P_D = 166\,\text{K}$ and a moment $M_D = 107\,\text{K·ft}$ due to dead load and $P_L = 115\,\text{K}$ and $M_L = 80\,\text{K·ft}$ due to live load. The column is bent in single curvature and has an unsupported length $l_u = 16\,\text{ft}$. The moment at the top of the column is $M_2 = 1.5M_1$, the moment at the bottom of the column. Check if the section is adequate using $f'_c = 5$ ksi, $f_y = 60\,\text{ksi}$, ψ (top) = 2.0, and ψ (bottom) = 1.0.

12.5 Repeat Problem 12.4 if the column length is $l_u = 14\,\text{ft}$.

12.6 Repeat Problem 12.4 if the frame is braced against sidesway and $M_1 = M_2$.

12.7 Repeat Problem 12.4 using $f'_c = 4$ ksi and $f_y = 60\,\text{ksi}$.

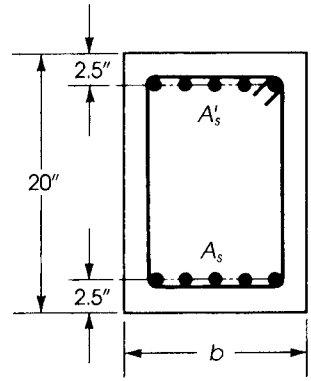


Figure 12.8 Problem 12.1 ($A_s = A'_s$) = 5 no. 9 bars and b = 14 in.

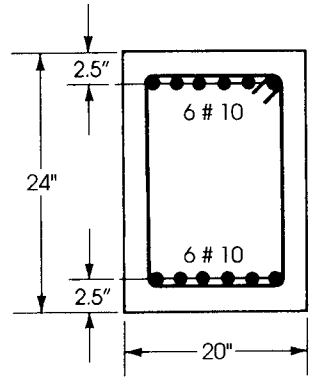


Figure 12.9 Problem 12.4.

12.8 Design a 20-ft-long rectangular tied column for an axial load $P_D = 214.5$ K and a moment $M_D = 64$ K·ft due to dead load and an axial load $P_L = 120$ K and a moment $M_L = 40$ K·ft due to live load. The column is bent in single curvature about its major axis, braced against sidesway, and the end moments are equal. The end-restraint factors are ψ (top) = 2.5 and ψ (bottom) = 1.4. Use $f'_c = 5$ ksi, $f_y = 60$ ksi, and $b = 15$ in.

12.9 Design the column in Problem 12.8 if the column length is 10 ft.

12.10 Repeat Problem 12.8 if the column is unbraced against sidesway.

CHAPTER 13

FOOTINGS



Office building under construction, New Orleans, Louisiana.

13.1 INTRODUCTION

Reinforced concrete footings are structural members used to support columns and walls and to transmit and distribute their loads to the soil. The design is based on the assumption that the footing is rigid, so that the variation of the soil pressure under the footing is linear. Uniform soil pressure is achieved when the column load coincides with the centroid of the footing. Although this assumption is acceptable for rigid footings, such an assumption becomes less accurate as the footing becomes relatively more flexible. The proper design of footings requires that

1. The load capacity of the soil is not exceeded.
2. Excessive settlement, differential settlement, or rotations are avoided.
3. Adequate safety against sliding and/or overturning is maintained.

The most common types of footings used in buildings are the single footings and wall footings (Figs. 13.1 and 13.2). When a column load is transmitted to the soil by the footing, the soil becomes compressed. The amount of settlement depends on many factors, such as the type of soil, the load intensity, the depth below ground level, and the type of footing. If different footings of the same structure have different settlements, new stresses develop in the structure. Excessive differential settlement may lead to the damage of nonstructural members in the buildings or even failure of the affected parts.

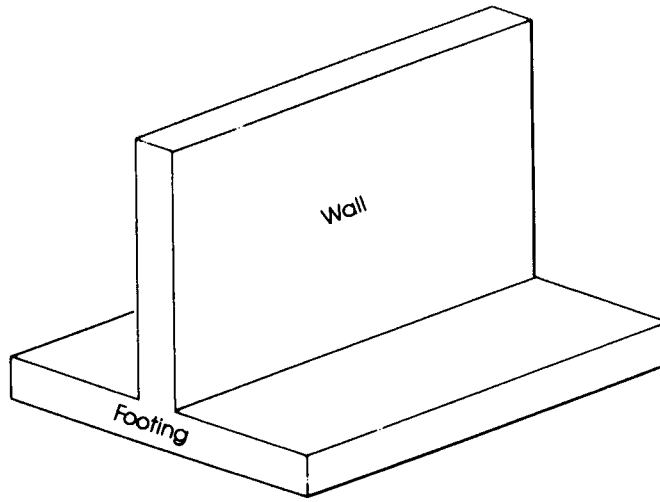


Figure 13.1 Wall footing.

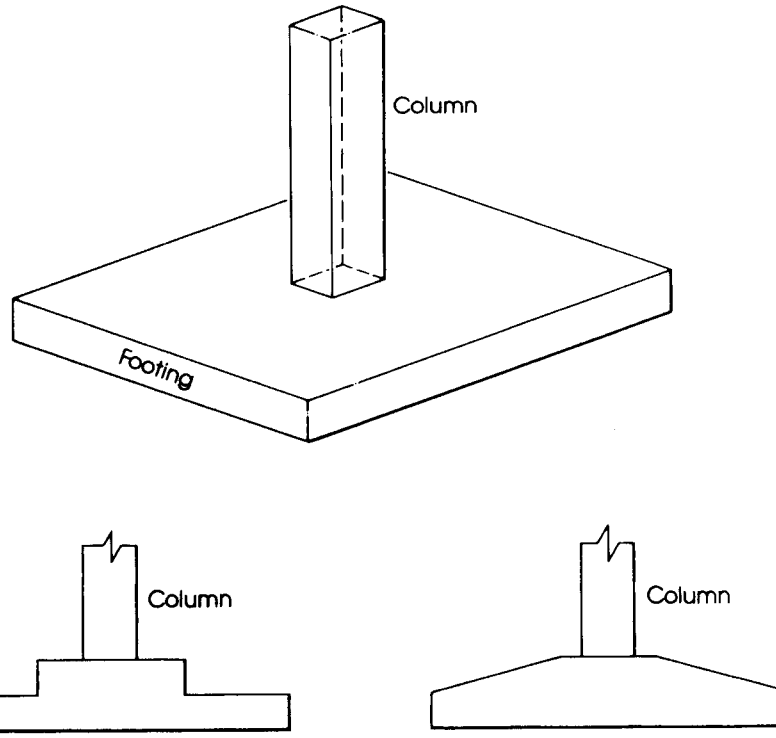


Figure 13.2 Single footing.

Vertical loads are usually applied at the centroid of the footing. If the resultant of the applied loads does not coincide with the centroid of the bearing area, a bending moment develops. In this case, the pressure on one side of the footing will be greater than the pressure on the other side.

If the bearing soil capacity is different under different footings—for example, if the footings of a building are partly on soil and partly on rock—a differential settlement will occur. It is usual in such cases to provide a joint between the two parts to separate them, allowing for independent settlement.

The depth of the footing below the ground level is an important factor in the design of footings. This depth should be determined from soil tests, which should provide reliable information on safe bearing capacity at different layers below ground level. Soil test reports specify the allowable bearing capacity to be used in the design. In cold areas where freezing occurs, frost action may cause heaving or subsidence. It is necessary to place footings below freezing depth to avoid movements.

13.2 TYPES OF FOOTINGS

Different types of footings may be used to support building columns or walls. The most common types are as follows:

1. *Wall footings* are used to support structural walls that carry loads from other floors or to support nonstructural walls. They have a limited width and a continuous length under the wall (Fig. 13.1). Wall footings may have one thickness, be stepped, or have a sloped top.
2. *Isolated*, or *single, footings* are used to support single columns (Fig. 13.2). They may be square, rectangular, or circular. Again, the footing may be of uniform thickness, stepped, or have a sloped top. This is one of the most economical types of footings, and it is used when columns are spaced at relatively long distances. The most commonly used are square or rectangular footings with uniform thickness.
3. *Combined footings* (Fig. 13.3) usually support two columns or three columns not in a row. The shape of the footing in plan may be rectangular or trapezoidal, depending on column loads. Combined footings are used when two columns are so close that single footings cannot be used or when one column is located at or near a property line.
4. *Cantilever*, or *strap, footings* (Fig. 13.4) consist of two single footings connected with a beam or a strap and support two single columns. They are used when one footing supports an eccentric column and the nearest adjacent footing lies at quite a distance from it. This type replaces a combined footing and is sometimes more economical.
5. *Continuous footings* (Fig. 13.5) support a row of three or more columns. They have limited width and continue under all columns.
6. *Raft*, or *mat, foundations* (Fig. 13.6) consist of one footing, usually placed under the entire building area, and support the columns of the building. They are used when
   a. The soil-bearing capacity is low.
   b. Column loads are heavy.
   c. Single footings cannot be used.
   d. Piles are not used.
   e. Differential settlement must be reduced through the entire footing system.
7. *Pile caps* (Fig. 13.7) are thick slabs used to tie a group of piles together and to support and transmit column loads to the piles.

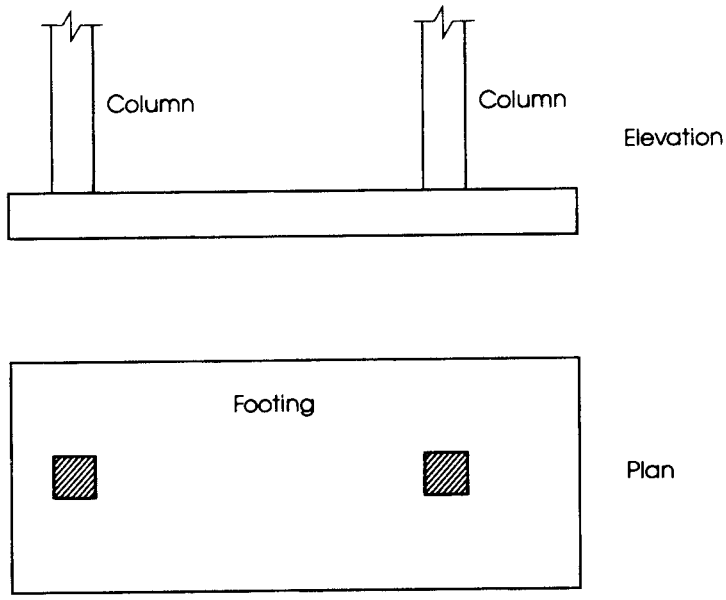


Figure 13.3 Combined footing.

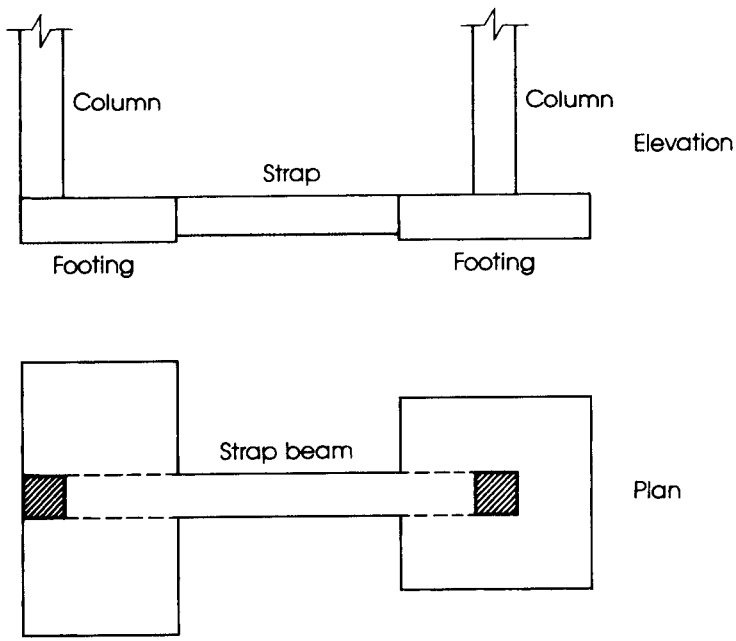


Figure 13.4 Strap footing.

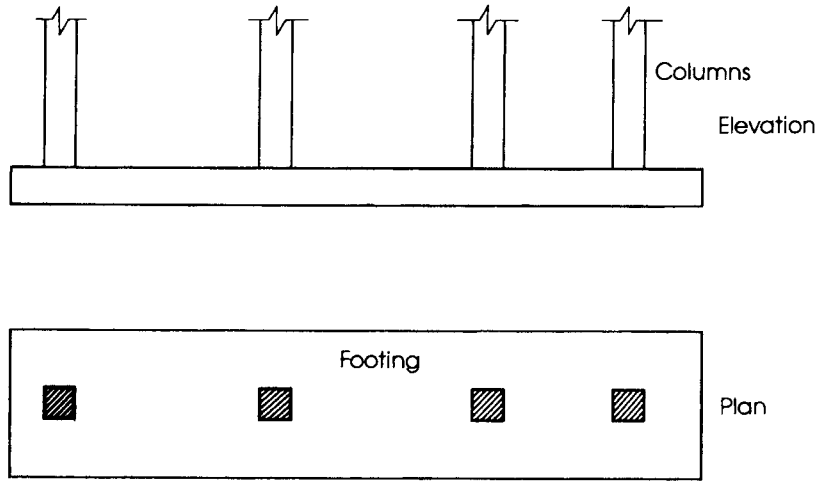


Figure 13.5 Continuous footing.

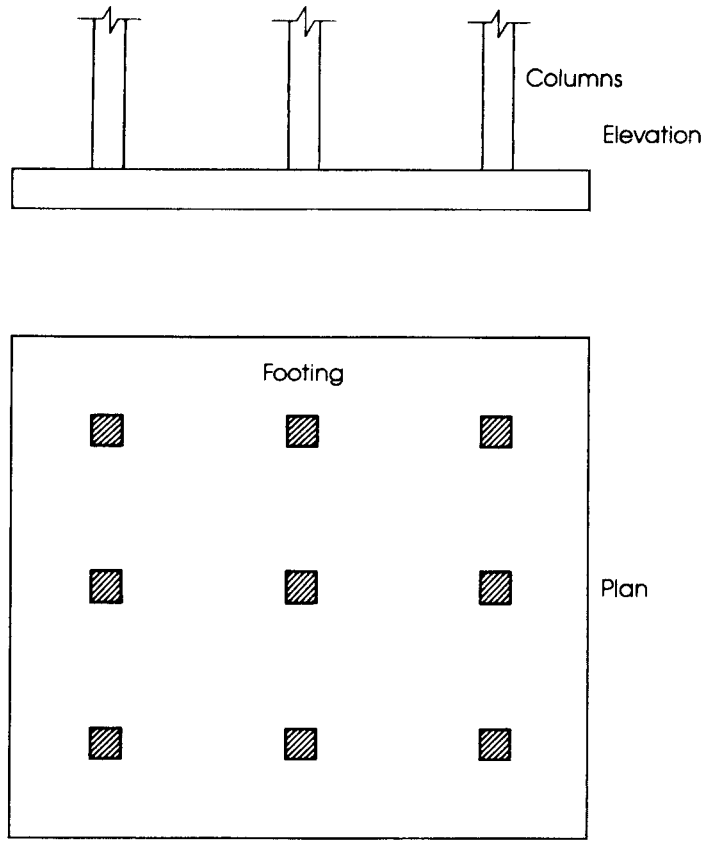


Figure 13.6 Raft, or mat, foundation.

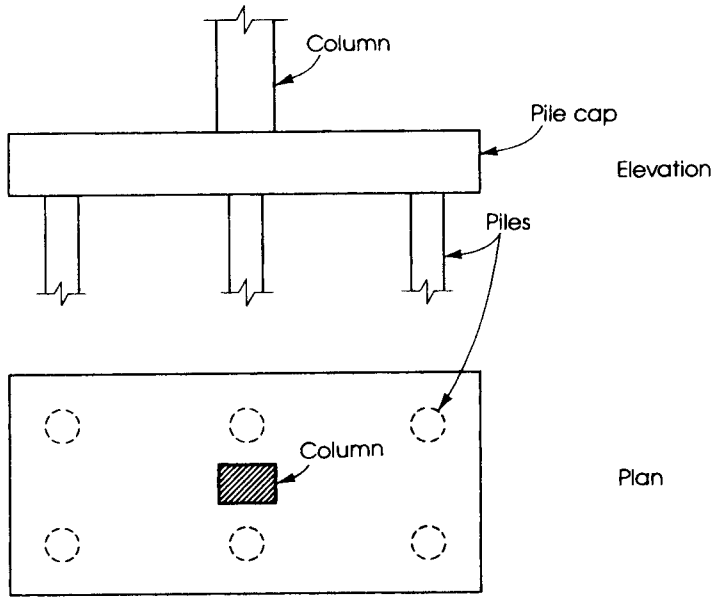


Figure 13.7 Pile cap footing.

13.3 DISTRIBUTION OF SOIL PRESSURE

Fig. 13.8 shows a footing supporting a single column. When the column load, P, is applied on the centroid of the footing, a uniform pressure is assumed to develop on the soil surface below the footing area. However, the actual distribution of soil pressure is not uniform but depends on many factors, especially the composition of the soil and the degree of flexibility of the footing.

For example, the distribution of pressure on cohesionless soil (sand) under a rigid footing is shown in Fig.13.9. The pressure is maximum under the center of the footing and decreases toward the ends of the footing. The cohesionless soil tends to move from the edges of the footing, causing a reduction in pressure, whereas the pressure increases around the center to satisfy equilibrium conditions. If the footing is resting on a cohesive soil such as clay, the pressure under the edges is greater than at the center of the footing (Fig. 13.10). The clay near the edges has a strong cohesion with the adjacent clay surrounding the footing, causing the nonuniform pressure distribution.

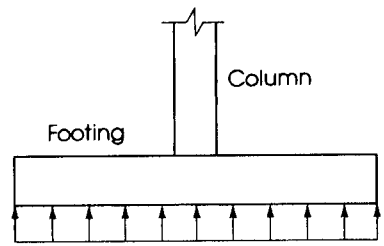


Figure 13.8 Distribution of soil pressure assuming uniform pressure.

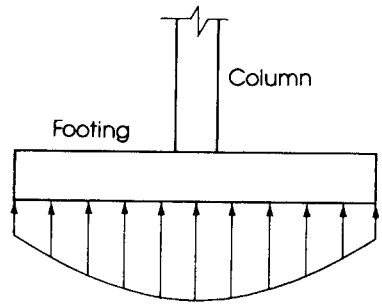


Figure 13.9 Soil pressure distribution in cohesionless soil (sand).

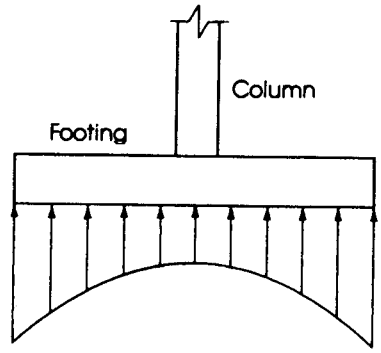


Figure 13.10 Soil pressure distribution in cohesive soil (clay).

The allowable bearing soil pressure, q_a, is usually determined from soil tests. The allowable values vary with the type of soil, from extremely high in rocky beds to low in silty soils. For example, q_a, for sedimentary rock is 30 ksf, for compacted gravel is 8 ksf, for well-graded compacted sand is 6 ksf, and for silty-gravel soils is 3 ksf.

Referring to Fig. 13.8, when the load P is applied, the part of the footing below the column tends to settle downward. The footing will tend to take a uniform curved shape, causing an upward pressure on the projected parts of the footing. Each part acts as a cantilever and must be designed for both bending moments and shearing forces. The design of footings is explained in detail later.

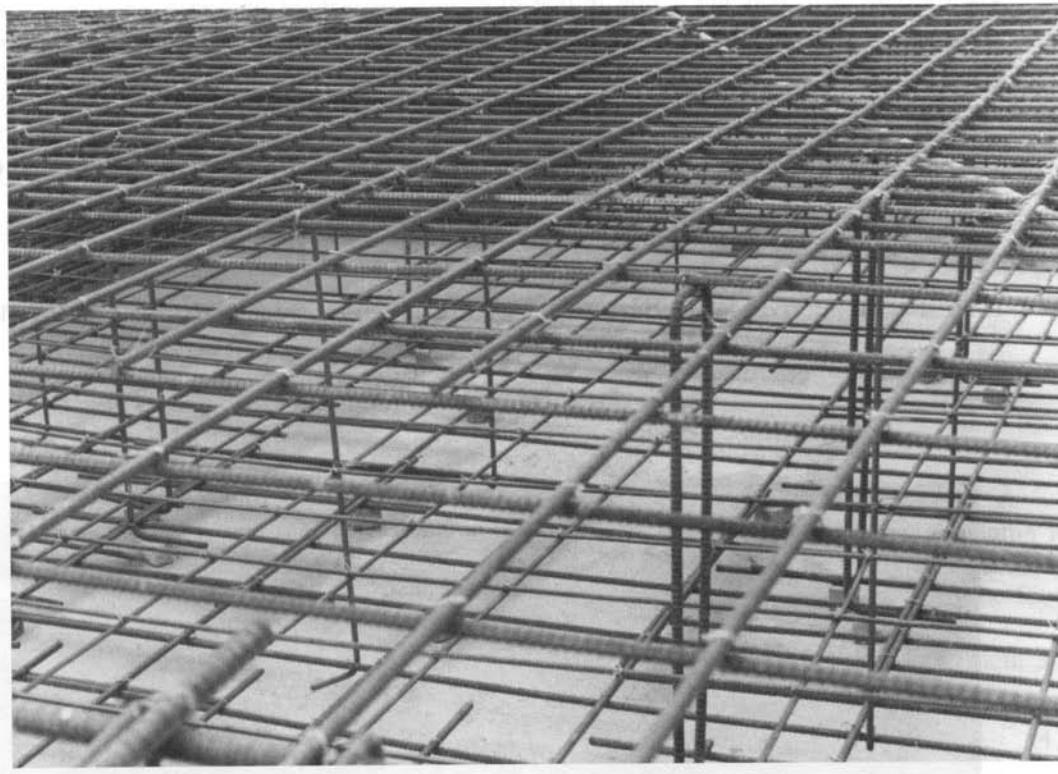
13.4 DESIGN CONSIDERATIONS

Footings must be designed to carry the column loads and transmit them to the soil safely. The design procedure must take the following strength requirements into consideration:

1. The area of the footing based on the allowable bearing soil capacity
2. On-way shear
3. Two-way shear, or punching shear
4. Bending moment and steel reinforcement required

5. Bearing capacity of columns at their base and dowel requirements
6. Development length of bars
7. Differential settlement

These strength requirements are explained in the following sections.



Reinforcing rebars placed in two layers in a raft foundation.

13.4.1 Size of Footings

The area of the footings can be determined from the actual external loads (unfactored forces and moments) such that the allowable soil pressure is not exceeded. In general, for vertical loads

$$\text{Area of footing} = \frac{\text{total service load (including self-weight)}}{\text{allowable soil pressure, } q_a} \tag{13.1}$$

or

$$\text{Area} = \frac{P(\text{total})}{q_a}$$

where the total service load is the unfactored design load. Once the area is determined, a factored soil pressure is obtained by dividing the factored load, $P_u = 1.2D + 1.6L$, by the area of the

footing. This is required to design the footing by the strength design method.

$$q_u = \frac{P_u}{\text{area of footing}} \tag{13.2}$$

The allowable soil pressure, q_a, is obtained from soil test and is based on service load conditions.

13.4.2 One-Way Shear (Beam Shear) (V_{u_1})

For footings with bending action in one direction, the critical section is located at a distance d from the face of the column. The diagonal tension at section m-m in Fig. 13.11 can be checked as was done before in beams. The allowable shear in this case is equal to

$$\phi V_c = 2\phi\lambda\sqrt{f'_c}bd \quad (\phi = 0.75) \tag{13.3}$$

where b = width of section m-m. The factored shearing force at section m-m can be calculated as follows:

$$V_{u_1} = q_u b\left(\frac{L}{2} - \frac{C}{2} - d\right) \tag{13.4}$$

If no shear reinforcement is to be used, then d can be determined, assuming $V_u = \phi V_c$:

$$d = \frac{V_{u_1}}{2\phi\lambda\sqrt{f'_c}b} \tag{13.5}$$



Wall and column footings, partly covered.

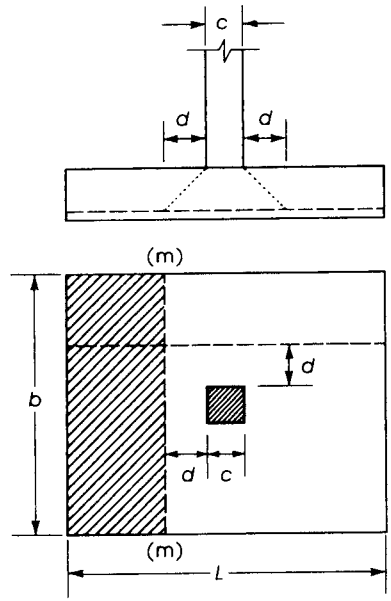


Figure 13.11 One-way shear.

13.4.3 Two-Way Shear (Punching Shear) (V_{u_2})

Two-way shear is a measure of the diagonal tension caused by the effect of the column load on the footing. Inclined cracks may occur in the footing at a distance $d/2$ from the face of the column on all sides. The footing will fail as the column tries to punch out part of the footing (Fig. 13.12).

The ACI Code, Section 11.11.2 allows a shear strength, V_c, in footings without shear reinforcement for two-way shear action, the smallest of

$$V_{c_1} = 4\lambda\sqrt{f'_c}b_0 d \tag{13.6}$$

$$V_{c_2} = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c}b_0 d \tag{13.7}$$

$$V_{c_3} = \left(\frac{\alpha_s d}{b_0} + 2\right)\lambda\sqrt{f'_c}b_0 d \tag{13.8}$$

where

β = Ratio of long side to short side of the rectangular column

b_0 = perimeter of the critical section taken at $d/2$ from the loaded area (column section) (see Fig. 13.12)

d = effective depth of footing

λ = is a modification factor for type of concrete (ACI 8.6.1)

λ = 1.0 Normal-weight concrete

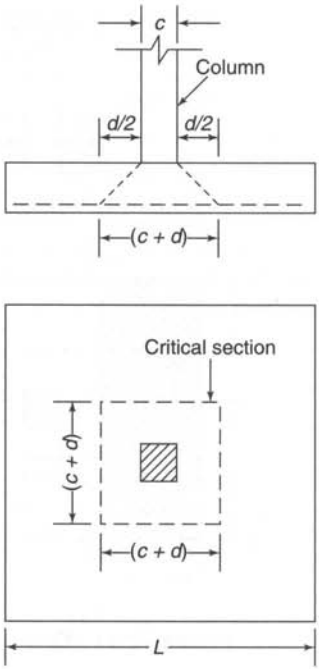
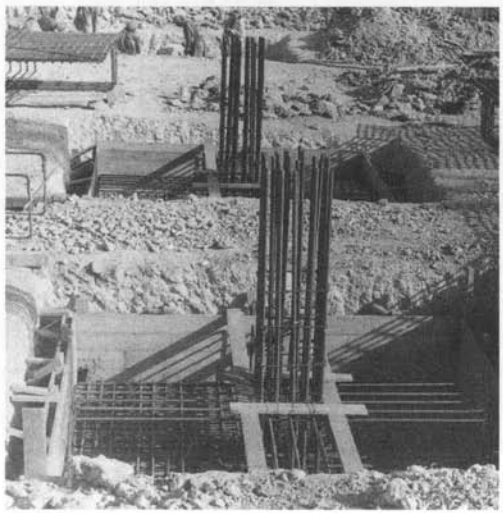


Figure 13.12 Punching shear (two-way).

$\lambda = 0.85$ sand-lightweight concrete
$\lambda = 0.75$ for all-lightweight concrete

Linear interpolation shall be permitted between 0.85 and 1.0 on the basis of volumetric fractions, for concrete containing normal-weight fine aggregate and a blend of lightweight and normal-weight coarse aggregate.



Reinforced concrete single footings.

For the values of V_{c_1} and V_{c_2} it can be observed that V_{c_1} controls (less than V_{c_2}) whenever $\beta_c \leq 2$, whereas V_{c_2} controls (less than V_{c_1}) whenever $\beta_c > 2$. This indicates that the allowable shear V_c is reduced for relatively long footings. The actual soil pressure variation along the long side increases with an increase in β. For shapes other than rectangular, β is taken to be the ratio of the longest dimension of the effective loaded area in the long direction to the largest width in the short direction (perpendicular to the long direction).

For Eq. 13.8, α_s is assumed to be 40 for interior columns, 30 for edge columns, and 20 for corner columns. The concrete shear strength, V_{c_3} represents the effect of an increase in b_0 relative to d. For a high ratio of b_0/d, V_{c_3} may control.

Based on the preceding three values of V_c, the effective depth, d, required for two-way shear is the largest obtained from the following formulas ($\phi = 0.75$):

$$d_1 = \frac{V_{u_2}}{\phi 4\lambda\sqrt{f'_c}b_0} \quad \text{(where } \beta \leq 2\text{)} \tag{13.9}$$

or

$$d_1 = \frac{V_{u_2}}{\phi\left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c}b_0} \quad \text{(where } \beta > 2\text{)} \tag{13.10}$$

$$d_2 = \frac{V_{u_2}}{\phi\left(\frac{\alpha_s d}{b_0} + 2\right)\lambda\sqrt{f'_c}b_0} \tag{13.11}$$

The two-way shearing force, V_{u_2}, and the effective depth, d, required (if shear reinforcement is not provided) can be calculated as follows (refer to Fig. 13.12):

1. Assume d.
2. Determine b_0: $b_0 = 4(c + d)$ for square columns, where one side $= c$. $b_0 = 2\,(c_1 + d) + 2(c_2 + \text{d})$ for rectangular columns of sides c_1 and c_2.
3. The shearing force V_{u_2} acts at a section that has a length $b_0 = 4\,(c + d)$ or $[2(c_1 + d) + 2(c_2 + d)]$ and a depth d; the section is subjected to a vertical downward load, P_u, and a vertical upward pressure, q_u (Eq. 13.2). Therefore,

$$V_{u_2} = P_u - q_u(c + d)^2 \text{ for square columns} \tag{13.12a}$$

$$V_{u_2} = P_u - q_u(c_1 + d)(c_2 + d) \text{ for rectangular columns} \tag{13.12b}$$

4. Determine the largest d (of d_1 and d_2). If d is not close to the assumed d, revise your assumption and repeat.

13.4.4 Flexural Strength and Footing Reinforcement

The critical sections for moment occur at the face of the column (section n-n, Fig. 13.13). The bending moment in each direction of the footing must be checked and the appropriate reinforcement must be provided. In square footings and square columns, the bending moments in both directions are equal. To determine the reinforcement required, the depth of the footing in each direction may be used. Because the bars in one direction rest on top of the bars in the other direction, the effective depth, d, varies with the diameter of the bars used. An average value of d may be adopted. A practical value of d may be assumed to be $(h - 4.5)$ in.

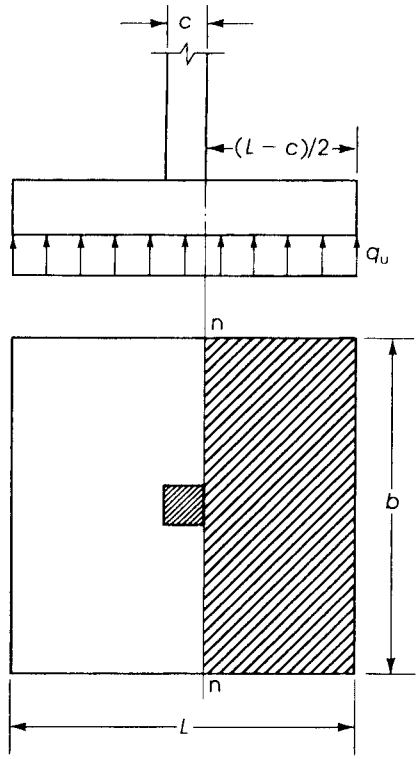


Figure 13.13 Critical section of bending moment.

The depth of the footing is often controlled by shear, which requires a depth greater than that required by the bending moment. The steel reinforcement in each direction can be calculated in the case of flexural members as follows:

$$M_u = \phi A_s f_y \left(d - \frac{A_s f_y}{1.7 f_c' b} \right) \tag{13.13}$$

Also, the steel ratio, ρ, can be determined as follows (Eq. 4.2):

$$\rho = \frac{0.85 f_c'}{f_y} \left[1 - \sqrt{1 - \frac{2R_u}{\phi(0.85 f_c')}} \right] \tag{13.14}$$

where $R_u = M_u / bd^2$. When R_u is determined, ρ can also be obtained from Eq. 13.15.

The minimum steel ratio requirement in flexural members is equal to $200/f_y$ when $f_c' < 4500$ psi and equal to $3\sqrt{f_c'}/f_y$ when $f_c' \geq 4500$ psi. However, the ACI Code, Section 10.5, indicates that for structural slabs of uniform thickness, the minimum area and maximum spacing of steel bars in the direction of bending shall be as required for shrinkage and temperature reinforcement. This last minimum steel requirement is very small, and a higher minimum reinforcement ratio is recommended, but it should not be greater than $200/f_y$.

The reinforcement in one-way footings and two-way footings must be distributed across the entire width of the footing. In the case of two-way rectangular footings, the ACI Code, Section 15.4.4, specifies that in the long direction, a portion of the total reinforcement $\gamma_s A_s$

distributed uniformly along the width of the footing. In the short direction, a certain ratio of the total reinforcement in this direction must be placed uniformly within a bandwidth equal to the length of the short side of the footing according to

$$\gamma_s = \frac{2}{\beta + 1} \tag{13.15}$$

where

$$\beta = \frac{\text{long side of footing}}{\text{short side of footing}} \tag{13.16}$$

The bandwidth must be centered on the centerline of the column (Fig. 13.14). The remaining reinforcement in the short direction must be uniformly distributed outside the bandwidth. This remaining reinforcement percentage shall not be less than that required for shrinkage and temperature.

When structural steel columns or masonry walls are used, then the critical sections for moments in footings are taken at halfway between the middle and the edge of masonry walls and halfway between the face of the column and the edge of the steel base place (ACI Code, Section 15.4.2).

13.4.5 Bearing Capacity of Column at Base

The loads from the column act on the footing at the base of the column, on an area equal to the area of the column cross-section. Compressive forces are transferred to the footing directly by bearing on the concrete.

Forces acting on the concrete at the base of the column must not exceed the bearing strength of concrete as specified by the ACI Code, Section 10.14:

$$\text{Bearing strength } N_1 = \phi(0.85 f'_c A_1) \tag{13.17}$$

where $\phi = 0.65$ and $A_1 =$ the bearing area of the column. The value of the bearing strength given in Eq. 13.17 may be multiplied by a factor $\sqrt{A_2/A_1} \leq 2.0$ for bearing on footings when the supporting surface is wider on all sides than the loaded area. Here A_2 is the area of the part of the supporting footing that is geometrically similar to and concentric with the loaded area (Fig. 13.15). Because $A_2 > A_1$, the factor $\sqrt{A_2/A_1}$ is greater than unity, indicating that the allowable

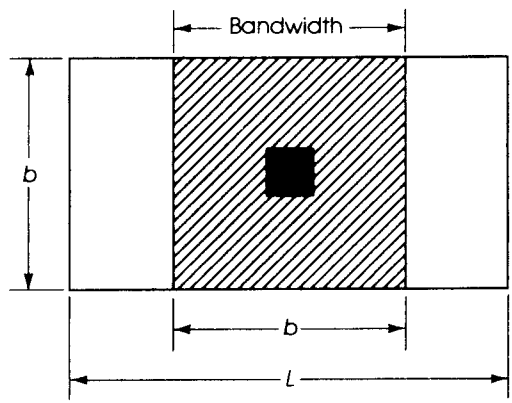


Figure 13.14 Bandwidth for reinforcement distribution.

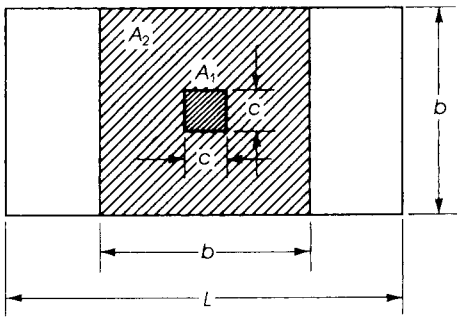


Figure 13.15 Bearing areas on footings. $A_1 = c^2$, $A_2 = b^2$.

bearing strength is increased because of the lateral support from the footing area surrounding the column base. The modified bearing strength is

$$N_2 = \phi(0.85 f'_c A_1)\sqrt{\frac{A_2}{A_1}} \leq 2\phi(0.85 f'_c A_1) \tag{13.18}$$

If the factored force, P_u, is greater than either N_1 or N_2 reinforcement must be provided to transfer the excess force. This is achieved by providing dowels or extending the column bars into the footing. The excess force is $P_{ex} = P_u - N_1$ and the area of the dowel bars is $A_{sd} = (P_{ex}/f_y) \geq 0.005\ A_1$, where A_1 is the area of the column section. At least four bars should be used at the four corners of the column. If the factored force is less than either N_1 or N_2, then minimum reinforcement must be provided. The ACI Code, Section 15.8.2, indicates that the minimum area of the dowel reinforcement is at least $0.005A_g$ (and not less than four bars), where A_g is the gross area of the column section. The minimum reinforcement requirements apply also to the case when the factored forces are greater than N_1 and N_2. The dowel bars may be placed at the four corners of the column and extended in both the column and footing. The dowel diameter shall not exceed the diameter of the longitudinal bars in the columns by more than 0.15 in. This requirement is necessary to ensure proper action between the column and footing. The development length of the dowels must be checked to determine proper transfer of the compression force into the footing.

13.4.6 Development Length of the Reinforcing Bars

The critical sections for checking the development length of the reinforcing bars are the same as those for bending moments. The development length for compression bars was given in Chapter 7:

$$l_{dc} = \frac{0.02 f_y d_b}{\lambda\sqrt{f'_c}} \tag{7.15}$$

but this value cannot be less than $0.0003\ f_y d_b \geq 8$ in. For other values, refer to Chapter 7.

13.4.7 Differential Settlement (Balanced Footing Design)

Footings usually support the following loads:

- Dead loads from the substructure and superstructure
- Live load resulting from occupancy

- Weight of materials used in backfilling
- Wind loads

Each footing in a building is designed to support the maximum load that may occur on any column due to the critical combination of loadings, using the allowable soil pressure.

The dead load, and maybe a small portion of the live load (called the *usual* live load), may act continuously on the structure. The rest of the live load may occur at intervals and on some parts of the structure only, causing different loadings on columns. Consequently, the pressure on the soil under different footings will vary according to the loads on the different columns, and differential settlement will occur under the various footings of one structure. Because partial settlement is inevitable, the problem turns out to be the amount of differential settlement that the structure can tolerate. The amount of differential settlement depends on the variation in the compressibility of the soils, the thickness of the compressible material below foundation level, and the stiffness of the combined footing and superstructure. Excessive differential settlement results in cracking of concrete and damage to claddings, partitions, ceilings, and finishes.

Differential settlement may be expressed in terms of angular distortion of the structure. Bjerrum [5] indicated that the danger limits of distortion for some conditions vary between $\frac{1}{600}$ to $\frac{1}{150}$ depending on the damage that will develop in the building.

For practical purposes it can be assumed that the soil pressure under the effect of sustained loadings is the same for all footings, thus causing equal settlements. The sustained load (or the usual load) can be assumed to be equal to the dead load plus a percentage of the live load, which occurs very frequently on the structure. Footings then are proportioned for these sustained loads to produce the same soil pressure under all footings. In no case is the allowable soil bearing capacity to be exceeded under the dead load plus the maximum live load for each footing. Example 13.4 explains the procedure for calculating the areas of footings, taking into consideration the effect of differential settlement.

13.5 PLAIN CONCRETE FOOTINGS

Plain concrete footings may be used to support masonry walls or other light loads and transfer them to the supporting soil. The ACI Code Section 22.7 allows the use of plain concrete pedestals and footings on soil, provided that the design stresses shall not exceed the following:

1. Maximum flexural stress in tension is less than or equal to $5\phi\lambda\sqrt{f'_c}$ (where $\phi = 0.60$).
2. Maximum stress in one-way shear (beam action) is less than or equal to $\frac{4}{3}\phi\lambda\sqrt{f'_c}$ (where $\phi = 0.60$).
3. Maximum shear stress in two-way action according to ACI Code Section 22.5.4 is

$$\left(\frac{4}{3} + \frac{8}{3\beta}\right)\phi\lambda\sqrt{f'_c} \leq 2.66\,\phi\sqrt{f'_c} \quad (\text{where } \phi = 0.60) \tag{13.19}$$

where

β = Ratio of long side to short side of the rectangular column
λ = modification factor described in 13.4.3.

4. Maximum compressive strength shall not exceed the concrete bearing strengths specified; f'_c of plain concrete should not be less than 2500 psi.

5. The minimum thickness of plain concrete footings shall not be less than 8 in.
6. The critical sections for bending moments are at the face of the column or wall.
7. The critical sections for one-way shear and two-way shear action are at distances d and $d/2$ from the face of the column or wall, respectively. Although plain concrete footings do not require steel reinforcement, it will be advantageous to provide shrinkage reinforcement in the two directions of the footing.
8. Stresses due to factored loads are computed assuming a linear distribution in concrete.
9. The effective depth, d, must be taken equal to the overall thickness minus 3 in.
10. For flexure and one-way shear, use a gross section bh, whereas for two-way shear, use b_0h to calculate ϕV_c.

Example 13.1

Design a reinforced concrete footing to support a 20-in.-wide concrete wall carrying a dead load of 26 K/ft, including the weight of the wall, and a live load of 20 K/ft. The bottom of the footing is 6 ft below final grade. Use normal-weight concrete with $f'_c = 4$ ksi, $f_y = 60$ ksi, and an allowable soil pressure of 5 ksf.

Solution

1. Calculate the effective soil pressure. Assume a total depth of footing of 20 in. Weight of footing is $(\frac{20}{12})(150) = 250$ psf. Weight of the soil fill on top of the footing, assuming that soil weighs 100 lb/ft^3, is $(6 - \frac{20}{12}) \times 100 = 433$ psf. Effective soil pressure at the bottom of the footing is $5000 - 250 - 433 = 4317$ psf $= 4.32$ ksf.
2. Calculate the width of the footing for a 1-ft length of the wall:

$$\text{Width of footing} = \frac{\text{total load}}{\text{effective soil pressure}}$$

$$= \frac{26 + 20}{4.32} = 10.7\text{ft}$$

Use 11 ft.

3. Net upward pressure = (factored load)/(footing width) (per 1 ft):

$$P_u = 1.2D + 1.6L = 1.2 \times 26 + 1.6 \times 20 = 63.2 \text{ K}$$

$$\text{Net pressure} = q_u = \frac{63.2}{11} = 5.745 \text{ ksf}$$

4. Check the assumed depth for shear requirements. The concrete cover in footings is 3 in., and assume no. 8 bars; then $d = 20 - 3.5 = 16.5$. The critical section for one-way shear is at a distance d from the face of the wall:

$$V_u = q_u\left(\frac{B}{2} - d - \frac{c}{2}\right) = 5.745\left(\frac{11}{2} - \frac{16.5}{12} - \frac{20}{2 \times 12}\right) = 18.91 \text{ K}$$

$$\text{Allowable one-way shear} = 2\lambda\sqrt{f'_c} = (2)(1)\sqrt{4000} = 126.5 \text{ psi}$$

$$\text{Required } d = \frac{V_u}{\phi(2\sqrt{f'_c})b} = \frac{18.91 \times 1000}{0.75(126.5)(12)} = 16.6 \text{ in.}$$

$$b = 1\text{-ft length of footing} = 12 \text{ in.}$$

Total depth is $16.6 + 3.5 = 20.1$ in., or 20 in. Actual d is $20 - 3.5 = 16.5$ in. (as assumed). Note that few trials are needed to get the assumed and calculated d quite close.

5. Calculate the bending moment and steel reinforcement. The critical section is at the face of the wall:

$$M_u = \frac{1}{2}q_u\left(\frac{B}{2} - \frac{c}{2}\right)^2 = \frac{5.745}{2}\left(\frac{11}{2} - \frac{20}{24}\right)^2 = 62.6 \text{ K·ft}$$

$$R_u = \frac{M_u}{bd^2} = \frac{62.6 \times 12{,}000}{12(16.5)^2} = 230 \text{ psi}$$

From Table A.1 in Appendix A, for $R_u = 230$ psi, $f'_c = 4$ ksi, and $f_y = 60$ ksi, the steel percentage is $\rho = 0.0045$ (or from Eq. 13.14). Minimum steel percentage for flexural members is

$$\rho_{\min} = \frac{200}{f_y} = \frac{200}{60{,}000} = 0.0033$$

Percentage of shrinkage reinforcement is 0.18% (for $f_y = 60$ ksi). Therefore, use $\rho = 0.0045$ as calculated.

$$A_s = 0.0045 \times 12 \times 16.5 = 0.89 \text{ in.}^2$$

Use no. 8 bars spaced at 9 in. ($A_s = 1.05$ in.^{2}) (Table A.14).

6. Check the development length for no. 8 bars:

$$l_d = 48d_b = 48(1) = 48 \text{ in. (Refer to Chapter 7).}$$

Provided

$$l_d = \frac{B}{2} - \frac{c}{2} - 3 \text{ in.} = \frac{11(12)}{2} - \frac{20}{2} - 3 = 53 \text{ in.}$$

7. Calculate secondary reinforcement in the longitudinal direction: $A_s = 0.0018(12)(20) = 0.43$ in.^{2}/ft. Choose no. 5 bars spaced at 8 in. ($A_s = 0.46$ in.^{2}). Details are shown in Fig. 13.16.

Example 13.2

Design a square single footing to support an 18-in.-square tied interior column reinforced with eight no. 9 bars. The column carries an unfactored axial dead load of 245 K and an axial live load of 200 K. The base of the footing is 4 ft below final grade and the allowable soil pressure is 5 ksf. Use normal-weight concrete, with $f'_c = 4$ ksi and $f_y = 60$ ksi.

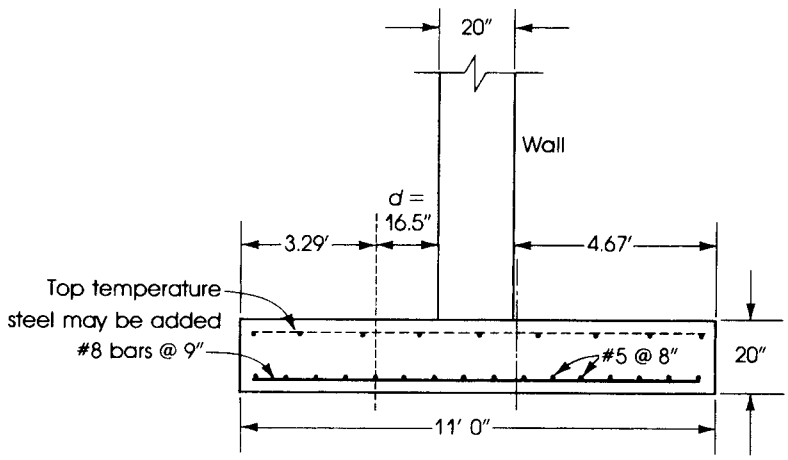


Figure 13.16 Example 13.1: Wall footing.

Solution

1. Calculate the effective soil pressure. Assume a total depth of footing of 2 ft. The weight of the footing is $2 \times 150 = 300$ psf. The weight of the soil on top of the footing (assuming the weight of soil = 100 pcf) is $2 \times 100 = 200$ psf.

$$\text{Effective soil pressure} = 5000 - 300 - 200 = 4500 \text{ psf}$$

2. Calculate the area of the footing:

$$\text{Actual loads} = D + L = 245 + 200 = 445 \text{ K}$$

$$\text{Area of footing} = \frac{445}{4.5} = 98.9 \text{ ft}^2$$

$$\text{Side of footing} = 9.94 \text{ ft.}$$

Thus, use 10 ft (Fig.13.17).

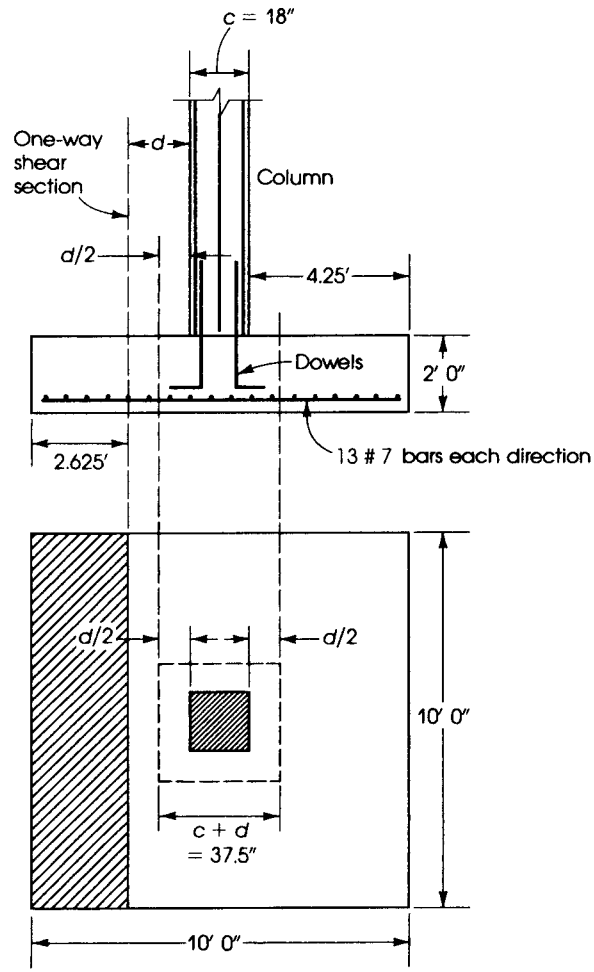


Figure 13.17 Example 13.2: Square footing.

3. Net upward pressure equals (factored load)/(area of footing).

$$P_u = 1.2D + 1.6L$$

$$= 1.2 \times 245 + 1.6 \times 200 = 614 \text{ K}$$

$$\text{Net upward pressure, } q_u = \frac{614}{10 \times 10} = 6.14 \text{ ksf}$$

4. Check depth due to two-way shear. If no shear reinforcement is used, two-way shear determines the critical footing depth required. For an assumed total depth of 24 in., calculate d to the centroid of the top layer of the steel bars to be placed in the two directions within the footing. Let the bars to be used be no. 8 bars for calculating d.

$$d = 24 - 3 \text{ (cover)} - 1.5 \text{ (bar diameters)} = 19.5 \text{ in.}$$

It is quite practical to assume $d = h - 4.5$ in.

$$b_0 = 4(c + d) = 4(18 + 19.5) = 150 \text{ in.}$$

$$c + d = 18 + 19.5 = 37.5 \text{ in.} = 3.125 \text{ ft}$$

$$V_{u_2} = P_u - q_u(c + d)^2 = 614 - 6.14(3.125)^2 = 554 \text{ K}$$

$$\text{Required } d_1 = \frac{V_{u_2}}{4\phi\lambda(\sqrt{f'_c}b_0)}$$

$$= \frac{554(1000)}{(4)(0.75)(1)\sqrt{4000}(150)} = 19.5 \text{ in.} \quad (\beta = 1; \text{ Eq. } 13.9)$$

$$\text{Required } d_2 = \frac{554(1000)}{0.75\left(\dfrac{40 \times 19.5}{150} + 2\right)(\sqrt{4000})(150)}$$

$$= 10.8 \text{ in.} \quad \text{(not critical)}$$

($\alpha_s = 40$ for interior columns.) Thus, the assumed depth is adequate. Two or more trials may be needed to reach an acceptable d that is close to the assumed one.

5. Check depth due to one-way shear action: The critical section is at a distance d from the face of the column.

$$\text{Distance from edge of footing} = \left(\frac{L}{2} - \frac{c}{2} - d\right) = 2.625 \text{ ft}$$

$$V_{u_1} = 6.14 \times (2.625)(10) = 161.2 \text{ K}$$

The depth required for one-way shear is

$$d = \frac{V_{u_1}}{(0.75)(2)\lambda\sqrt{f'_c}b}$$

$$= \frac{161.2(1000)}{(0.75)(2)(1)(\sqrt{4000})(10 \times 12)} = 14.2 \text{ in.} < 19.5 \text{ in.}$$

6. Calculate the bending moment and steel reinforcement. The critical section is at the face of the column. The distance from edge of footing is

$$\left(\frac{L}{2} - \frac{c}{2}\right) = 5 - \frac{1.5}{2} = 4.25 \text{ ft}$$

$$M_u = \frac{1}{2}q_u\left(\frac{L}{2} - \frac{c}{2}\right)^2 b = \frac{1}{2}(6.14)(4.25)^2(10) = 554.5 \text{ K·ft}$$

$$R_u = \frac{M_u}{bd^2} = \frac{554.5(12{,}000)}{(10 \times 12)(19.5)^2} = 145.8 \text{ psi}$$

Applying Eq. 13.14, $\rho = 0.0028$.

$$A_s = \rho bd = 0.0028(10 \times 12)(19.5) = 6.55 \text{ in.}^2$$

$$\text{Minimum } A_s \text{ (shrinkage steel)} = 0.0018(10 \times 12)(24)$$

$$= 5.18 \text{ in.}^2 < 6.55 \text{ in.}^2$$

$$\text{Minimum } A_s \text{ (flexure)} = 0.0033(10 \times 12)(19.5) = 7.72 \text{ in.}^2$$

Therefore, $A_s = 7.72 \text{ in.}^2$ can be adopted. Use 13 no. 7 bars ($A_s = 7.82 \text{ in.}^2$), spaced at $s = (120 - 6)/12 = 9.5$ in. in both directions.

7. Check bearing stress:
   a. Bearing strength, N_1 at the base of the column ($A_1 = 18 \times 18$ in.) is

$$N_1 = \phi(0.085 f'_c A_1) = 0.65(0.85 \times 4)(18 \times 18) = 716 \text{ K}$$

   b. Bearing strength, N_2, at the top of footing ($A_2 = 10 \times 10$ ft) is

$$N_2 = N_1\sqrt{\frac{A_2}{A_1}} \le 2N_1$$

$$A_2 = 10 \times 10 = 100 \text{ ft}^2 \quad A_1 = \frac{18 \times 18}{144} = 2.25 \text{ ft}^2$$

$$\sqrt{\frac{A_2}{A_1}} = 6.67 > 2$$

   Therefore, $N_2 = 2N_1 = 1432$ K. Because $P_u = 614 \text{ K} < N_1$, bearing stress is adequate. The minimum area of dowels required is $0.005\ A_1 = 0.005\ (18 \times 18) = 1.62 \text{ in.}^2$. The minimum number of bars is four, so use four no. 8 bars placed at the four corners of the column.
   c. Development length of dowels in compression:

$$l_{dc} = \frac{0.02 d_b f_y}{\lambda\sqrt{f'_c}} = \frac{0.02(1)(60{,}000)}{(1)\sqrt{4000}} = 19 \text{ in.}$$

   (controls). Minimum l_{dc} is $0.0003 d_b f_y = 0.0003(1)\ (60{,}000) = 18$ in. ≥ 8 in. Therefore, use four no. 8 dowels extending 19 in. into column and footing. Note that l_d is less than d of 19.5 in., which is adequate.
8. The development length of main bars in footing for no. 7 bars is $l_d = 48 d_b = 42$ in. (refer to Chapter 7), provided $l_d = L/2 - c/2 - 3$ in. $= 48$ in. Details of the footing are shown in Fig. 13.17 on page 430.

Example 13.3

Design a rectangular footing for the column of Example 13.2 if one side of the footing is limited to 8.5 ft.

Solution

1. The design procedure for rectangular footings is similar to that of square footings, taking into consideration the forces acting on the footing in each direction separately.
2. From the previous example, the area of the footing required is 98.9 ft^2:

$$\text{Length of footing} = \frac{98.9}{8.5} = 11.63 \quad \text{ft}$$

so use 12 ft (Fig. 13.18). Footing dimensions are 8.5 × 12 ft.

3. $P_u = 614$ K. Thus, net upward pressure is

$$q_u = \frac{614}{8.5 \times 12} = 6.02 \text{ ksf}$$

4. Check the depth due to one-way shear. The critical section is at a distance d from the face of the column. In the longitudinal direction,

$$V_{u_1} = \left(\frac{L}{2} - \frac{c}{2} - d\right) \times q_u b$$

$$= \left(\frac{12}{2} - \frac{1.5}{2} - \frac{19.5}{12}\right) \times 6.02 \times 8.5 = 185.5 \text{ K}$$

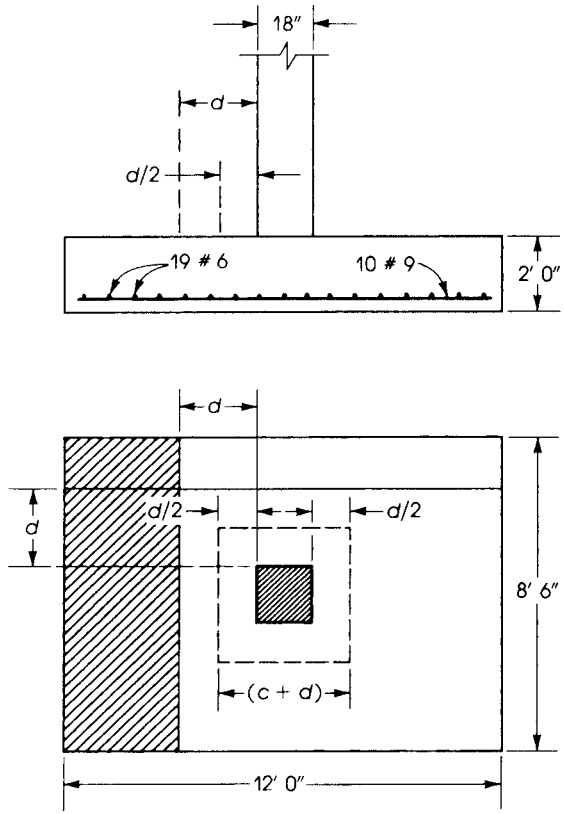


Figure 13.18 Example 13.3: Rectangular footing.

This shear controls. In the short direction, $V_u = 135.4$ K (not critical).

$$\text{Required } d = \frac{V_{u_1}}{2\phi\lambda\sqrt{f'_c}b} = \frac{185.5 \times 1000}{(2)(0.75)(1)\sqrt{4000} \times (8.5 \times 12)} = 19.2 \text{ in.}$$

d provided $= 19.5$ in. > 19.2 in.

5. Check the depth for two-way shear action (punching shear). The critical section is at a distance $d/2$ from the face of the column on four sides.

$$b_0 = 4(18 + 19.5) = 150 \text{ in.}$$

$$(c + d) = 18 + 19.5 = 37.5 \text{ in.} = 3.125 \text{ ft}$$

$$\beta = \frac{12}{8.5} = 1.41 < 2$$

(Use $V_c = 4\phi\lambda\sqrt{f'_c}b_0 d$.)

$$V_{u_2} = P_u - q_u(c + d)^2 = 614 - 6.02(3.125)^2 = 555.2 \text{ K}$$

$$d_1 = \frac{V_{u_2}}{4\phi\lambda\sqrt{f'_c}b_0} = \frac{555.2 \times 1000}{4(0.75)(1)\sqrt{4000} \times 150} = 19.5 \text{ in.}$$

$d_2 = 10.6$ in. (Does not control.)

6. Design steel reinforcement in the longitudinal direction. The critical section is at the face of the support. The distance from the edge of the footing is

$$\frac{L}{2} - \frac{c}{2} = \frac{12}{2} - \frac{1.5}{2} = 5.25 \text{ ft}$$

$$M_u = \frac{1}{2}(6.02)(5.25)^2(8.5) = 705.2 \text{ K·ft}$$

$$R_u = \frac{M_u}{bd^2} = \frac{705.2(12{,}000)}{(8.5 \times 12)(19.5)^2} = 218 \text{ psi}$$

Applying Eq. 13.14, $\rho = 0.0042$:

$$A_s = 0.0042(8.5 \times 12)(19.5) = 8.35 \text{ in.}^2$$

$$\text{Min } A_s \text{ (shrinkage)} = 0.0018(8.5 \times 12)(24) = 4.4 \text{ in.}^2$$

$$\text{Min } A_s \text{ (flexure)} = 0.0033(8.5 \times 12)(19.5) = 6.56 \text{ in.}^2$$

Use $A_s = 8.35$ in. and 10 no. 9 bars ($A_s = 10$ in.^{2}) spaced at $S = (102 - 6)/9 = 10.7$ in.

7. Design steel reinforcement in the short direction. The distance from the face of the column to the edge of the footing is

$$\frac{8.5}{2} - \frac{1.5}{2} = 3.5 \text{ ft}$$

$$M_u = \frac{1}{2}(6.02)(3.5)^2(12) = 422.5 \text{ K·ft}$$

$$R_u = \frac{M_u}{bd^2} = \frac{442.5(12{,}000)}{(12 \times 12)(19.5)^2} = 97 \text{ psi}$$

Applying Eq. 13.4, $\rho = 0.0019$:

$$A_s = 0.0019(12 \times 12)(19.5) = 5.34 \text{ in.}^2$$

$$\text{Min } A_s \text{ (shrinkage)} = 0.0018(12 \times 12)(24) = 6.22 \text{ in.}^2$$

$$\text{Min } A_s \text{ (flexure)} = 0.0033(12 \times 12)(19.5) = 9.26 \text{ in.}^2$$

The value of A_s to be used must be greater than or equal to 6.22 in.^{2} Use 18 no. 6 bars (A_s = 7.92 in.^{2}).

$$\gamma_s = \frac{2}{\beta + 1} = \frac{2}{\left(\frac{12}{8.5}\right) + 1} = 0.83$$

The number of bars in an 8.5-ft band is 18(0.83) = 15 bars. The number of bars left on each side is (18 − 15)/2 = 2 bars. Therefore, place 15 no. 6 bars within the 8.5-ft band; then place two no. 6 bars (A_s = 0.88 in.^{2}) within (12 − 8.5)/2 = 1.625 ft on each side of the band. The total number of bars is 19 no. 6 bars (A_s = 8.36 in.^{2}). In this example, the bars may be distributed at equal spacings all over the 12-ft length; S = (144 − 6)/18 = 7.6 in. Details of reinforcement are shown in Fig.13.18.

8. Check the bearing stress at the base of the column, as explained in the previous example. Use four no. 8 dowel bars.
9. Development length of the main reinforcement: l_d = 29 in. for no. 6 bars and 54 in. for no. 9 bars.

$$\text{Provided } l_d \text{ (long direction)} = \left(\frac{l}{2} - \frac{c}{2} - 3 \text{ in.}\right) = 60 \text{ in.}$$

$$\text{Provided } l_d \text{ (short direction)} = 39 \text{ in.} > 29 \text{ in.}$$

Example 13.4

Determine the footing areas required for equal settlement (balanced footing design) if the usual live load is 20% for all footings. The footings are subjected to dead loads and live loads as indicated in the following table. The allowable net soil pressure is 6 ksf.

	Footing Number				
	1	2	3	4	5
Dead load	120 K	180 K	140 K	190 K	210 K
Live load	150 K	220 K	200 K	170 K	240 K

Solution

1. Determine the footing that has the largest ratio of live load to dead load. In this example, the footing 3 ratio of 1.43 is higher than the other ratios.
2. Calculate the usual load for all footings. The usual load is the dead load and the portion of live load that most commonly occurs on the structure. In this example,

$$\text{Usual load} = \text{D.L.} + 0.2(\text{L.L.})$$

The values of the usual loads are shown in the following table.

3. Determine the area of the footing that has the highest ratio of L.L./D.L.

$$\text{Area of footing 3} = \frac{\text{D.L.} + \text{L.L}}{\text{allowable soil pressure}} = \frac{140 + 200}{6} = 56.7 \text{ ft}^2$$

The usual soil pressure under footing 3 is

$$\frac{\text{Usual load}}{\text{Area of footing}} = \frac{180}{56.7} = 3.18 \text{ ksf}$$

4. Calculate the area required for each footing by dividing its usual load by the soil pressure of footing 3. The areas are tabulated in the following table. For footing 1, for example, the required area is $150/3.18 = 47.2\,\text{ft}^2$.
5. Calculate the maximum soil pressure under each footing:

$$q_{\max} = \frac{D+L}{\text{area}} \leq 6 \text{ ksf} \quad \text{(allowable soil pressure)}$$

Description	Footing Number 1	2	3	4	5
$\frac{\text{Live load}}{\text{Dead load}}$	1.25	1.22	1.43	0.90	1.14
Usual load = D.L. + 0.2 (L.L.) (kips)	150	224	180	224	258
Area required = $\frac{\text{usual load}}{3.18 \text{ ksf}}(\text{ft}^2)$	47.2	70.4	56.7	70.4	81.1
Max. soil pressure = $\frac{D+L}{\text{area}}$(ksf)	5.72	5.68	6.00	5.11	5.55

Example 13.5

Design a plain concrete footing to support a 16-in.-thick concrete wall. The loads on the wall consist of a 16-K/ft dead load (including the self-weight of wall) and a 10-K/ft live load. The base of the footing is 4 ft below final grade. Use $f'_c = 3$ ksi and an allowable soil pressure of 5 ksf.

Solution

1. Calculate the effective soil pressure. Assume a total depth of footing of 28 in.

$$\text{Weight of footing} = \frac{28}{12} \times 145 = 338 \text{ psf}$$

The weight of the soil, assuming that soil weighs 100 pcf, is $(4 - 2.33) \times 100 = 167$ psf. Effective soil pressure is $5000 - 338 - 167 = 4495$ psf.

2. Calculate the width of the footing for a 1-ft length of the wall ($b = 1$ ft):

$$\text{Width of footing} = \frac{\text{total load}}{\text{effective soil pressure}}$$

$$= \frac{16+10}{4.495} = 5.79 \text{ ft}$$

Use 6.0 ft (Fig. 13.19).

3. $U = 1.2D + 1.6L = 1.2 \times 16 + 1.6 \times 10 = 35.2$ K/ft. The net upward pressure is $q_u = 35.2/6 = 5.87$ ksf.
4. Check bending stresses. The critical section is at the face of the wall. For a 1-ft length of wall and footing,

$$M_u = \frac{1}{2} q_u \left(\frac{L}{2} - \frac{c}{2}\right)^2 = \frac{1}{2}(5.87)\left(\frac{6}{2} - \frac{16}{2 \times 12}\right)^2 = 16 \text{ K·ft}$$

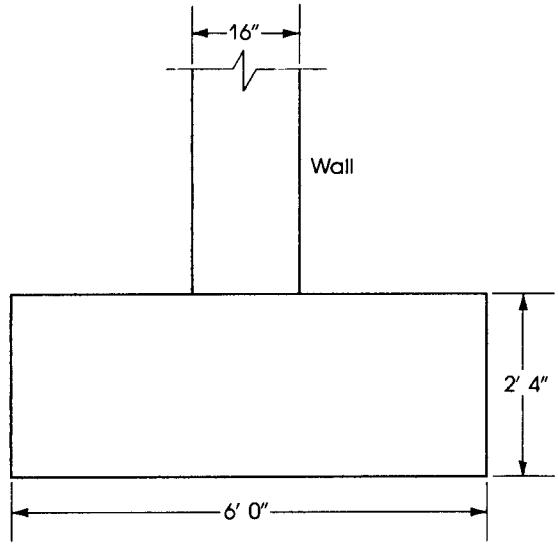


Figure 13.19 Example 13.5: Plain concrete wall footing.

Let the effective depth, d, be $28 - 3 = 25$ in., assuming that the bottom 3 in. is not effective.

$$I_g = \frac{bd^3}{12} = \frac{12}{12}(25)^3 = 15{,}625 \text{ in.}^4$$

The flexural tensile stress is

$$f_t = \frac{M_u c}{I} = \frac{(16 \times 12{,}000)}{15{,}625}\left(\frac{25}{2}\right) = 153 \text{ psi}$$

The allowable flexural tensile stress is $5\phi\sqrt{f'_c} = 5 \times 0.55\sqrt{3000} = 151$ psi (close).

5. Check shear stress: The critical section is at a distance $d = 25$ in. from the face of the wall.

$$V_u = q_u\left(\frac{L}{2} - \frac{c}{2} - d\right) = 5.87\left(\frac{6}{2} - \frac{16}{2 \times 12} - \frac{25}{12}\right) = 1.47 \text{ K}$$

$$\phi V_c = \phi\left(\frac{4}{3}\right)\lambda\sqrt{f'_c}bd = \frac{(0.55)\left(\frac{4}{3}\right)(1)\sqrt{3000}(12)(25)}{1000} = 12.05 \text{ K}$$

Therefore, the section is adequate. It is advisable to use minimum reinforcement in both directions.

13.6 COMBINED FOOTINGS

When a column is located near a property line, part of the single footing might extend into the neighboring property. To avoid this situation, the column may be placed on one side or edge of the footing, causing eccentric loading. This may not be possible under certain conditions, and sometimes it is not an economical solution. A better design can be achieved by combining the footing with the nearest internal column footing, forming a combined footing. The center of gravity of the combined footing coincides with the resultant of the loads on the two columns.

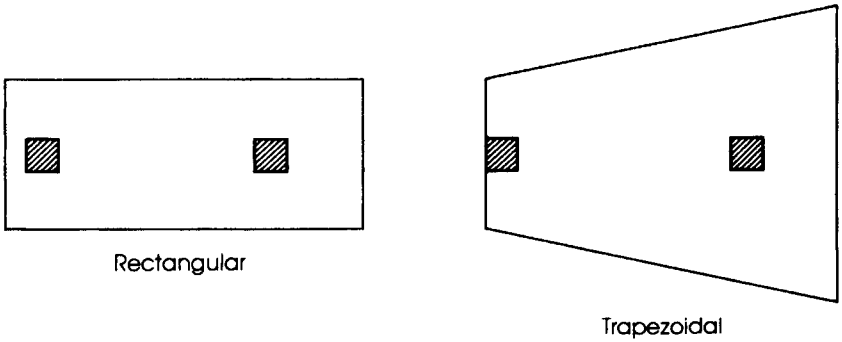


Figure 13.20 Combined footings.

Another case where combined footings become necessary is when the soil is poor and the footing of one column overlaps the adjacent footing. The shape of the combined footing may be rectangular or trapezoidal (Fig. 13.20). When the load of the external column near the property line is greater than the load of the interior column, a trapezoidal footing may be used to keep the centroid of footing in line with the resultant of the two column loads. In most other cases, a rectangular footing is preferable.

The length and width of the combined footing are chosen to the nearest 3 in., which may cause a small variation in the uniform pressure under the footing, but it can be tolerated. For a uniform upward pressure, the footing will deflect, as shown in Fig. 13.21. The ACI Code, Section 15.10, does not provide a detailed approach for the design of combined footings. The design, in general, is based on structural analysis.

A simple method of analysis is to treat the footing as a beam in the longitudinal direction, loaded with uniform upward pressure, q_u. For the transverse direction, it is assumed that the

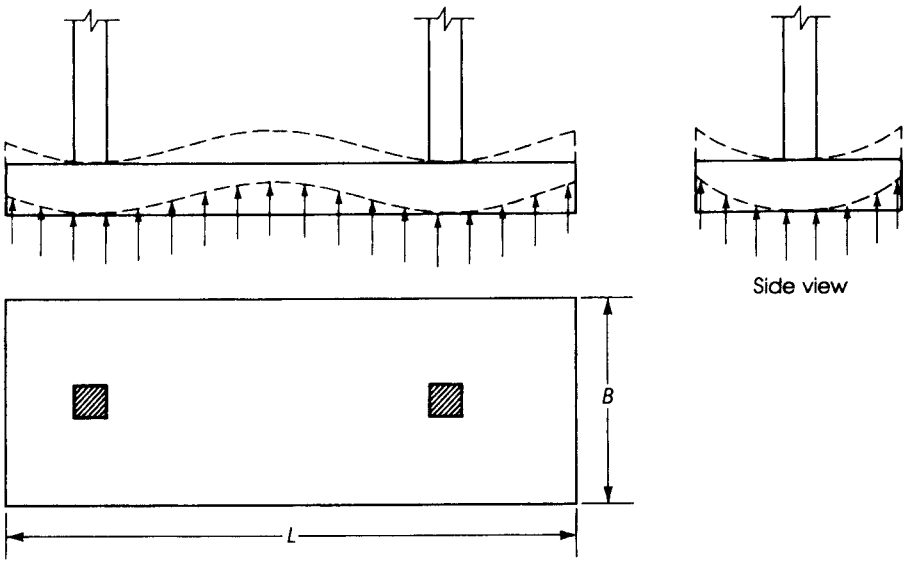


Figure 13.21 Upward deflection of a combined footing in two directions.

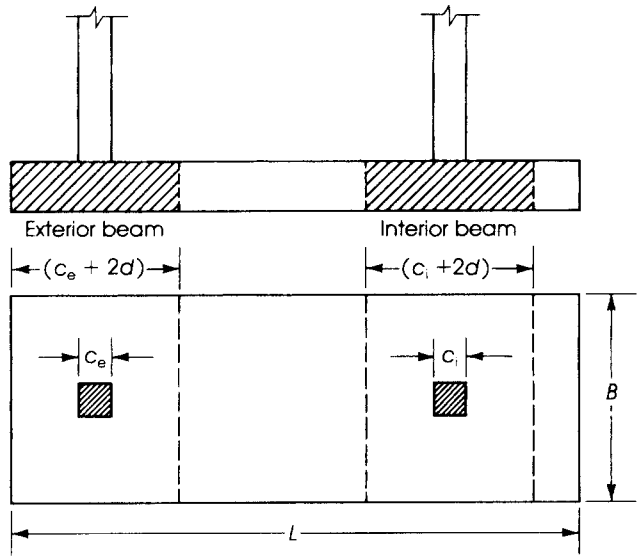


Figure 13.22 Analysis of combined footing in the transverse direction.

column load is spread over a width under the column equal to the column width plus d on each side, whenever that is available. In other words, the column load acts on a beam under the column within the footing, which has a maximum width of $(c + 2d)$ and a length equal to the short side of the footing (Fig. 13.22). A smaller width, down to $(c + d)$, may be used. The next example explains the design method in detail.

Example 13.6

Design a rectangular combined footing to support two columns, as shown in Fig. 13.23. The edge column, I, has a section 16 by 16 in. and carries a D.L. of 180 K and an L.L. of 120 K. The interior column, II, has a section 20 by 20 in. and carries a D.L. of 250 K and an L.L. of 140 K. The allowable soil pressure is 5 ksf and the bottom of the footing is 5 ft below final grade. Design the footing using $f'_c = 4$ ksi, $f_y = 60$ ksi, and the ACI strength design method.

Solution

1. Determine the location of the resultant of the column loads. Take moments about the center of the exterior column I:

$$x = \frac{(250 + 140) \times 16}{(250 + 140) + (180 + 120)} = 9 \text{ ft from column I}$$

The distance of the resultant from the property line is $9 + 2 = 11.0$ ft. The length of the footing is $2 \times 11 = 22.0$ ft. In this case the resultant of column loads will coincide with the resultant of the upward pressure on the footing.

2. Determine the area of the footing. Assume the footing total depth is 36 in. ($d = 36 - 4.5 = 31.5$ in.)

$$\text{Total actual (working) loads} = 300 + 390 = 690 \text{ K}$$

$$\text{New upward pressure} = 5000 - \left(\frac{36}{12} \times 150\right) - (2 \times 100) = 4500 \text{ psf}$$

(Assumed weight of soil is 100 psf.)

$$\text{Required area} = \frac{690}{4.5} = 153.3 \text{ ft}^2$$

$$\text{Width of footing} = \frac{153.3}{22} = 6.97 \text{ ft}$$

Use 7 ft. Choose a footing 22 by 7 ft (area = 154 ft^2).

3. Determine the factored upward pressure using factored loads:

$$P_{u1} \text{ (column I)} = 1.2 \times 180 + 1.6 \times 120 = 408 \text{ K}$$

$$P_{u2} \text{ (column II)} = 1.2 \times 250 + 1.6 \times 140 = 524 \text{ K}$$

The net factored soil pressure is $q_u = (408 + 524)/154 = 6.05$ ksf.

4. Draw the factored shearing force diagram as for a beam of $L = 22$ ft supported on two columns and subjected to an upward pressure of 6.05 ksf × 7 (width of footing) = 42.35 K/ft (per foot length of footing).

$$V_u \text{ (at outer face column I)} = 42.35\left(2 - \frac{8}{12}\right) = 56.5 \text{ K}$$

$$V_u \text{ (at interior face column I)} = 408 - 42.35\left(2 + \frac{8}{12}\right) = 295 \text{ K}$$

$$V_u \text{ (at outer face column II)} = 42.35\left(4 - \frac{10}{12}\right) = 134.1 \text{ K}$$

$$V_u \text{ (at interior face column I)} = 524 - \left(4 + \frac{10}{12}\right) \times 42.35 = 319.3 \text{ K}$$

Find the point of zero shear, x; distance between interior faces of columns I and II is

$$16 - \frac{8}{12} - \frac{10}{12} = 14.5 \text{ ft}$$

$$x = \frac{295}{(295 + 319.3)}(14.5) = 6.9 \text{ ft}$$

5. Draw the factored moment diagram considering the footing as a beam of $L = 22$ ft supported by the two columns. The uniform upward pressure is 47.5 K/ft.

$$M_{u1} \text{ (at outer face column I)} = 423.5\frac{(1.33)^2}{2} = 37.6 \text{ K·ft}$$

$$M_{u2} \text{ (at outer face column II)} = 42.35\frac{(3.17)^2}{2} = 212.8 \text{ K·ft}$$

The maximum moment occurs at zero shear:

$$\text{Maximum } M_u \text{ (calculated from column I side)} = 408\left(6.9 + \frac{8}{12}\right) - \frac{42.35}{2}\left(6.9 + \frac{8}{12} + 2\right)^2$$

$$= 1149 \text{ K·ft}$$

$$\text{Maximum } M_u \text{ (from column II side)} = 524\left(7.6 + \frac{10}{12}\right) - \frac{42.35}{2}\left(7.6 + \frac{10}{12} + 4\right)^2$$

$$= 1146 \text{ K·ft}$$

The moments calculated from both sides of the footings are close enough, and $M_{u\max} = 1149$ K·ft may be adopted. This variation occurred mainly because of the adjustment of the length and width of the footing.

6. Check the depth for one-way shear. Maximum shear occurs at a distance $d = 31.5$ in. from the interior face of column II (Fig. 13.23).

$$V_{u_1} = 319.3 - \frac{31.5}{12}(42.35) = 208.7 \text{ K}$$

$$d = \frac{V_{u1}}{\phi(2\lambda\sqrt{f'_c})b} = \frac{202.7 \times 100}{0.75(2(1)\sqrt{4000})(7 \times 12)} = 26.1 \text{ in.}$$

The effective depth provided is 31.5 in. > 26.1 in.; thus, the footing is adequate.

7. Check depth for two-way shear (punching shear). For the interior column,

$$b_0 = 4(c + d) = \left(\frac{4}{12}\right)(20 + 31.5) = 17.17 \text{ ft}$$

$$(c + d) = \frac{20 + 31.5}{12} = 4.29 \text{ ft}$$

The shear V_{u_2} at a section $d/2$ from all sides of the column is equal to

$$V_{u_2} = P_{u2} - q_u(c + d)^2 = 524 - 6.05(4.29)^2 = 413 \text{ K}$$

$$d = \frac{V_{u_2}}{\phi(4\lambda\sqrt{f'_c})b_0} = \frac{413(1000)}{0.75(4(1)\sqrt{4000})(17.7 \times 12)} = 10.3 \text{ in.} < 31.5 \text{ in.}$$

The exterior column is checked and proved not to be critical.

8. Check the depth for moment and determine the required reinforcement in the long direction.

Maximum bending moment = 1149 K·ft

$$R_u = \frac{M_u}{bd^2} = \frac{1149(12{,}000)}{(7 \times 12)(31.5)^2} = 166 \text{ psi}$$

Applying Eq. 13.14, the steel percentage is $\rho = 0.0033 = 0.0033$ ($\rho_{\min}$).

$$A_s = 0.0033(84 \times 31.5) = 8.73 \text{ in.}^2$$

$$\text{Min } A_s \text{ (shrinkage)} = 0.0018(84)(36) = 5.44 \text{ in.}^2$$

$A_s = 8.73$ in.^{2} controls. Use 10 no. 9 bars ($A_s = 10$ in.^{2}).

$$\text{Spacing of bars} = \frac{84 - 6 \text{ (concrete cover)}}{9 \text{ (no. of spacings)}} = 8.67 \text{ in.}$$

The bars are extended between the columns at the top of the footing with a concrete cover of 3 in. Place minimum reinforcement at the bottom of the projecting ends of the footing beyond the columns to take care of the positive moments. Extend the bars a development length l_d beyond the side of the column.

The minimum shrinkage reinforcement is 5.44 in.^{2} Use seven no. 8 bars ($A_s = 5.5$ in.^{2})

The development length required for the main top bars is $1.3l_d = 1.3(54) = 70$ in. beyond the point of maximum moment. Development lengths provided to both columns are adequate.

9. For reinforcement in the short direction, calculate the bending moment in the short (transverse) direction, as in the case of single footings. The reinforcement under each column is to be placed within a maximum bandwidth equal to the column width twice the effective depth d of the footing (Fig. 13.24).

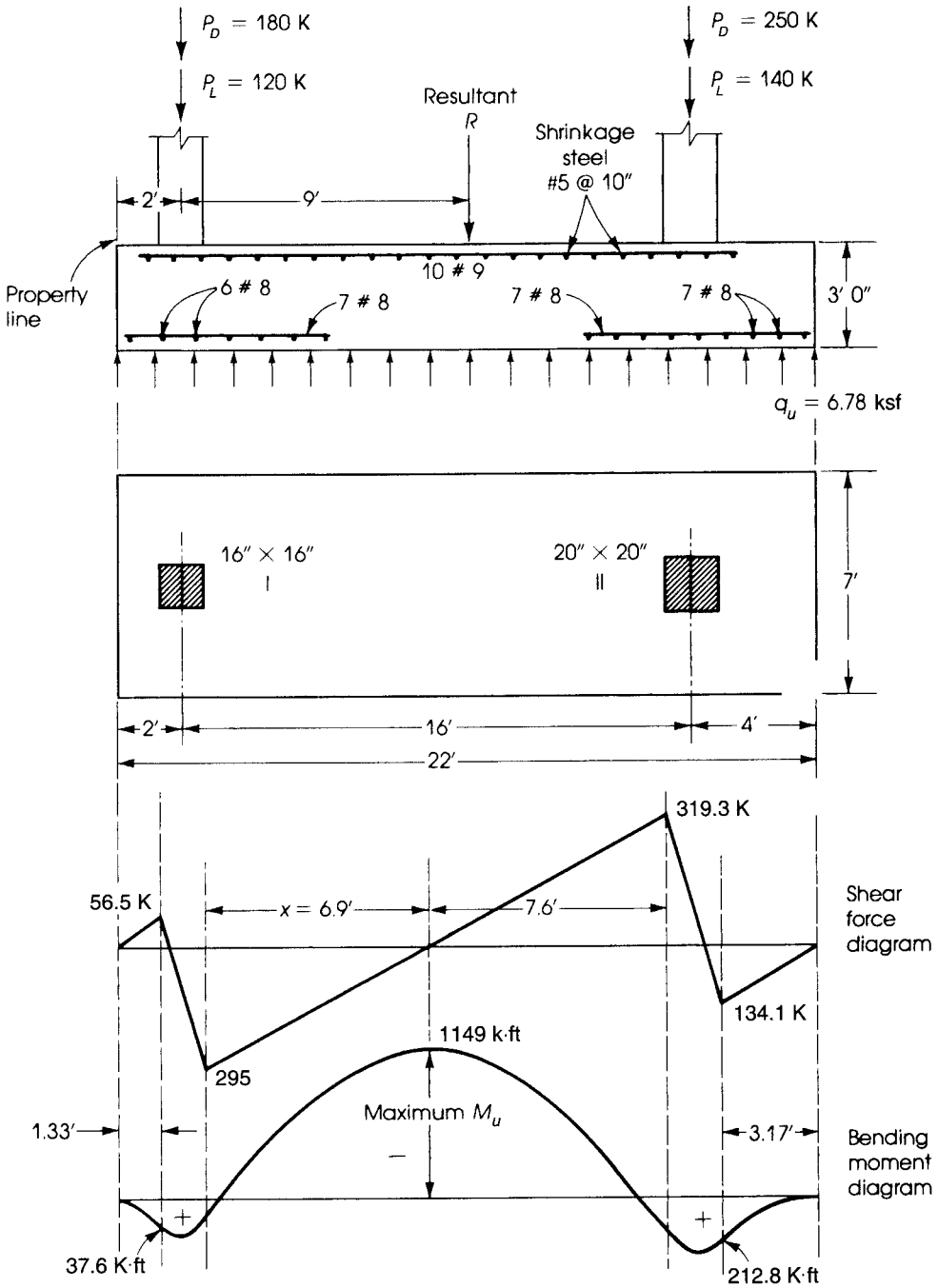


Figure 13.23 Example 13.6: Design of a combined footing.

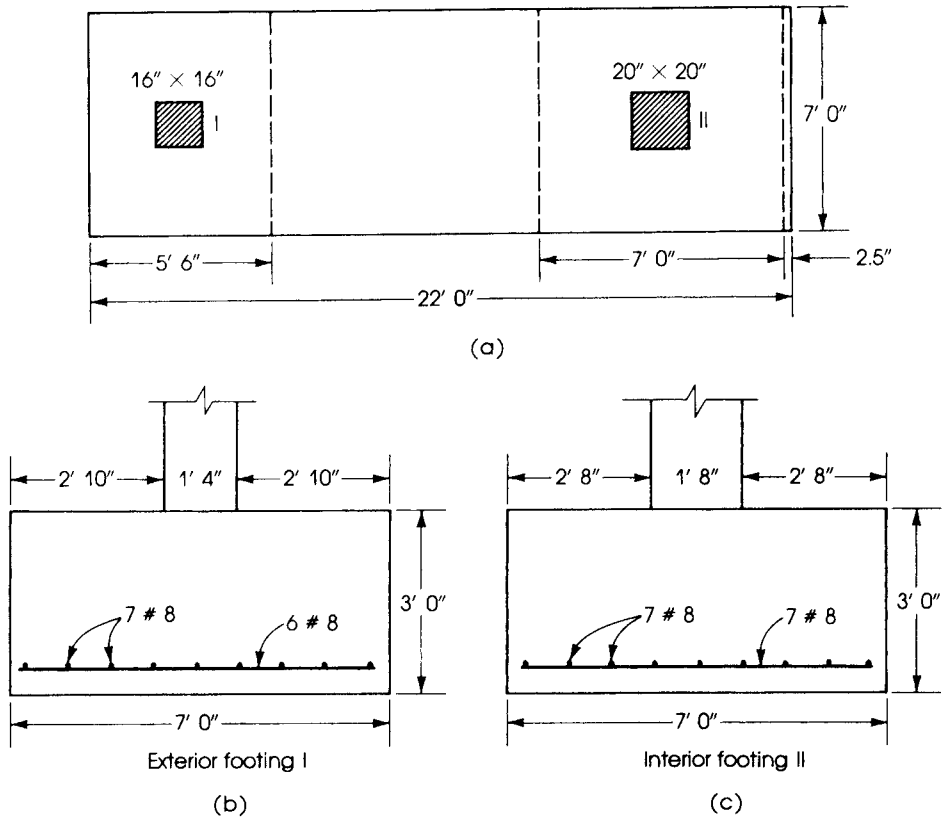


Figure 13.24 Design of combined footing, transverse direction: (*a*) plan, (*b*) exterior footing, and (*c*) interior footing.

a. Reinforcement under exterior column I:

$$\begin{aligned}\text{Bandwidth} &= 16 \text{ in. (column width)} \\ &\quad + 16 \text{ in. (on exterior side of column)} \\ &\quad + 31.5 \text{ in. } (d) \\ &= 63.5 \text{ in.} = 5.3 \text{ ft}\end{aligned}$$

Use 5.5 ft. Net upward pressure in the short direction under column I is

$$\frac{P_{u1}}{\text{width of footing}} = \frac{408}{7} = 58.3 \text{ K/ft}$$

Distance from the free end to the face of the column is $\frac{7}{2} - \frac{8}{12} = 2.83$ ft.

$$M_u \text{ (at face of column I)} = \frac{58.3}{2}(2.83)^2 = 233.5 \text{ K·ft}$$

$$R_u = \frac{M_u}{bd^2} = \frac{233.5 \times 12{,}000}{(5.5 \times 12)(31.5)^2} = 43 \text{ psi}$$

The steel percentage, ρ, is less than minimum ρ for shrinkage reinforcement ratio of 0.0018.

$$A_{s\min} = (0.0018)(5.5 \times 12)(36) = 4.3 \text{ in.}^2$$

Use six no. 8 bars ($A_s = 4.71$ in.^{2}) placed within the bandwidth of 66 in.

b. Reinforcement under the interior column II:

$$\text{Bandwidth} = 20 + 31.5 + 31.5 = 83 \text{ in.} = 6.91 \text{ ft}$$

Use 7 ft (84 in.).

$$\text{Net upward pressure} = \frac{P_{u2}}{\text{width of footing}} = \frac{524}{7} = 75 \text{ K/ ft}$$

$$\text{Distance to face of column} = \frac{7}{2} - \frac{10}{12} = 2.67 \text{ ft}$$

$$M_u \text{ (at face of column II in short direction)} = \tfrac{1}{2}(75)(2.67)^2 = 267 \text{ K·ft}$$

$$R_u = \frac{(267)(12{,}000)}{(84)(31.5)^2} = 38 \text{ psi}$$

which is very small. Use a minimum shrinkage reinforcement ratio of 0.0018.

$$A_s = (0.0018)(84)(36) = 5.44 \text{ in.}^2$$

Use seven no. 8 bars placed within the bandwidth of 84 in. under column II, as shown in Figs. 13.23 and 13.24. The development length l_d of no. 8 bars in the short direction is 48 in.

13.7 FOOTINGS UNDER ECCENTRIC COLUMN LOADS

When a column transmits axial loads only, the footing can be designed such that the load acts at the centroid of the footing, producing uniform pressure under the footing. However, in some cases, the column transmits an axial load and a bending moment, as in the case of the footings of fixed-end frames. The pressure q that develops on the soil will not be uniform and can be evaluated from the following equation:

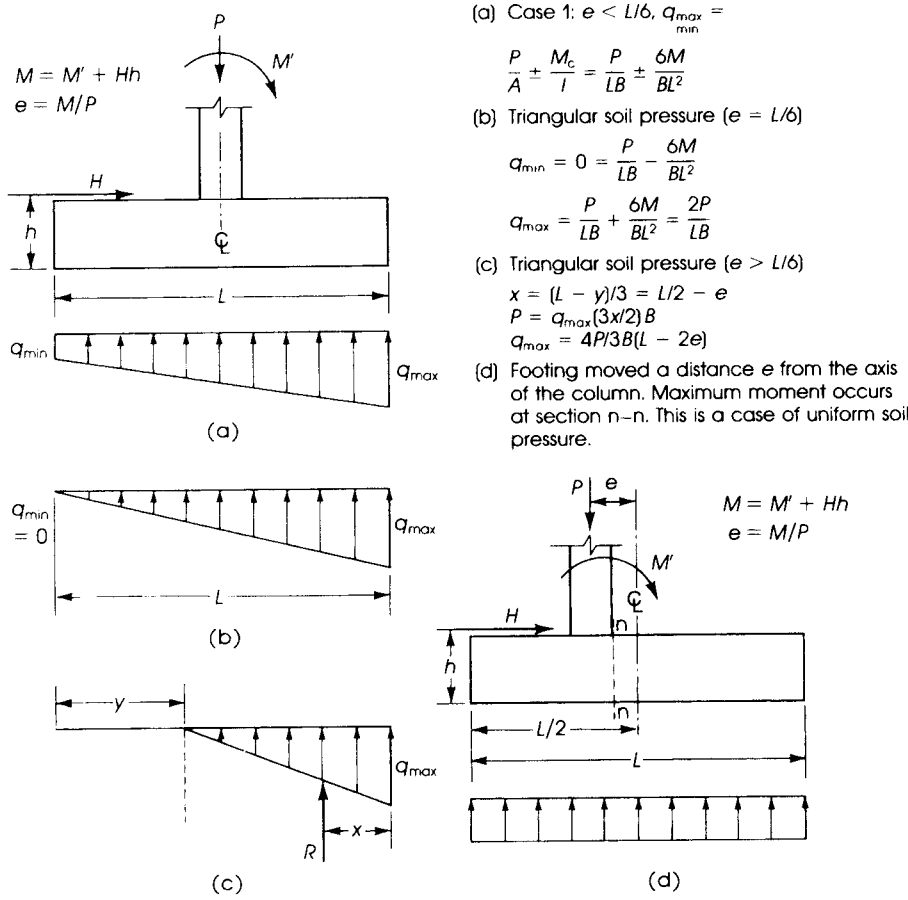
$$q = \frac{P}{A} \pm \frac{Mc}{I} \geq 0 \tag{13.19}$$

where A and I are the area and moment of inertia of the footing, respectively. Different soil conditions exist, depending on the magnitudes of P and M, and allowable soil pressure. The different design conditions are shown in Fig. 13.25 and are summarized as follows:

1. When $e = M/P < L/6$, the soil pressure is trapezoidal.

$$q_{\max} = \frac{P}{A} + \frac{Mc}{I} = \frac{P}{LB} + \frac{6M}{BL^2} \tag{13.20}$$

$$q_{\min} = \frac{P}{A} - \frac{Mc}{I} = \frac{P}{LB} - \frac{6M}{BL^2} \tag{13.21}$$



(a) Case 1: $e < L/6$, q_{max}, q_{min} =

$$\frac{P}{A} \pm \frac{M_c}{I} = \frac{P}{LB} \pm \frac{6M}{BL^2}$$

(b) Triangular soil pressure ($e = L/6$)

$$q_{min} = 0 = \frac{P}{LB} - \frac{6M}{BL^2}$$

$$q_{max} = \frac{P}{LB} + \frac{6M}{BL^2} = \frac{2P}{LB}$$

(c) Triangular soil pressure ($e > L/6$)

$x = (L - y)/3 = L/2 - e$
$P = q_{max}(3x/2)B$
$q_{max} = 4P/3B(L - 2e)$

(d) Footing moved a distance e from the axis of the column. Maximum moment occurs at section n–n. This is a case of uniform soil pressure.

Figure 13.25 Single footing subjected to eccentric loading: L = length of footing, B = width, and h = height.

2. When $e = M/P = L/6$, the soil pressure is triangular.

$$q_{max} = \frac{P}{LB} + \frac{6M}{BL^2} = \frac{2P}{LB} \tag{13.22}$$

$$q_{min} = 0 = \frac{P}{LB} - \frac{6M}{BL^2} \text{ or } \frac{P}{LB} = \frac{6M}{BL^2} \tag{13.23}$$

3. When $e > L/6$, the soil pressure is triangular.

$$x = \frac{L - y}{3} = \frac{L}{2} - e$$

$$P = q_{max}\left(\frac{3x}{2}\right)B \tag{13.24}$$

$$q_{max} = \frac{2P}{3xB} = \frac{4P}{3B(L - 2e)}$$

4. When the footing is moved a distance e from the axis of the column to produce uniform soil pressure under the footing. Maximum moment occurs at section n-n.

$$M = M' - Hh \quad \text{and} \quad e = \frac{M}{P}$$

13.8 FOOTINGS UNDER BIAXIAL MOMENT

In some cases, a footing may be subjected to an axial force and biaxial moments about its x- and y-axes; such a footing may be needed for a factory crane that rotates 360°. The footing then must be designed for the critical loading.

Referring to Fig. 13.26, if the axial load P acts at a distance e_x from the y-axis and e_y from the x-axis, then

$$M_x = Pe_y \quad \text{and} \quad M_y = Pe_x$$

The soil pressure at corner 1 is

$$q_{\max} = \frac{P}{A} + \frac{M_x c_y}{I_x} + \frac{M_y c_x}{I_y}$$

At corner 2,

$$q_2 = \frac{P}{A} - \frac{M_x c_y}{I_x} + \frac{M_y c_x}{I_y}$$

At corner 3,

$$q_3 = \frac{P}{A} - \frac{M_x c_y}{I_x} - \frac{M_y c_x}{I_y}$$

At corner 4,

$$q_4 = \frac{P}{A} + \frac{M_x c_y}{I_x} - \frac{M_y c_x}{I_y}$$

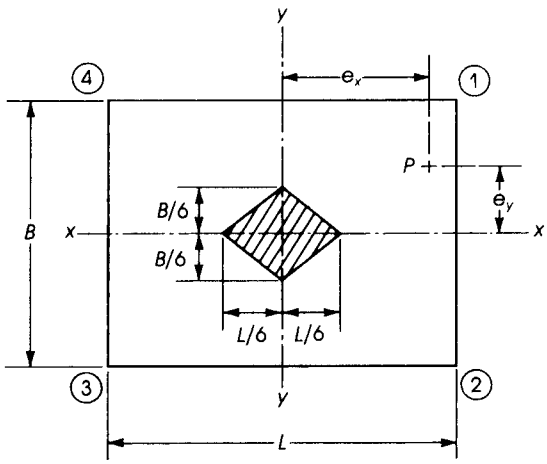


Figure 13.26 Footing subjected to P and biaxial moment. If $e_x < L/6$ and $e_y < B/6$, footing will be subjected to upward soil pressure on all bottom surface (nonuniform pressure).

Again, note that the allowable soil pressure must not be exceeded and the soil cannot take any tension; that is, $q \geq 0$.

Example 13.7

A 12-in. by 24-in. column of an unsymmetrical shed shown in Fig. 13.27*a* is subjected to an axial load $P_D = 220\,\text{K}$ and a moment $M_d = 180\,\text{K·ft}$ due to dead load and an axial load $P_L = 165\,\text{K}$ and a moment $M_L = 140\,\text{K·ft}$ due to live load. The base of the footing is 5 ft below final grade, and the allowable soil bearing pressure is 5 ksf. Design the footing using $f'_c = 4$ ksi and $f_y = 60$ ksi.

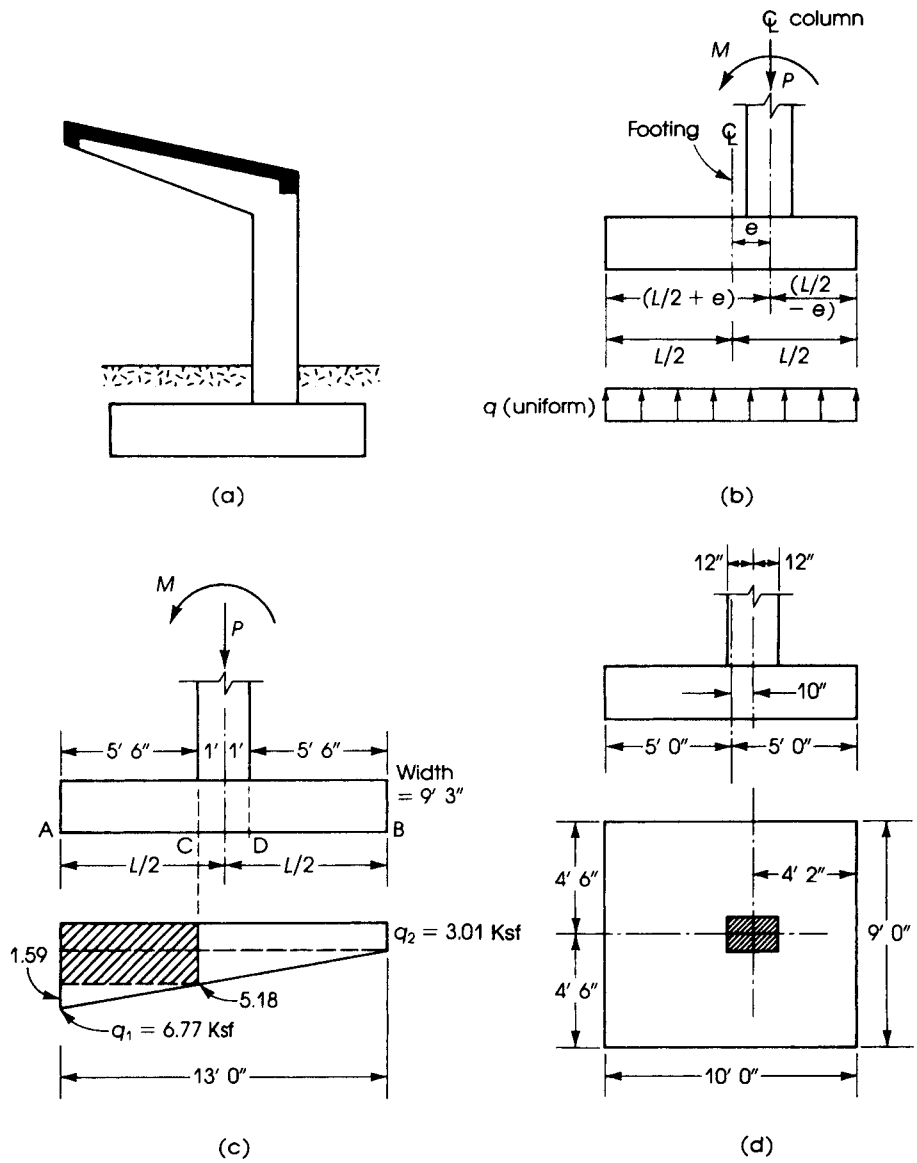


Figure 13.27 Example 13.7.

Solution

The footing is subjected to an axial load and a moment

$$P = 220 + 165 = 385 \text{ K}$$

$$M = 180 + 140 = 320 \text{ K·ft}$$

The eccentricity is

$$e = \frac{M}{P} = \frac{320 \times 12}{385} = 9.97 \text{ in.} \qquad \text{say, } 10 \text{ in.}$$

The footing may be designed by two methods.

Method 1: Move the center of the footing a distance $e = 10$ in. from the center of the column. In this case, the soil pressure will be considered uniformly distributed under the footing (Fig. 13.27*b*).

Method 2: The footing is placed concentric with the center of the column. In this case, the soil pressure will be trapezoidal or triangular (Fig. 13.27c), and the maximum and minimum values can be calculated as shown in Fig. 13.27.

The application of the two methods to Example 13.7 can be explained briefly as follows:

1. For the first method, assume a footing depth of 20 in. ($d = 16.5$ in.) and assume the weight of soil is 100 pcf. Net upward pressure is $5000 - \frac{20}{12} \times 150$ (footing) $- \left(5 - \frac{20}{12}\right) \times 100 = 4417$ psf.

$$\text{Area of footing} = \frac{385}{4.42} = 87.1 \text{ ft}^2$$

Assume a footing width of 9 ft; then the footing length is $87.1/9 = 9.7$ ft, say, 10 ft. Choose a footing 9 by 10 ft and place the column eccentrically, as shown in Fig. 13.27*d*. The center of the footing is 10 in. away from the center of the column.

2. The design procedure now is similar to that for a single footing. Check the depth for two-way and one-way shear action. Determine the bending moment at the face of the column for the longitudinal and transverse directions. Due to the eccentricity of the footing, the critical section will be on the left face of the column in Fig. 13.27*d*. The distance to the end of footing is $(5 \times 12) - 2 = 58$ in. $= 4.833$ ft.

$$P_u = 1.2D + 1.6L = 1.2 \times 200 + 1.6 \times 165 = 504 \text{ K}$$

$$q_u = \frac{504}{9 \times 10} = 5.6 \text{ ksf}$$

$$\text{Maximum } M_u = (5.6 \times 9) \times \frac{(4.833)^2}{2} = 588.6 \text{ K·ft} \quad \text{(in 9-ft width)}$$

In the transverse direction,

$$M_u = (5.6 \times 10) \times \frac{(4)^2}{2} = 448 \text{ K·ft}$$

Revise the assumed depth if needed and choose the required reinforcement in both directions of the footing, as was explained in the single-footing example.

3. For the second method, calculate the area of the footing in the same way as explained in the first method; then calculate the maximum soil pressure and compare it with that allowable using actual loads.

$$\text{Total load } P = 385 \text{ K}$$

$$\text{Size of footing} = 10 \times 9 \text{ ft}$$

Because the eccentricity is $e = 10$ in. $< L/6 = 10 \times \frac{12}{6} = 20$ in., the shape of the upward soil pressure is trapezoidal. Calculate the maximum and minimum soil pressure:

$$q_{\max} = \frac{P}{LB} + \frac{6M}{BL^2} = \frac{385}{10 \times 9} + \frac{6 \times 320}{9(10)^2} = 6.42 \text{ ksf} > 4.42 \text{ ksf}$$

The footing is not safe. Try a footing 9.25 × 13 ft (area = 120.25 ft^2).

$$q_{max} = \frac{385}{120.25} + \frac{6 \times 320}{9.25(13)^2} = 4.22 \text{ ksf} < 4.42 \text{ ksf}$$

$$q_{min} = 3.2 - 1.22 = 1.98 \text{ ksf}$$

4. Calculate the factored upward pressure using factored loads; then calculate moments and shears, as explained in previous examples.

13.9 SLABS ON GROUND

A concrete slab laid directly on ground may be subjected to

1. Uniform load over its surface, producing small internal forces.
2. Nonuniform or concentrated loads, producing some moments and shearing forces. Tensile stresses develop, and cracks will occur in some parts of the slab.

Tensile stresses are generally induced by a combination of

1. Contraction due to temperature and shrinkage, restricted by the friction between the slab and the subgrade, causing tensile stresses
2. Warping of the slab
3. Loading conditions
4. Settlement

Contraction joints may be formed to reduce the tensile stresses in the slab. Expansion joints may be provided in thin slabs up to a thickness of 10 in.

Basement floors in residential structures may be made of 4- to 6-in. concrete slabs reinforced in both directions with a wire fabric reinforcement. In warehouses, slabs may be 6 to 12 in. thick, depending on the loading on the slab. Reinforcement in both directions must be provided, usually in the form of wire fabric reinforcement. Basement floors are designed to resist upward earth pressure and any water pressure. If the slab rests on very stable or incompressible soils, then differential settlement is negligible. In this case the slab thickness will be a minimum if no water table exists. Columns in the basement will have independent footings. If there is any appreciable differential settlement, the floor slab must be designed as a stiff ra ft foundation.

13.10 FOOTINGS ON PILES

When the ground consists of so ft material for a great depth, and its bearing capacity is very low, it is not advisable to place the footings directly on the soil. It may be better to transmit the loads through piles to a deep stratum that is strong enough to bear the loads or to develop sufficient friction around the surface of the piles.

Many different kinds of piles are used for foundations. The choice depends on ground conditions, presence of ground water, function of the pile, and cost. Piles may be made of concrete, steel, or timber.

In general, a pile cap (or footing) is necessary to distribute the load from a column to the heads of a number of piles. The cap should be of sufficient size to accommodate deviation in the position of the pile heads. The caps are designed as beams spanning between the pile heads

and carrying concentrated loads from columns. When the column is supported by two piles, the cap may be designed as a reinforced concrete truss of a triangular shape.

The ACI Code, Section 15.2, indicates that computations for moments and shears for footings on piles may be based on the assumption that the reaction from any pile is concentrated at the pile center. The base area of the footing or number of piles shall be determined from the unfactored forces and moments.

The minimum concrete thickness above the reinforcement in a pile footing is limited to 12 in. (ACI Code, Section 15.7). For more design details of piles and pile footings, refer to books on foundation engineering.

13.11 SI EQUATIONS

1. One-way shear:

$$\phi V_c = 0.17\lambda\phi\sqrt{f'_c}bd \tag{13.3}$$

2. Two-way shear:

$$V_{c_1} = 0.33\lambda\sqrt{f'_c}b_0d \tag{13.6}$$

$$V_{c_2} = 0.17\left(1 + \frac{2}{\beta}\right)\lambda\sqrt{f'_c}b_0d \tag{13.7}$$

$$V_{c_3} = 0.083\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}b_0d \tag{13.8}$$

Other equations remain the same.

SUMMARY

Sections 13.1–13.4

1. *General:*
   H = distance of the bottom of footing from final grade (ft)
   h = total depth of footing (in.)
   c = wall thickness (in.)
   q_a = allowable soil pressure (ksf)
   q_e = effective soil pressure
   W_s = weight of soil (pcf) (Assume 100 pcf if not given)
2. *Design of wall footings:* The design steps can be summarized as follows.
   a. Assume a total depth of footing h (in.). Consider 1-ft length of footing.
   b. Calculate $q_e = q_a - (h/12)(150) - W_s(H - h/12)$ (q_a in psf).
   c. Calculate width of footing: B = (total service load)$/q_e = (P_D + P_n)/q_e$. (Round to the nearest higher half foot.) The footing size is $(B \times 1)$ ft.
   d. Calculate the factored upward pressure, $q_u = P_u/B$

$$P_u = 1.2P_D + 1.6P_L$$

e. Check the assumed depth for one-way shear requirements considering $d_a = (h - 3.5)$ in. (Two-way shear does not apply.)

$$V_u = q_u\left(\frac{B}{2} - d - \frac{c}{2}\right) \qquad \text{(Use kips.)}$$

$$\text{Required } d = \frac{V_u(1000)}{\phi(2\lambda\sqrt{f'_c})(12)} \geq d_a$$

f. Calculate the bending moment and main steel. The critical section is at the face of the wall.

- $M_u = 0.5q_u\ (L/2 - c/2)^2$; get $R_u = M_u/bd^2$.
- Determine ρ from tables in Appendix A or from Eq. 13.14.
- $A_s = \rho bd = 12\ \rho d$ in.^{2}/ft; $A_s \geq A_{s\,\min}$.
- Minimum steel for shrinkage is

$$A_{\text{sh}} = 0.0018\ (bh) \text{ for } f_y = 60 \text{ ksi}$$

$$A_{\text{sh}} = 0.0020\ (bh) \text{ for } f_y = 40, \text{ or } 50 \text{ ksi}$$

Minimum steel for flexure is

$$A_{\text{sf}} = \left(\frac{200}{f_y}\right)bd = \left(\frac{200}{f_y}\right)(12d) \qquad \text{when } f'_c < 4500 \text{ psi}$$

$$A_{\text{sf}} = \frac{(3\sqrt{f'_c})(12d)}{f_y} \qquad \text{when } f'_c > 4500 \text{ psi}$$

A_s calculated must be greater than A_{sh} (shrinkage). However, if $A_s < A_{\text{sf}}$, it is recommended to use $A_s = A_{\text{sf}}$ and then choose bars and spacings.

g. Check development length: Refer to Tables 7.1, 7.2, 7.3 and 7.4.

h. Calculate secondary reinforcement in the direction of the wall. $A_s = A_{\text{sh}}$ as calculated in step 6*d* using $b = 12$ in. Choose bars and spacings.

3. *Design of square/rectangular footings:* The design steps are as follows.

a. Assume a total depth h (in.); let d_a (assumed) $= (h - 4.5)$ in. Calculate $q_e = q_a - (h/12)(150) - W_s(H - h/12)$. (Use psf.)

b. Calculate the area of the footing, $AF = (P_D + P_L)/q_e$. Choose either a square footing, side $= \sqrt{AF}$, or a rectangular footing of length L and width B (short length); then round dimensions to the higher half ft.

c. Calculate $q_u = P_u/(LB)$.

d. Check footing depth due to two-way shear first. Maximum V_{u2} occurs at a section located at a distance equal to $d/2$ around the column.

1. Calculate $b_0 = 4(c + d)$ for square columns and $b_0 = 2\ (c_1 + d) + 2\ (c_2 + d)$ for rectangular columns.

$$V_{u_2} = P_u - q_u(c + d)^2 \qquad \text{for square columns}$$

$$V_{u_2} = P_u - q_u(c_1 + d)(c_2 + d) \qquad \text{for rectangular columns}$$

2. Calculate $d_1 = V_{u_2}/4\phi\lambda\sqrt{f'_c}b_0$ when $\beta = L/B \leq 2$.

$$d_1 = \frac{V_{u_2}}{\phi(2 + 4/\beta)\lambda\sqrt{f'_c}b_0} \qquad \text{when } \beta > 2$$

3. Calculate

$$d_2 = \frac{V_{u_2}}{\phi(\alpha_s d/b_0 + 2)\lambda\sqrt{f'_c}b_0}$$

Let d = the larger of d_1 and d_2. If d is less than d_a (assumed), increase d_a (or h) and repeat. The required d should be close to the assumed d_a (within 5% or 1 in. higher).

e. Check one-way shear (normally does not control in single footings):

1. $V_{u_{11}} = q_u B(L/2 - c/2 - d)$ in the long direction (or for square footings).

$$d_{11} = \frac{V_{u_{11}}}{2\phi\lambda\sqrt{f'_c}B}$$

2. $V_{u_{12}} = q_u L(B/2 - c/2 - d)$ in the short direction.

$$d_{12} = \frac{V_{u_{12}}}{2\phi\lambda\sqrt{f'_c}L} \qquad \text{(for rectangular footings)}$$

3. Let d be the larger of d_{11} and d_{12}; then use the larger d from steps 4 and 5.
4. Determine h = (d + 4.5) in.; round to the nearest higher inch.
5. Calculate the final $d = (h - 4.5)$ in.

f. Calculate the bending moment and the main steel in one direction only for square footings and two directions for rectangular footings.

1. In the long direction (or for square footings)

$$M_{uL} = 0.5q_u\left(\frac{L}{2} - \frac{c}{2}\right)^2 \qquad R_u = \frac{M_{uL}}{Bd^2}$$

2. In the short direction (for rectangular footings):

$$M_{us} = 0.5q_u\left(\frac{B}{2} - \frac{c}{2}\right)^2 \qquad R_{us} = \frac{M_{us}}{Ld^2}$$

3. Calculate the reinforcement in the long direction, A_{sL}, and in the short direction, A_{ss}, using Eq. 13.14.
4. Check that A_{sL} and A_{ss} are greater than the minimum steel reinforcement. Choose bars and spacings. For square footings, the same bars are used in both directions. Distribute bars in the bandwidth of rectangular columns according to Eq. 13.15.

g. Check bearing stress:

1. Calculate N_1 and N_2 : $N_1\,\phi(0.85f'_cA_1)$, where $\phi = 0.65$ and A_1 = area of column section; $N_2 = N_1\sqrt{A_2/A_1} \leq 2N_1$, where A_2 = square area of footing under column ($A_2 = B^2$).
2. If $P_u \leq N_1$ bearing stress is adequate. Minimum area of dowels is $0.005\ A_1$. Choose four bars to be placed at the four corners of column section.
3. If $P_u > N_1$, determine the excess load, $P_{ex} = (P_u - N_1)$, and then calculate A_{sd} (dowels) = P_{ex}/f_y. A_{sd} must be equal to or greater than $0.005A_1$. Choose at least four dowel bars.
4. Determine the development length in compression for dowels in the column and in the footing.

h. Calculate the development lengths, l_d, of the main bars in the footings. The calculated l_d must be greater than or equal to l_d provided in the footing. Provided l_d is $(L/2 - c/2 - 3)$ in. in the long direction and $l_d = (B/2 - c/2 - 3)$ in the short direction. Examples 13.2 and 13.3 explain these steps.

Section 13.5

Plain concrete may be used to support walls. The maximum flexural stress in tension should be calculated and must be less than the allowable stress of $5\phi\sqrt{f'_c}$.

Section 13.6

A combined footing is used when a column is located near a property line. Design of such footings is explained in Example 13.6.

Sections 13.7–13.9

Footings under eccentric column loads are explained in Example 13.7.

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1. R. W. Furlong. "Design Aids for Square Footings." *ACI Journal* 62 (March 1965).
2. R. Peck, W. Hanson, and T. Thornburn. *Foundation Engineering*, 2d ed. New York: John Wiley, 1974.
3. ACI Committee 436. "Suggested Design Procedures for Combined Footings and Mats." *ACI Journal* 63 (October 1966).
4. F. Kramrisch and P. Rogers. "Simplified Design of Combined Footings." *Journal of Soil Mechanics and Foundation Division, ASCE* 85 (October 1961).
5. L. S. Blake (ed). *Civil Engineer's Reference Book*, 3d ed. London: Butterworths, 1975.
6. American Concrete Institute. "Building Code Requirements for Structural Concrete." *ACI Code* 318–08. Detroit, Michigan, 2008.

PROBLEMS

For all problems in this chapter, use the following:

H_a = distance from bottom of footing to the final grade

h = depth of concrete footing

q_a = allowable soil pressure in ksf

Assume the weight of the soil is 100 pcf and $f_y = 60$ ksi.

13.1 For each problem in Table 13.1, design a wall footing to support the given reinforced concrete wall loads. Design for shear and moment; check the development length requirements. Also, determine the footing bars and their distribution. (Assume $d = h - 3.5$ in.)

13.2 For each problem in Table 13.2, design a square single footing to support the given square and round column loads. Design for moments, shear, load transfer, dowel length, and development lengths for footing main bars. Choose adequate bars and spacings. (Assume $d = h - 4.5$ in. for all problems.)

Table 13.1 Problem 13.1

Number	Wall Thickness (in.)	Dead Load (K/ft)	Live Load (K/ft)	f'_c (Ksi)	q_a (Ksf)	H (ft)	Part Answers L (ft)	Part Answers h (in.)
(a)	12	22	12	3	4	5	10	19
(b)	12	18	14	3	5	4	7.5	17
(c)	14	28	16	3	6	6	8.5	20
(d)	14	26	24	3	4	5	14.5	27
(e)	16	32	16	3	5	5	11	23
(f)	16	24	20	4	6	8	9	19
(g)	14	20	18	4	4	6	11.5	19
(h)	14	28	20	4	5	4	10.5	21
(i)	12	18	14	4	6	5	6	14
(j)	14	16	20	4	6	5	7	16

Table 13.2 Problem 13.2

Number	Column (in.)	Column bars	Dead Load (K)	Live Load (K)	f'_c (Ksi)	q_s (Ksf)	H (ft)	Part Answers L (ft)	Part Answers h (in.)
(a)	16 × 16	8 no. 8	150	115	3	5	6	8	20
(b)	18 × 18	8 no. 9	160	100	3	6	5	7	19
(c)	20 × 20	12 no. 9	245	159	3	6	7	9	23
(d)	12 × 12	8 no. 8	180	140	3	5	8	9	24
(e)	14 × 14	8 no. 9	140	160	4	5	6	8.5	21
(f)	16 × 16	8 no. 9	190	140	4	4	5	10	21
(g)	18 × 18	12 no. 8	200	120	4	6	7	8	20
(h)	20 × 20	12 no. 9	195	195	4	5	8	10	22
(i)	Dia. 20	8 no. 9	120	85	4	5	5	7	16
(j)	Dia. 16	8 no. 8	110	90	3	4	6	8	18

13.3 Repeat Problem 13.2*a*–*h* using rectangular footings with widths of 6, 6, 8, 8, 7, 8, 6, and 9 ft, respectively.

13.4 Repeat Problem 13.2*a*–*d* using rectangular columns of 14 × 20 in., 16 × 20 in., 16 × 24 in., and 12 × 18 in., respectively, and rectangular footings with the length equal to about 1.5 times the width.

13.5 Repeat Problem 13.1*a*–*d* using plain concrete wall footings and one-half the applied dead and live loads.

13.6 Design a rectangular combined footing to support the two columns shown in Fig. 13.28. The center of the exterior column is 1 ft away from the property line and 14 ft from the center of the interior column. The exterior column is square with 18-in. sides, is reinforced with no. 8 bars, and carries an axial dead load of 160 K and a live load of 140 K. The interior column is square with 20-in. sides, is reinforced with no. 9 bars, and carries an axial dead load of 240 K and a live load of 150 K. The bottom of the footing is 5 ft below final grade. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, and an allowable soil pressure of 5 ksf.

13.7 Determine the footing areas required for a balanced footing design (equal settlement approach) if the usual load is 25% for all footings. The allowable soil pressure is 5 ksi and the dead and live loads are given in Table 13.3.

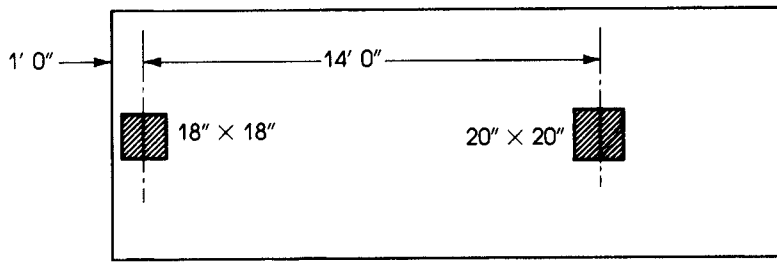


Figure 13.28 Problem 13.6.

Table 13.3 Problem 13.7

	Footing no.					
	1	**2**	**3**	**4**	**5**	**6**
Dead load	130 K	220 K	150 K	180 K	200 K	240 K
Live load	160 K	220 K	210 K	180 K	220 K	200 K

13.8 The 12- by 20-in., (300- by 500-mm) column of the frame shown in Fig. 13.29 is subjected to an axial load $P_D = 200$ K and a moment $M_D = 120$ K·ft due to dead load and an axial load $P_L = 160$ K and a moment $M_L = 110$ K·ft due to live load. The base of the footing is 4 ft below final grade. Design the footing using $f'_c = 4$ ksi, $f_y = 40$ ksi, and an allowable soil pressure of 4 ksi. Use a uniform pressure and eccentric footing approach.

13.9 Repeat Problem 13.8 if both the column and the footing have the same centerline (concentric case).

13.10 Determine the size of a square or round footing for the case of Problem 13.9, assuming that the loads and moments on the footing are for a rotating crane fixed at its base.

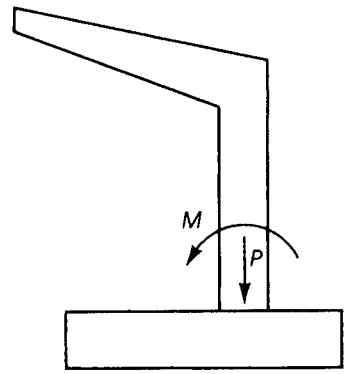


Figure 13.29 Problem 13.8.

CHAPTER 14

RETAINING WALLS



Apartment building, Miami, Florida.

14.1 INTRODUCTION

Retaining walls are structural members used to provide stability for soil or other materials and to prevent them from assuming their natural slope. In this sense, the retaining wall maintains unequal levels of earth on its two faces. The retained material on the higher level exerts a force on the retaining wall that may cause its overturning or failure. Retaining walls are used in bridges as abutments, in buildings as basement walls, and in embankments. They are also used to retain liquids, as in water tanks and sewage-treatment tanks.

14.2 TYPES OF RETAINING WALLS

Retaining walls may be classified as follows (refer to Fig. 14.1):

1. *Gravity walls* usually consist of plain concrete or masonry and depend entirely on their own weight to provide stability against the thrust of the retained material. These walls are proportioned so that tensile stresses do not develop in the concrete or masonry due to the exerted forces on the wall. The practical height of a gravity wall does not exceed 10 ft.
2. *Semigravity walls* are gravity walls that have a wider base to improve the stability of the wall and to prevent the development of tensile stresses in the base. Light reinforcement is sometimes used in the base or stem to reduce the large section of the wall.
3. The *cantilever retaining wall* is a reinforced concrete wall that is generally used for heights from 8 to 20 ft. It is the most common type of retaining structure because of economy and simplicity of construction. Various types of cantilever retaining walls are shown in Fig. 14.1.
4. *Counterfort retaining walls* higher than 20 ft develop a relatively large bending moment at the base of the stem, which makes the design of such walls uneconomical. One solution in this case is to introduce transverse walls (or counterforts) that tie the stem and the base together at intervals. The counterforts act as tension ties supporting the vertical walls. Economy is achieved because the stem is designed as a continuous slab spanning

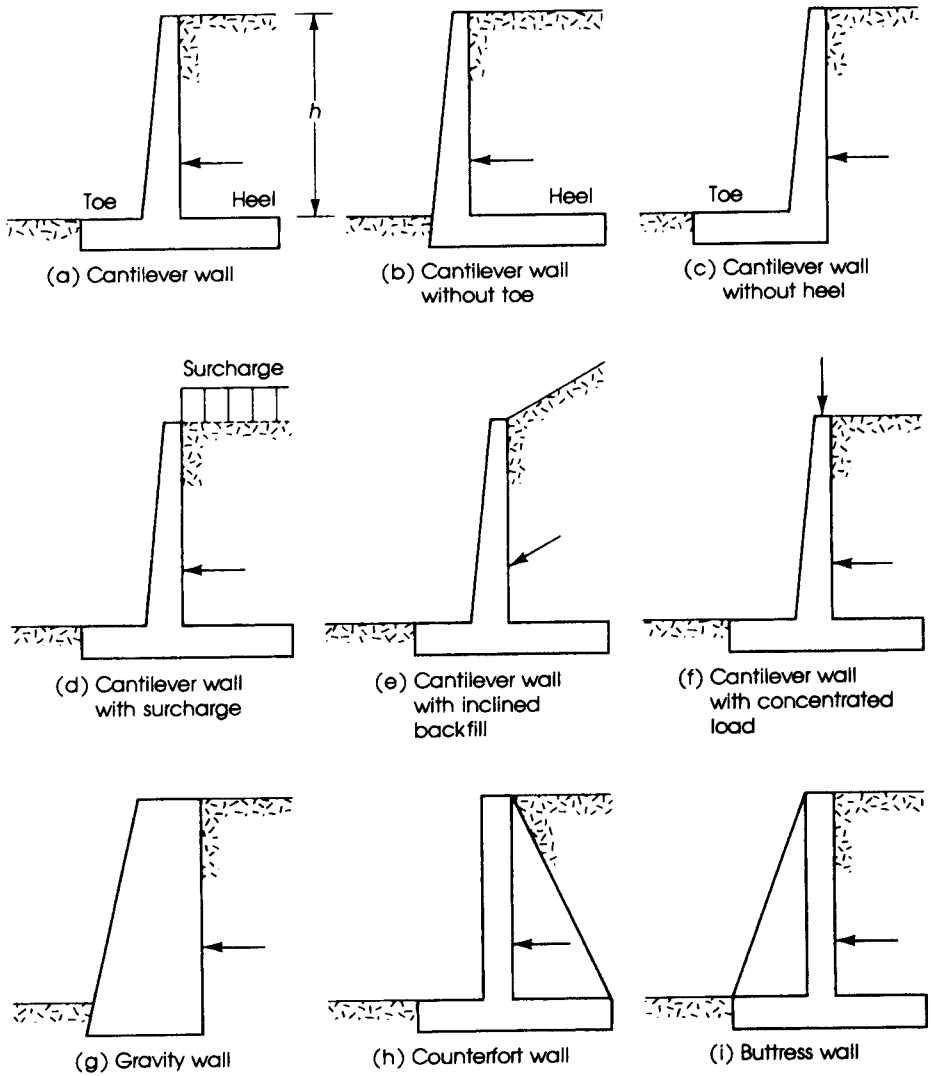


Figure 14.1 Types of retaining walls.

horizontally between counterforts, whereas the heel is designed as a slab supported on three sides (Fig. 14.1*h*).

5. The *buttressed retaining wall* is similar to the counterfort wall, but in this case the transverse walls are located on the opposite, visible side of the stem and act in compression (Fig 14.1*i*). The design of such walls becomes economical for heights greater than 20 ft. They are not popular because of the exposed buttresses.
6. *Bridge abutments* are retaining walls that are supported at the top by the bridge deck. The wall may be assumed fixed at the base and simply supported at the top.
7. *Basement walls* resist earth pressure from one side of the wall and span vertically from the basement-floor slab to the first-floor slab. The wall may be assumed fixed at the base and simply supported or partially restrained at the top.

14.3 FORCES ON RETAINING WALLS

Retaining walls are generally subjected to gravity loads and to earth pressure due to the retained material on the wall. Gravity loads due to the weights of the materials are well defined and can be calculated easily and directly. The magnitude and direction of the earth pressure on a retaining wall depends on the type and condition of soil retained and on other factors and cannot be determined as accurately as gravity loads. Several references on soil mechanics [1,2] explain the theories and procedure for determining the soil pressure on retaining walls. The stability of retaining walls and the effect of dynamic reaction on walls are discussed in Refs. 3 and 4.

Granular materials, such as sand, behave differently from cohesive materials, such as clay, or from any combination of both types of soils. Although the pressure intensity of soil on a retaining wall is complex, it is common to assume a linear pressure distribution on the wall. The pressure intensity increases with depth linearly, and its value is a function of the height of the wall and the weight and type of soil. The pressure intensity, p, at a depth h below the earth's surface may be calculated as follows:

$$p = Cwh \tag{14.1}$$

where w is the unit weight of soil and C is a coefficient that depends on the physical properties of soil. The value of the coefficient C varies from 0.3 for loose granular soil, such as sand, to about 1.0 for cohesive soil, such as wet clay. If the retaining wall is assumed absolutely rigid, a case of earth pressure at rest develops. Under soil pressure, the wall may deflect or move a small amount from the earth, and active soil pressure develops, as shown in Fig. 14.2. If the wall moves toward the soil, a passive soil pressure develops. Both the active and passive soil pressures are assumed to vary linearly with the depth of wall (Fig. 14.2). For dry, granular, noncohesive materials, the assumed linear pressure diagram is fairly satisfactory; cohesive soils or saturated sands behave in a different, nonlinear manner. Therefore, it is very common to use granular materials as backfill to provide an approximately linear pressure diagram and also to provide for the release or drainage of water from behind the wall.

For a linear pressure, the active and passive pressure intensities are determined as follows:

$$P_a = C_a wh \tag{14.2}$$

$$P_p = C_p wh \tag{14.3}$$

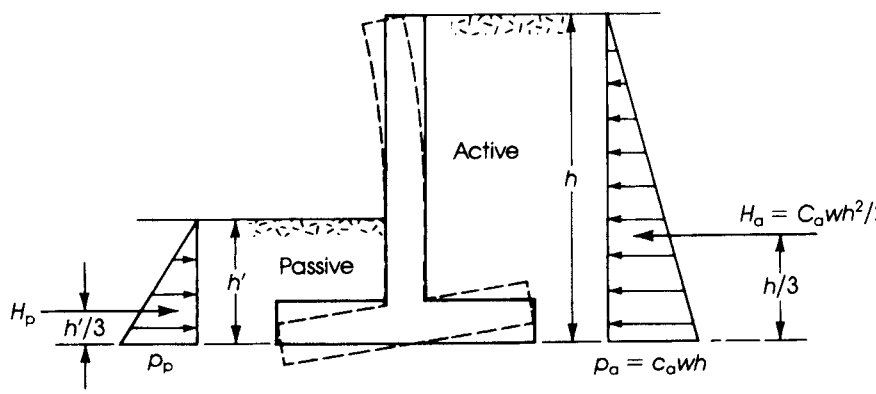


Figure 14.2 Active and passive earth pressure.

where C_a and C_p are the approximate coefficients of the active and passive pressures, respectively.

14.4 ACTIVE AND PASSIVE SOIL PRESSURES

The two theories most commonly used in the calculation of earth pressure are those of Rankine and Coulomb [1,6].

1. In Rankine's approach, the retaining wall is assumed to yield a sufficient amount to develop a state of plastic equilibrium in the soil mass at the wall surface. The rest of the soil remains in the state of elastic equilibrium. The theory applies mainly to a homogeneous, incompressible, cohesionless soil and neglects the friction between soil and wall. The active soil pressure at a depth h on a retaining wall with a horizontal backfill based on Rankine's theory is determined as follows:

$$P_a = C_a w h = w h \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) \tag{14.4}$$

where

$$C_a = \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$$

ϕ = angle of internal friction of the soil (Table 14.1)

$$\text{Total active pressure, } H_a = \frac{w h^2}{2}\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) \tag{14.5}$$

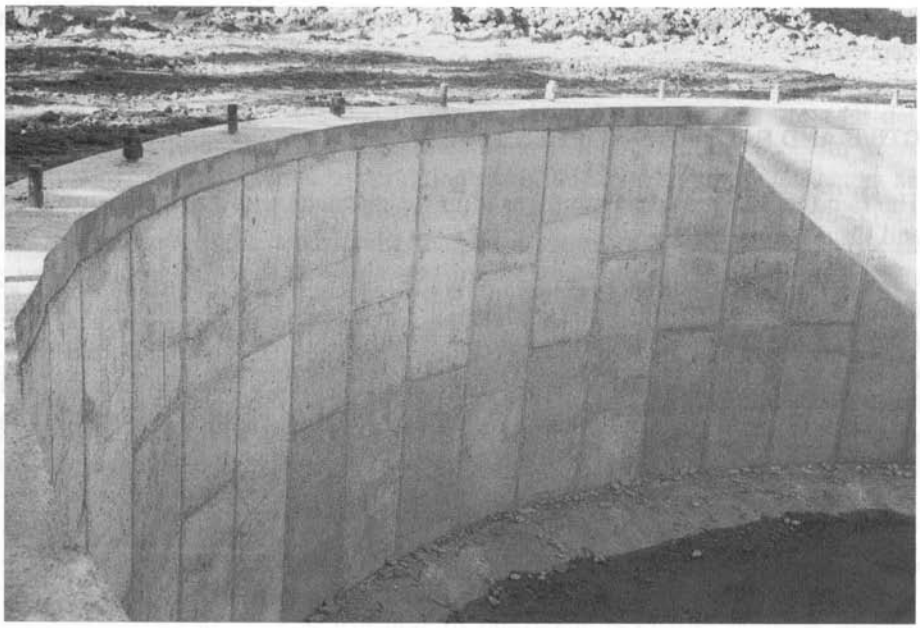
The resultant, H_a, acts at $h/3$ from the base (Fig. 14.2). When the earth is surcharged at an angle δ to the horizontal, then

$$C_a = \cos\delta \left(\frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}}\right) \tag{14.6}$$

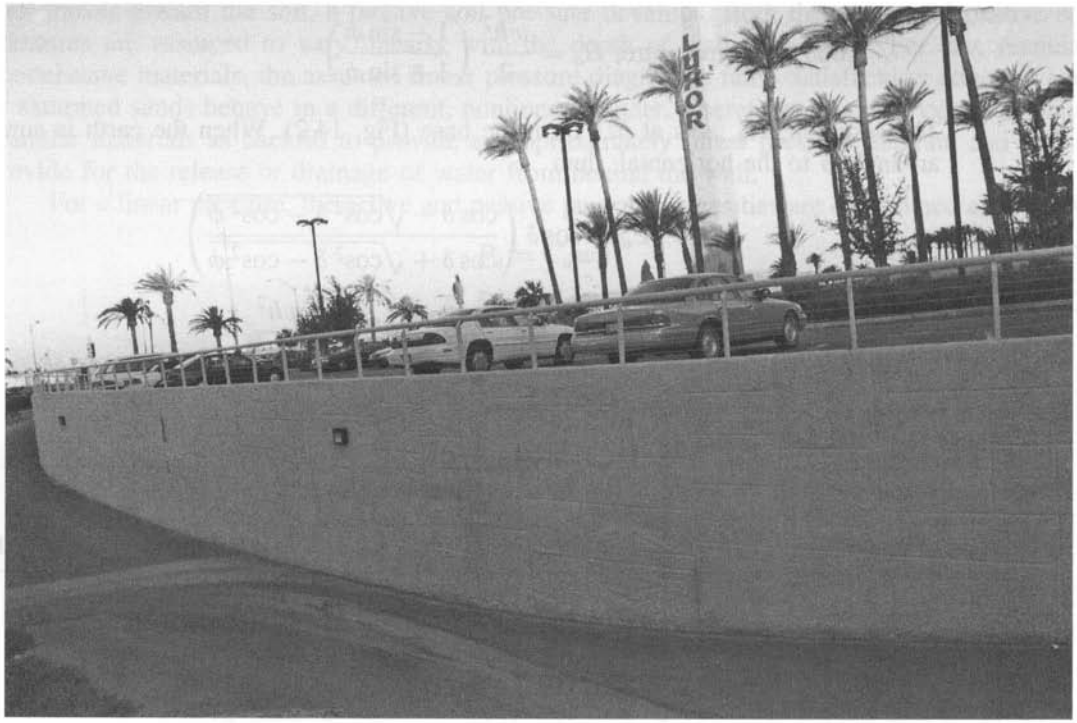
$$P_a = C_a w h \quad \text{and} \quad H_a = C_a \frac{w h^2}{2}$$

Table 14.1 Values of w and ϕ

Type of Backfill	Unit Weight, W		Angle of Internal Friction, ϕ
	pcf	*kg/m³*	
Soft clay	90–120	1440–1920	0°–15°
Medium clay	100–120	1600–1920	15°–30°
Dry loose silt	100–120	1600–1920	27°–30°
Dry dense silt	110–120	1760–1920	30°–35°
Loose sand and gravel	100–130	1600–2100	30°–40°
Dense sand and gravel	120–130	1920–2100	25°–35°
Dry loose sand, graded	115–130	1840–2100	33°–35°
Dry dense sand, graded	120–130	1920–2100	42°–46°



Reinforced concrete retaining wall.



Retaining wall in a parking area.

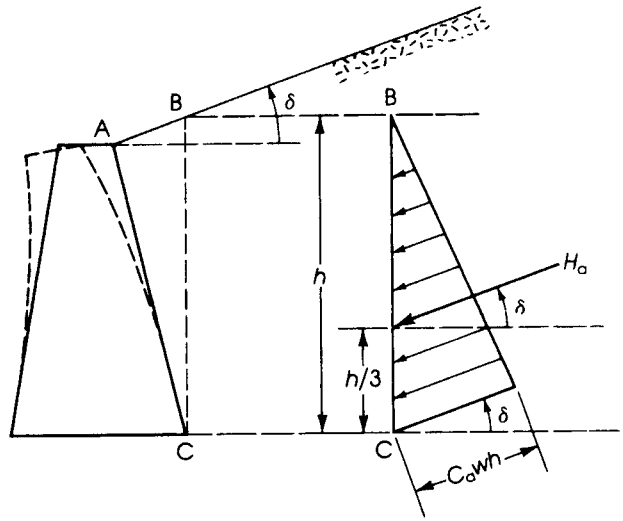


Figure 14.3 Active soil pressure with surcharge.

Table 14.2 Values of C_a

	ϕ (Angle of Internal Friction)						
Δ	28°	30°	32°	34°	36°	38°	40°
0°	0.361	0.333	0.307	0.283	0.260	0.238	0.217
10°	0.380	0.350	0.321	0.294	0.270	0.246	0.225
20°	0.461	0.414	0.374	0.338	0.306	0.277	0.250
25°	0.573	0.494	0.434	0.385	0.343	0.307	0.275
30°	0	0.866	0.574	0.478	0.411	0.358	0.315

The resultant, H_a, acts at $h/3$ and is inclined at an angle δ to the horizontal (Fig. 14.3). The values of C_a expressed by Eq. 14.6 for different values of δ and ϕ are shown in Table 14.2.

Passive soil pressure develops when the retaining wall moves against and compresses the soil. The passive soil pressure at a depth h on a retaining wall with horizontal backfill is determined as follows:

$$P_p = C_p wh = wh\left(\frac{1+\sin\phi}{1-\sin\phi}\right) \tag{14.7}$$

where

$$C_p = \left(\frac{1+\sin\phi}{1-\sin\phi}\right) = \frac{1}{C_a}$$

Total passive pressure is

$$H_p = \frac{wh^2}{2}\left(\frac{1+\sin\phi}{1-\sin\phi}\right) \tag{14.8}$$

The resultant, H_p, acts at $h'/3$ from the base (Fig. 14.2). When the earth is surcharged at an angle δ to the horizontal, then

$$C_p = \cos\delta \left(\frac{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}} \right) \tag{14.9}$$

$$P_p = C_p w h \quad \text{and} \quad H_p = C_p \frac{wh^2}{2}$$

H_p acts at $h'/3$ and is inclined at an angle δ to the horizontal (Fig. 14.4). The values of C_p expressed by Eq.14.9 for different values of δ and ϕ are shown in Table 14.3.
The values of ϕ and w vary with the type of backfill used. As a guide, common values of ϕ and w are given in Table 14.1.

2. In Coulomb's theory, the active soil pressure is assumed to be the result of the tendency of a wedge of soil to slide against the surface of a retaining wall. Hence, Coulomb's theory is referred to as the wedge theory. While it takes into consideration the friction of the soil on the retaining wall, it assumes that the surface of sliding is a plane, whereas in reality it is slightly curved. The error in this assumption is negligible in calculating the active soil pressure. Coulomb's equations to calculate the active and passive soil pressure are as follows:

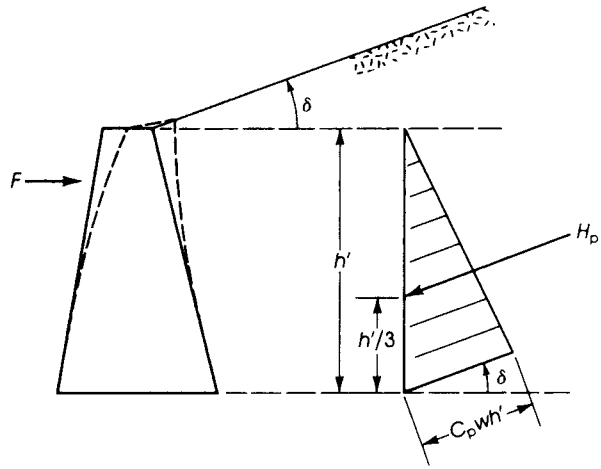


Figure 14.4 Passive soil pressure with surcharge.

Table 14.3 Values of C_p

	ϕ (Angle of Internal Friction)						
Δ	28°	30°	32°	34°	36°	38°	40°
0°	2.77	3.00	3.25	3.54	3.85	4.20	4.60
10°	2.55	2.78	3.02	3.30	3.60	3.94	4.32
20°	1.92	2.13	2.36	2.61	2.89	3.19	3.53
25°	1.43	1.66	1.90	2.14	2.40	2.68	3.00
30°	0	0.87	1.31	1.57	1.83	2.10	2.38

The active soil pressure is

$$P_a = C_a w h$$

where

$$C_a = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cos(\theta + \beta)\left[1 + \sqrt{\dfrac{\sin(\phi + \beta)\sin(\phi - \delta)}{\cos(\theta + \beta)\cos(\theta - \delta)}}\right]^2} \tag{14.10a}$$

where

ϕ = angle of internal friction of soil
θ = angle of the soil pressure surface from the vertical
β = angle of friction along the wall surface (angle between soil and concrete)
δ = angle of surcharge to the horizontal

The total active soil pressure is

$$H_a = C_a \frac{wh^2}{2} = p_a \frac{h}{2}$$

When the wall surface is vertical, $\theta = 0°$, and if $\beta = \delta$, then C_a in Eq. 14.10*a* reduces to Eq. 14.6 of Rankine.

Passive soil pressure is

$$P_p = C_p w h' \quad \text{and} \quad H_p = \left(\frac{wh'^2}{2}\right) C_p = P_p \frac{h'}{2}$$

where

$$C_p = \frac{\cos^2(\phi + \theta)}{\cos^2\theta \cos(\theta - \beta)\left[1 - \sqrt{\dfrac{\sin(\phi + \beta)\sin(\phi + \delta)}{\cos(\theta - \beta)\cos(\phi - \delta)}}\right]^2} \tag{14.10b}$$

The values of ϕ and w vary with the type of backfill used. As a guide, common values of ϕ and w are given in Table 14.1.

3. When the soil is saturated, the pores of the permeable soil are filled with water, which exerts hydrostatic pressure. In this case the buoyed unit weight of soil must be used. The buoyed unit weight (or submerged unit weight) is a reduced unit weight of soil and equals w minus the weight of water displaced by the soil. The effect of the hydrostatic water pressure must be included in the design of retaining walls subjected to a high water table and submerged soil. The value of the angle of internal friction may be used, as shown in Table 14.1.

14.5 EFFECT OF SURCHARGE

Different types of loads are often imposed on the surface of the backfill behind a retaining wall. If the load is uniform, an equivalent height of soil, h_s, may be assumed acting on the wall to account for the increased pressure. For the wall shown in Fig. 14.5, the horizontal pressure due to the surcharge is constant throughout the depth of the retaining wall.

$$h_s = \frac{w_s}{w} \tag{14.11}$$

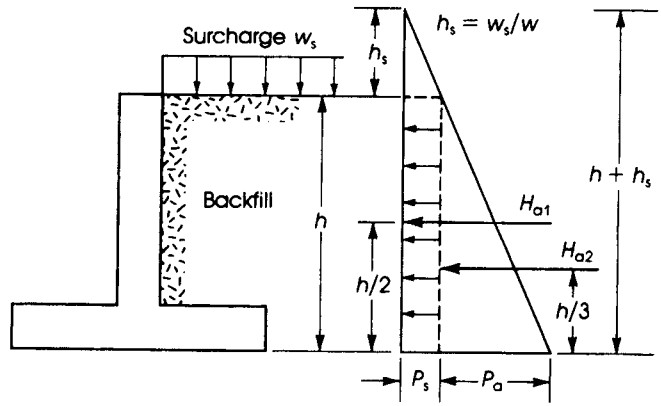


Figure 14.5 Surcharge effect under a uniform load.

where

h_s = equivalent height of soil (ft)
w_s = pressure of the surcharge (psf)
w = unit weight of soil (pcf)

The total pressure is

$$H_a = H_{a1} + H_{a2} = C_a w \left(\frac{h^2}{2} + h h_s \right) \tag{14.12}$$

In the case of a partial uniform load acting at a distance from the wall, only a portion of the total surcharge pressure affects the wall (Fig. 14.6).

It is a common practice to assume that the effective height of pressure due to partial surcharge is h', measured from point B to the base of the retaining wall [1]. The line AB forms an angle of 45° with the horizontal.

In the case of a wheel load acting at a distance from the wall, the load is to be distributed over a specific area, which is usually defined by known specifications such as AASHTO and AREA [4] specifications.

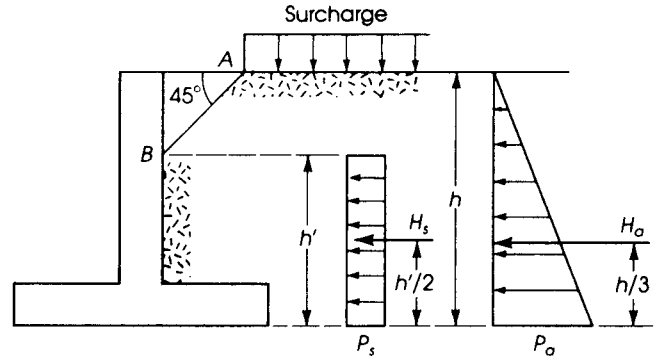


Figure 14.6 Surcharge effect under a partial uniform load at a distance from the wall.

14.6 FRICTION ON THE RETAINING WALL BASE

The horizontal component of all forces acting on a retaining wall tends to push the wall in a horizontal direction. The retaining wall base must be wide enough to resist the sliding of the wall. The coefficient of friction to be used is that of soil on concrete for coarse granular soils and the shear strength of cohesive soils [4]. The coefficients of friction μ that may be adopted for different types of soil are as follows:

- Coarse-grained soils without silt, $\mu = 0.55$
- Coarse-grained soils with silt, $\mu = 0.45$
- Silt, $\mu = 0.35$
- Sound rock, $\mu = 0.60$

The total frictional force, F, on the base to resist the sliding effect is

$$F = \mu R + H_p \tag{14.13}$$

where

μ = the coefficient of friction
R = the vertical force acting on the base
H_p = passive resisting force

The factor of safety against sliding is

$$\text{Factor of safety} = \frac{F}{H_{\text{ah}}} = \frac{\mu R + H_p}{H_{\text{ah}}} \geq 1.5 \tag{14.14}$$

where H_{ah} is the horizontal component of the active pressure, H_a. The factor of safety against sliding should not be less than 1.5 if the passive resistance H_p is neglected and should not be less than 2.0 if H_p is taken into consideration.

14.7 STABILITY AGAINST OVERTURNING

The horizontal component of the active pressure, H_a, tends to overturn the retaining wall about the point zero on the toe (Fig. 14.7). The overturning moment is equal to $M_0 = H_a h/3$. The weight of the concrete and soil tends to develop a balancing moment, or rightening moment, to resist the overturning moment. The balancing moment for the case of the wall shown in Fig. 14.7 is equal to

$$M_b = w_1 x_1 + w_2 x_2 + w_3 x_3 = \sum wx$$

The factor of safety against overturning is

$$\text{Factor of safety} = \frac{M_b}{M_o} = \frac{\sum wx}{\frac{H_a h}{3}} \geq 2.0 \tag{14.15}$$

This factor of safety should not be less than 2.0.

The resultant of all forces acting on the retaining wall, R_A, intersects the base at point C (Fig. 14.7). In general, point C does not coincide with the center of the base, L, thus causing eccentric loading on the footing. It is desirable to keep point C within the middle third to get the whole footing under soil pressure. (The case of a footing under eccentric load was discussed in Chapter 13.)

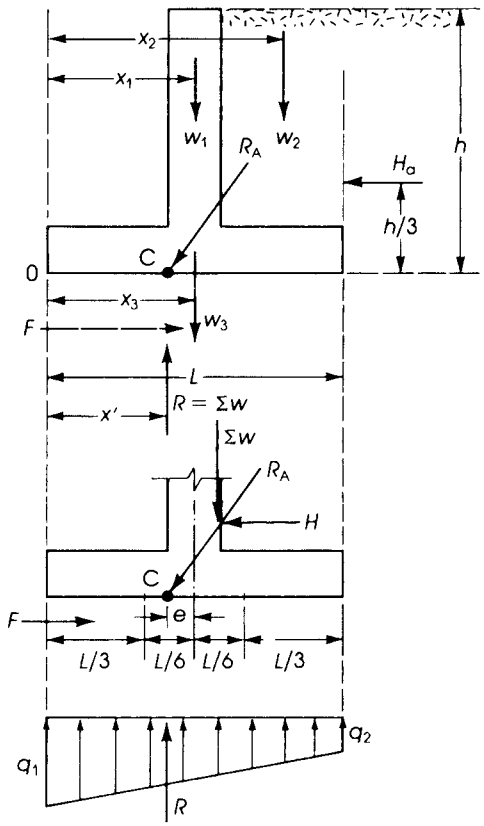


Figure 14.7 Overturning of a cantilever retaining wall.

14.8 PROPORTIONS OF RETAINING WALLS

The design of a retaining wall begins with a trial section and approximate dimensions. The assumed section is then checked for stability and structural adequacy. The following rules may be used to determine the approximate sizes of the different parts of a cantilever retaining wall.

1. *Height of the wall:* The overall height of the wall is equal to the difference in elevation required plus 3 to 4 ft, which is the estimated frost penetration depth in northern states.
2. *Thickness of the stem:* The intensity of the pressure increases with the depth of the stem and reaches its maximum value at the base level. Consequently the maximum bending moment and shear in the stem occur at its base. The stem base thickness may be estimated as $\frac{1}{12}$ to $\frac{1}{10}$ of the height, h. The thickness at the top of the stem may be assumed to be 8 to 12 in. Because retaining walls are designed for active earth pressure, causing a small deflection of the wall, it is advisable to provide the face of the wall with a batter (taper) of $\frac{1}{4}$ in. per foot of height, h, to compensate for the forward deflection. For short walls up to 10 ft high, a constant thickness may be adopted.
3. *Length of the base:* An initial estimate for the length of the base of $\frac{2}{5}$ to $\frac{2}{3}$ of the wall height, h, may be adopted.

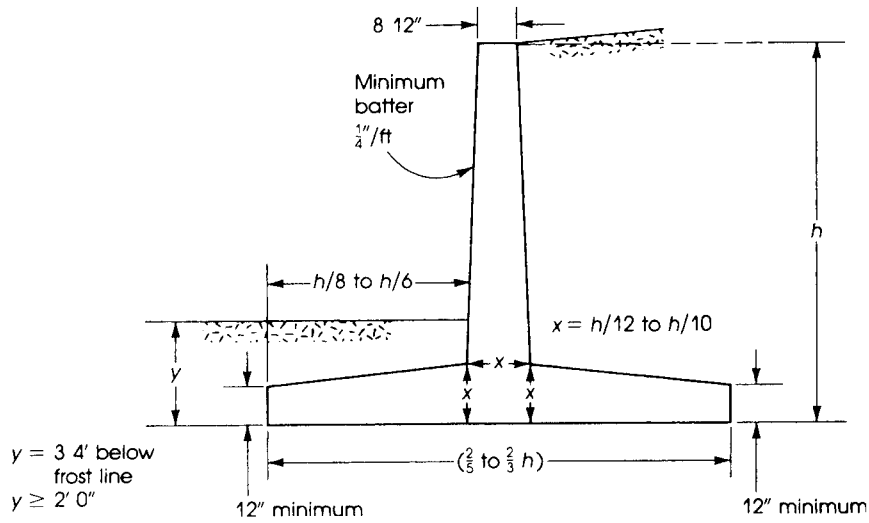


Figure 14.8 Trial proportions of a cantilever retaining wall.

4. *Thickness of the base:* The base thickness below the stem is estimated as the same thickness of the stem at its base, that is, $\frac{1}{12}$ to $\frac{1}{10}$ of the wall height. A minimum thickness of about 12 in. is recommended. The wall base may be of uniform thickness or tapered to the ends of the toe and heel, where the bending moment is 0.

The approximate initial proportions of a cantilever retaining wall are shown in Fig. 14.8.

14.9 DESIGN REQUIREMENTS

The ACI Code, Chapter 14, provides methods for bearing wall design. The main requirements are as follows:

1. The minimum thickness of bearing walls is $\frac{1}{25}$ the supported height or length, whichever is shorter, but not less than 4 in.
2. The minimum area of the horizontal reinforcement in the wall is $0.0025bh$, where bh is the gross concrete wall area. This value may be reduced to $0.0020bh$ if no. 5 or smaller deformed bars with $f_y \geq 60$ ksi are used. For welded wire fabric (plain or deformed), the minimum steel area is $0.0020bh$.
3. The minimum area of the vertical reinforcement is $0.0015bh$, but it may be reduced to $0.0012bh$ if no. 5 or smaller deformed bars with $f_y \geq 60$ ksi are used. For welded wire fabric (plain or deformed), the minimum steel area is $0.0012bh$.
4. The maximum spacing of the vertical or the horizontal reinforcing bars is the smaller of 18 in. or three times the wall thickness.
5. If the wall thickness exceeds 10 in., the vertical and horizontal reinforcement should be placed in two layers parallel to the exterior and interior wall surfaces, as follows:

   For exterior wall surfaces, at least $\frac{1}{2}$ of the reinforcement A_s (but not more than $\frac{2}{3}A_s$) should have a minimum concrete cover of 2 in. but not more than $\frac{1}{3}$ of the wall thickness.

This is because the exterior surface of the wall is normally exposed to different weather conditions and temperature changes.

For interior wall surfaces, the balance of the required reinforcement in each direction should have a minimum concrete cover of $\frac{3}{4}$ in. but not more than $\frac{1}{3}$ of the wall thickness.

The minimum steel area in the wall footing (heel or toe), according to the ACI Code, Section 10.5.3, is that required for shrinkage and temperature reinforcement, which is $0.0018bh$ when $f_y = 60$ ksi and $0.0020bh$ when $f_y = 40$ ksi or 50 ksi. Because this minimum steel area is relatively small, it is a common practice to increase it to that minimum A_s required for flexure:

$$A_{s\,\min} = \left(\frac{3\sqrt{f'_c}}{f_y}\right) bd \geq \left(\frac{200}{f_y}\right) bd \tag{14.16}$$

14.10 DRAINAGE

The earth pressure discussed in the previous sections does not include any hydrostatic pressure. If water accumulates behind the retaining wall, the water pressure must be included in the design. Surface or underground water may seep into the backfill and develop the case of submerged soil. To avoid hydrostatic pressure, drainage should be provided behind the wall. If well-drained cohesionless soil is used as a backfill, the wall can be designed for earth pressure only. The drainage system may consist of one or a combination of the following:

1. Weep holes in the retaining wall of 4 in. or more in diameter and spaced about 5 ft on centers horizontally and vertically (Fig. 14.9*a*).
2. Perforated pipe 8 in. in diameter laid along the base of the wall and surrounded by gravel (Fig. 14.9*b*).
3. Blanketing or paving the surface of the backfill with asphalt to prevent seepage of water from the surface.
4. Any other method to drain surface water.

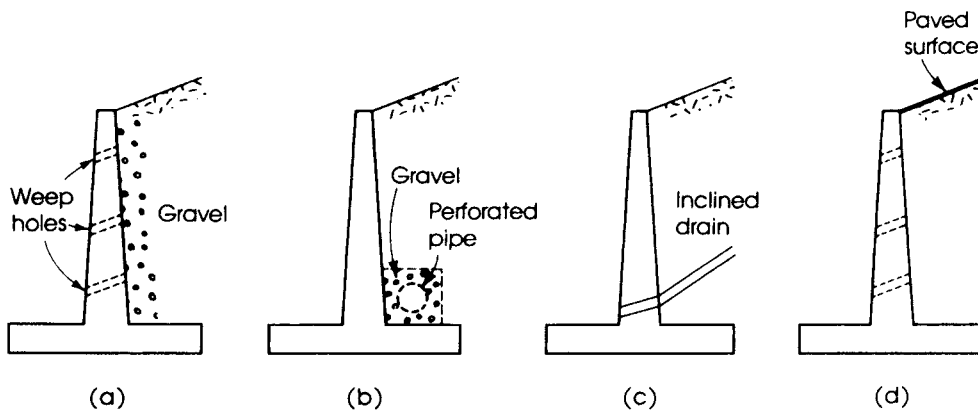


Figure 14.9 Drainage systems.

Example 14.1

The trial section of a semigravity plain concrete retaining wall is shown in Fig. 14.10. It is required to check the safety of the wall against overturning, sliding, and bearing pressure under the footing. Given: Weight of backfill is 110 pcf, angle of internal friction is $\phi = 35°$, coefficient of friction between concrete and soil is $\mu = 0.5$, allowable soil pressure is 2.5 ksf, and $f'_c = 3$ ksi.

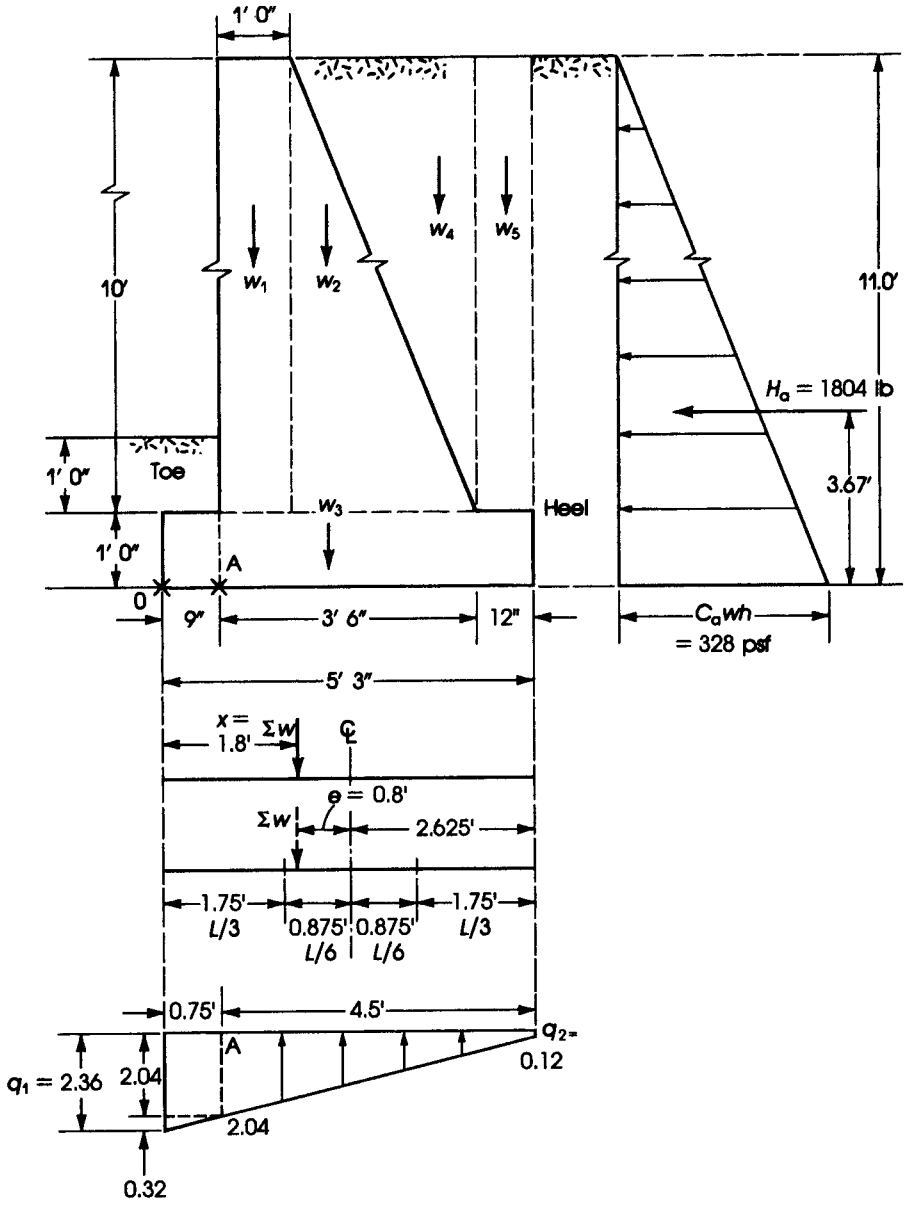


Figure 14.10 Example 14.1.

Solution

1. Using the Rankine equation,

$$C_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-0.574}{1+0.574} = 0.271$$

The passive pressure on the toe is that for a height of 1 ft, which is small and can be neglected.

$$H_a = \frac{C_a w h^2}{2} = \frac{0.271}{2}(110)(11)^2 = 1804 \text{ lb}$$

H_a acts at a distance $h/3 = \frac{11}{3} = 3.67$ ft from the bottom of the base.

2. The overturning moment is $M_0 = 1.804 \times 3.67 = 6.62$ K·ft.
3. Calculate the balancing moment, M_b, taken about the toe end 0 (Fig. 14.10):

Weight (lb)	Arm (ft)	Moment (K·ft)
$w_1 = 1 \times 10 \times 145 = 1450$	1.25	1.81
$w_2 = \frac{1}{2} \times 2.5 \times 10 \times 145 = 1812$	2.60	4.71
$w_3 = 5.25 \times 1 \times 145 = 725$	2.625	2.00
$w_4 = \frac{1}{2} \times 2.5 \times 10 \times 110 = 1375$	3.42	4.70
$w_5 = \frac{12}{12} \times 10 \times 110 = 1100$	4.75	5.22

$$\sum w = R = 6.50 \text{ K} \qquad M_b = \sum M = 18.44 \text{ K·ft}$$

4. The factor of safety against overturning is $18.44/6.50 = 2.82 > 2.0$.
5. The force resisting sliding, $F = \mu R$, is $F = 0.5(6.50) = 3.25$ K. The factor of safety against sliding is $F/H_a = 3.25/1.804 = 1.8 > 1.5$.
6. Calculate the soil pressure under the base:
   a. The distance of the resultant from toe end 0 is

$$x = \frac{M_b - M_0}{R} = \frac{18.44 - 6.62}{6.50} = 1.82 \text{ ft}$$

   The eccentricity is $e = 2.62 - 1.82 = 0.80$ ft. The resultant R acts just inside the middle third of the base and has an eccentricity of $e = 0.8$ ft from the center of the base (Fig. 14.10). For a 1-ft length of the footing, the effective length of footing is 5.25 ft.
   b. The moment of inertia is $I = 1.0(5.25)^3/12 = 12.1 \text{ ft}^4$. Area $= 5.25 \text{ ft}^2$.
   c. The soil pressures at the two extreme ends of the footing are q_1, $q_2 = R/A \pm Mc/I$. The moment M is $Re = 6.50(0.8) = 5.2$ K·ft; $c = 2.62$ ft.

$$q_1 = \frac{6.50}{5.25} + \frac{5.2(2.62)}{12.1} = 1.24 + 1.12 = 2.36 \text{ ksf}$$

$$q_2 = 1.24 - 1.12 = 0.12 \text{ ksf}$$

7. Check the bending stress in concrete at point A of the toe.
   a. Soil pressure at A (from geometry) is

$$q_A = 0.12 + \left(\frac{4.5}{5.25}\right)(2.36 - 0.12) = 2.04 \text{ ksf}$$

   b. M_A is calculated at A due to a rectangular stress and a triangular stress.

$$M_A = \frac{2.04(0.75)^2}{2} + (0.32 \times 0.75 \times 0.5)\left(0.75 \times \frac{2}{3}\right)$$

$$= 0.63 \text{ K·ft}$$

c. The flexural stress in concrete is

$$Mc/I = 0.63(12{,}000)(6)/1728 = 26 \text{ psi}$$

where $c = h/2 = 12/2 = 6$ in. and $I = 12(12)^3/12 = 1728$ in.^{4}

d. The modulus of rupture of concrete is $7.5\lambda\sqrt{f'_c} = 410$ psi > 26 psi. The factor of safety against cracking is $410/26 = 16$. Therefore, the section is adequate. No other sections need to be checked.

Example 14.2

Design a cantilever retaining wall to support a bank of earth 16.5 ft high. The top of the earth is to be level with a surcharge of 330 psf. Given: The weight of the backfill is 110 pcf, the angle of internal friction is $\phi = 35°$, the coefficient of friction between concrete and soil is $\mu = 0.5$, the coefficient of friction between soil layers is $\mu = 0.7$, allowable soil bearing capacity is 4 ksf, $f'_c = 3$ ksi, and $f_y = 60$ ksi.

Solution

1. Determine the dimensions of the retaining wall using the approximate relationships shown in Fig. 14.8.
   a. Height of wall: Allowing 3 ft for frost penetration, the height of the wall becomes $h = 16.5 + 3 = 19.5$ ft.
   b. Base thickness: Assume base thickness is $0.08h = 0.08 \times 19.5 = 1.56$ ft, or 1.5 ft. The height of the stem is $19.5 - 1.5 = 18$ ft.
   c. Base length: The base length varies between $0.4h$ and $0.67h$. Assuming an average value of $0.53h$, then the base length equals $0.53 \times 19.5 = 10.3$ ft, say, 10.5 ft. The projection of the base in front of the stem varies between $0.17h$ and $0.125h$. Assume a projection of $0.17h = 0.17 \times 19.5 = 3.3$ ft, say, 3.5 ft.
   d. Stem thickness: The maximum stem thickness is at the bottom of the wall and varies between $0.08h$ and $0.1h$. Choose a maximum stem thickness equal to that of the base, or 1.5 ft. Select a practical minimum thickness of the stem at the top of the wall of 1.0 ft. The minimum batter of the face of the wall is $\frac{1}{4}$ in./ft. For an 18-ft-high wall, the minimum batter is $\frac{3}{4} \times 18 = 4.5$in., which is less than the $1.5 - 1.0 = 0.5$ ft (6 in.) provided. The trial dimensions of the wall are shown in Fig. 14.11.
2. Using the Rankine equation:

$$C_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - 0.574}{1 + 0.574} = 0.271$$

3. The factor of safety against overturning can be determined as follows:
   a. Calculate the actual unfactored forces acting on the retaining wall. First, find those acting to overturn the wall:

$$h_s(\text{due to surcharge}) = \frac{w_s}{w} = \frac{330}{110} = 3 \text{ ft}$$

$$p_1 = C_a w h_s = 0.271 \times (110 \times 3) = 90 \text{ psf}$$

$$p_2 = C_a w h = 0.271 \times (110 \times 19.5) = 581 \text{ psf}$$

$$H_{a_1} = 90 \times 19.5 = 1755 \text{ lb} \qquad \text{arm} = \frac{19.5}{2} = 9.75 \text{ ft}$$

$$H_{a_2} = \frac{1}{2} \times 581 \times 19.5 = 5665 \text{ lb} \qquad \text{arm} = \frac{19.5}{3} = 6.5 \text{ ft}$$

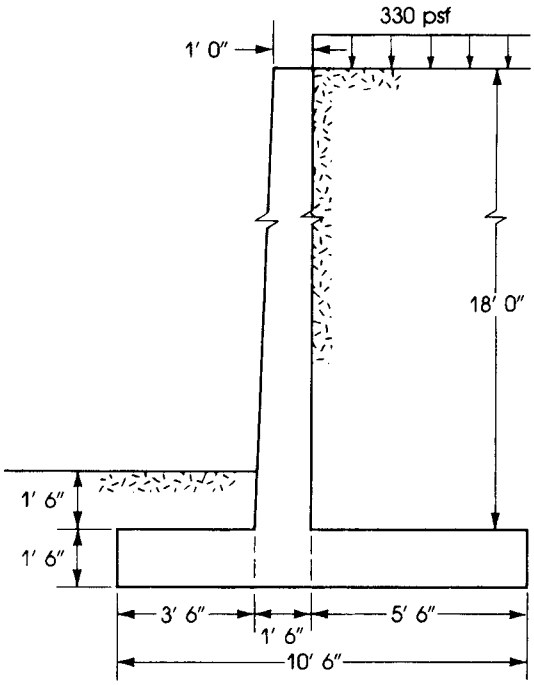


Figure 14.11 Example 14.2: trial configuration of retaining wall.

b. The overturning moment is $1.755 \times 9.75 + 5.665 \times 6.5 = 53.93$ K·ft.

c. Calculate the balancing moment against overturning (see Fig. 14.12):

Force (lb)	Arm (ft)	Moment (K·ft)
$w_1 = 1 \times 18 \times 150 = 2{,}700$	4.50	12.15
$w_2 = \frac{1}{2} \times 18 \times \frac{1}{2} \times 150 = 675$	3.83	2.59
$w_3 = 10.5 \times 1.5 \times 150 = 2{,}363$	5.25	12.41
$w_4 = 5.5 \times 21 \times 110 = 12{,}705$	7.75	98.46

$$\sum w = R = 18.44 \text{ K} \qquad \sum M = 125.61 \text{ K·ft}$$

$$\text{Factor of safety against overturning} = \frac{125.61}{53.93} = 2.33 > 2.0$$

4. Calculate the base soil pressure. Take moments about the toe end 0 (Fig. 14.12) to determine the location of the resultant R of the vertical forces.

$$x = \frac{\sum M - \sum Hy}{R} = \frac{\text{balancing } M - \text{overturning } M}{R}$$

$$= \frac{125.61 - 53.93}{18.44} = 3.89 \text{ ft} > \frac{10.5}{3} \quad \text{or} \quad 3.5 \text{ ft}$$

The eccentricity is $e = 10.5/2 - 3.89 = 1.36$ ft. The resultant R acts within the middle third of the base and has an eccentricity of $e = 1.36$ ft from the center of the base. For a 1-ft length

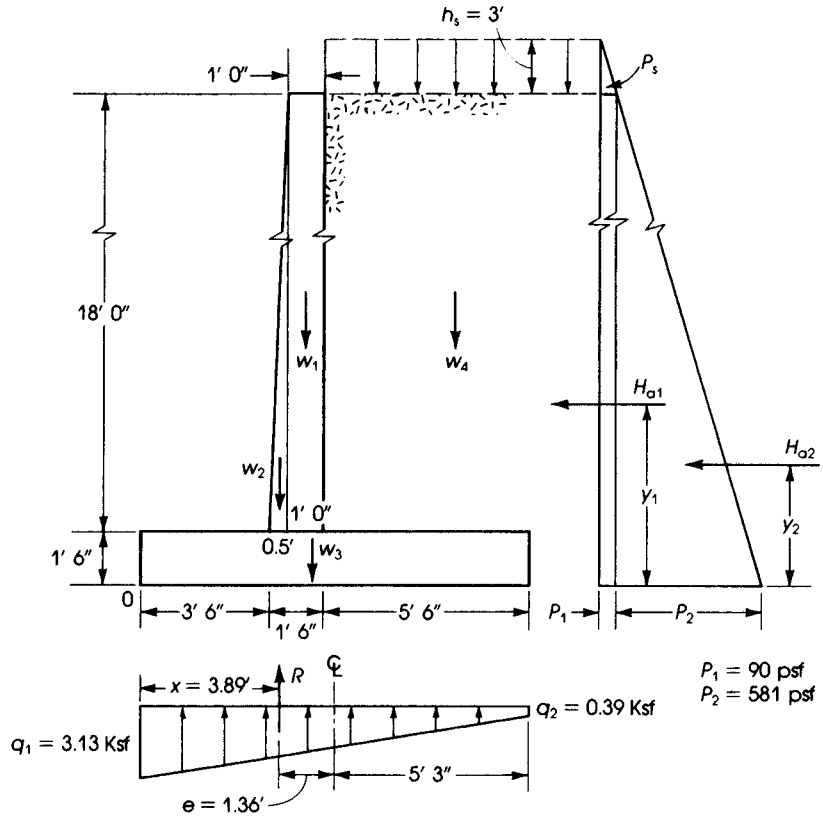


Figure 14.12 Example 14.2: forces acting on retaining wall.

of the footing, area $= 10.5 \times 1 = 10.5\,\text{ft}^2$.

$$I = 1 \times \frac{(10.5)^3}{12} = 96.47\ \text{ft}^4$$

$$q_1 = \frac{R}{A} + \frac{(Re)C}{I} = \frac{18.44}{10.5} + \frac{(18.44 \times 1.36) \times 5.25}{96.47}$$

$$= 1.76 + 1.37 = 3.13\ \text{ksf} < 4\ \text{ksf}$$

$$q_2 = 1.76 - 1.37 = 0.39\ \text{ksf}$$

Soil pressure is adequate.

5. Calculate the factor of safety against sliding. A minimum factor of safety of 1.5 must be maintained.

$$\text{Force causing sliding} = H_{a1} + H_{a2} = 1.76 + 5.67 = 7.43\ \text{K}$$

$$\text{Resisting force} = \mu R = 0.5 \times 18.44 = 9.22\ \text{K}$$

$$\text{Factor of safety against sliding} = \frac{9.22}{7.43} = 1.24 < 1.5$$

The resistance provided does not give an adequate safety against sliding. In this case, a key should be provided to develop a passive pressure large enough to resist the excess force

that causes sliding. Another function of the key is to provide sufficient development length for the dowels of the stem. The key is therefore placed such that its face is about 6 in. from the back face of the stem (Fig. 14.13). In the calculation of the passive pressure, the top foot of the earth at the toe side is usually neglected, leaving a height of 2 ft in this example. Assume a key depth of $t = 1.5$ ft and a width of $b = 1.5$ ft.

$$C_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1}{C_a} = \frac{1}{0.271} = 3.69$$

$$H_p = \frac{1}{2}C_p w(h'+t)^2 = \frac{1}{2} \times 3.69 \times 110(2+1.5)^2 = 2486 \text{ lb}$$

The sliding may occur now on the surfaces AC, CD, and EF (Fig. 14.13). The sliding surface AC lies within the soil layers with a coefficient of internal friction $= \tan\phi = \tan 35^\circ = 0.7$, whereas the surfaces CD and EF are those between concrete and soil with a coefficient of internal friction of 0.5, as given in this example. The frictional resistance is $F = \mu_1 R_1 + \mu_2 R_2$.

$$R_1 = \text{reaction of } AC = \left(\frac{3.13+1.96}{2}\right) \times 4.5 = 11.44 \text{ K}$$

$$R_2 = R - R_1 = 18.44 - 11.44 = 7.0 \text{ K}$$

$$R_2 = \text{reaction of } CDF = \left(\frac{1.96+0.39}{2}\right) \times 6 = 7.05 \text{ K}$$

$$F = 0.7(11.44) + 0.5(7.00) = 11.50 \text{ K}$$

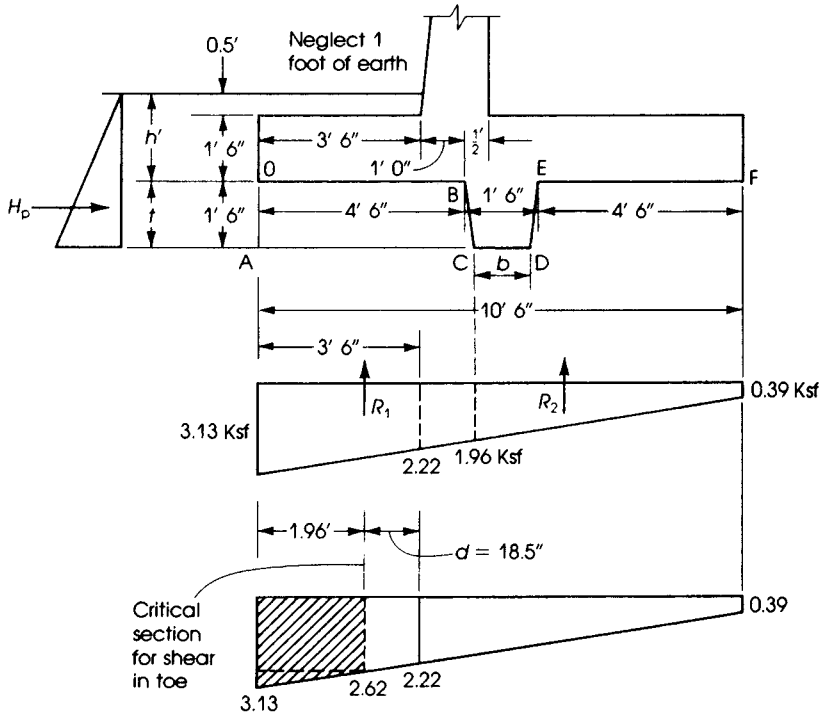


Figure 14.13 Example 14.2: footing details.

The total resisting force is

$$F + H_p = 11.50 + 2.49 = 13.99 \text{ K}$$

The factor of safety against sliding is

$$\frac{13.99}{7.43} = 1.9 \quad \text{or} \quad \frac{11.5}{7.43} = 1.55 > 1.5$$

The factor is greater than 1.5, which is recommended when passive resistance against sliding is not included.

6. Design the wall (stem). The design of the different reinforced concrete structural elements can be performed now using the ACI Code alternative design method, Appendix B and C.
   a. Main reinforcement: The lateral forces applied to the wall are calculated using a load factor of 1.6. The critical section for bending moment is at the bottom of the wall, height = 18 ft. Calculate the applied ultimate forces:

$$P_1 = 1.6(C_a w h_s) = 1.6(0.271 \times 110 \times 3) = 143 \text{ lb}$$

$$P_2 = 1.6(C_a w h) = 1.6(0.271 \times 110 \times 18) = 858.3 \text{ lb}$$

$$H_{a1} = 0.143 \times 18 = 2.57 \text{ K} \quad \text{arm} = \tfrac{18}{2} = 9 \text{ ft}$$

$$H_{a2} = \tfrac{1}{2} \times 0.858 \times 18 = 7.72 \text{ K} \quad \text{arm} = \tfrac{18}{3} = 6 \text{ ft}$$

$$M_u \text{(at bottom of wall)} = 2.57 \times 9 + 7.72 \times 6 = 69.45 \text{ K·ft}$$

The total depth used is 18 in., $b = 12$ in., and $d = 18 - 2$ (concrete cover) $- 0.5$ (half the bar diameter) $= 15.5$ in.

$$R_u = \frac{M_u}{bd^2} = \frac{69.45 \times 12{,}000}{12(15.5)^2} = 289 \text{ psi}$$

The steel ratio, ρ, can be obtained from Table A.1 in Appendix A or from

$$\rho = \frac{0.85 f_c'}{f_y}\left[1 - \sqrt{\frac{2R_u}{\phi 0.85 f_c'}}\right] = 0.007$$

$$A_s = 0.007(12)(15.5) = 1.3 \text{ in.}^2$$

Use no. 8 bars spaced at 7 in. (1.35 in.^{2}). The minimum vertical A_s according to the ACI Code, Section 14.3, is

$$A_{s\,\min} = 0.0015(12)(18) = 0.32 \text{ in.}^2 < 1.35 \text{ in.}^2$$

Because the moment decreases along the height of the wall, A_s may be reduced according to the moment requirements. It is practical to use one A_s or spacing, for the lower half and a second A_s, or spacing, for the upper half of the wall. To calculate the moment at midheight of the wall, 9 ft from the top,

$$P_1 = 1.6(0.271 \times 110 \times 3) = 143 \text{ lb}$$

$$P_2 = 1.6(0.271 \times 110 \times 9) = 429 \text{ lb}$$

$$H_{a_1} = 0.143 \times 9 = 1.29 \text{ K} \qquad \text{arm} = \frac{9}{2} = 4.5 \text{ ft}$$

$$H_{a_2} = \frac{1}{2} \times 0.429 \times 9 = 1.9 \text{ K} \qquad \text{arm} = \frac{9}{3} = 3 \text{ ft}$$

$$M_u = 1.29 \times 4.5 + 1.9 \times 3 = 11.5 \text{ K·ft}$$

The total depth at midheight of wall is

$$\frac{12+18}{2} = 15 \text{ in.}$$

$$d = 15 - 2 - 0.5 = 12.5 \text{ in.}$$

$$R_u = \frac{M_u}{bd^2} = \frac{11.5 \times 12{,}000}{12 \times (12.5)^2} = 73.6 \text{ psi}$$

$$\rho = 0.0017 \quad \text{and} \quad A_s = 0.0017(12)(12.5) = 0.25 \text{ in.}^2$$

$$A_{s\,\min} = 0.0015 \times 12 \times 15 = 0.27 \text{ in.}^2 > 0.25 \text{ in.}^2$$

Use no. 4 vertical bars spaced at 8 in. (0.29 in.^{2}) with similar spacing to the lower vertical steel bars in the wall.

b. Temperature and shrinkage reinforcement: The minimum horizontal reinforcement at the base of the wall according to ACI Code, Section 14.3, is

$$A_{s\,\min} = 0.0020 \times 12 \times 18 = 0.432 \text{ in.}^2$$

(for the bottom third), assuming no. 5 bars or smaller.

$$A_{s\,\min} = 0.0020 \times 12 \times 15 = 0.36 \text{ in.}^2$$

(for the upper two-thirds). Because the front face of the wall is mostly exposed to temperature changes, use one-half to two-thirds of the horizontal bars at the external face of the wall and place the balance at the internal face.

$$0.5A_s = 0.5 \times 0.432 = 0.22 \text{ in.}^2$$

Use no. 4 horizontal bars spaced at 8 in. ($A_s = 0.29$ in.^{2}) at both the internal and external surfaces of the wall. Use no. 4 vertical bars spaced at 12 in. at the front face of the wall to support the horizontal temperature and shrinkage reinforcement.

c. Dowels for the wall vertical bars: The anchorage length of no. 8 bars into the footing must be at least 22 in. Use an embedment length of 2 ft into the footing and the key below the stem.

d. Design for shear: The critical section for shear is at a distance $d = 15.5$ in. from the bottom of the stem. At this section, the distance from the top equals $18 - 15.5/12 = 16.7$ ft.

$$P_1 = 143 \text{ lb} \quad P_1 = 1.6(0.271 \times 110 \times 3) = 143 \text{ lb}$$

$$P_2 = 1.6(0.271 \times 110 \times 16.7) = 796 \text{ lb}$$

$$H_{a_1} = 0.143 \times 16.7 = 2.39 \text{ K}$$

$$H_{a_2} = \frac{1}{2} \times 0.796 \times 16.7 = 6.6 \text{ K}$$

$$\text{Total } H = 2.39 + 6.6 = 9.0 \text{ K}$$

$$\phi V_c = \phi(2\lambda\sqrt{f'_c})bd = \frac{0.75 \times 2 \times 1}{1000} \times \sqrt{3000} \times 12 \times 15.5$$

$$= 15.28 \text{ K} > 9.0\text{K}$$

7. Design of the heel: A load factor of 1.2 is used to calculate the factored bending moment and shearing force due to the backfill and concrete, whereas a load factor of 1.6 is used for the surcharge. The upward soil pressure is neglected, because it will reduce the effect of the

backfill and concrete on the heel. Referring to Fig. 14.12, the total load on the heel is

$$V_u = [1.2[(18 \times 5.5 \times 110) + (1.5 \times 5.5 \times 150)] + 1.6(3 \times 5.5 \times 100)]/1000$$
$$= 17.5 \text{ K}$$
$$M_u(\text{at face of wall}) = V_u \times \frac{5.5}{2} = 48.1 \text{ K·ft}$$

The critical section for shear is usually at a distance d from the face of the wall when the reaction introduces compression into the end region of the member. In this case, the critical section will be considered at the face of the wall, because tension and not compression develops in the concrete.

$$V_u = 17.2 \text{ K}$$
$$\phi V_c = \phi(2\lambda\sqrt{f'_c})bd = \frac{0.75 \times 2 \times 1}{1000} \times \sqrt{3000} \times 12 \times 14.5$$
$$= 14.3 \text{ K}$$

ϕV_c is less than V_u of 17.2 K, and the section must be increased by the ratio 17.5/14.3 or shear reinforcement must be provided.

$$\text{Required } d = \frac{17.2}{14.3} \times 14.5 = 17.4 \text{ in.}$$
$$\text{Total thickness required} = 17.4 + 3.5 = 20.9 \text{ in.}$$

Use a base thickness of 22 in. and $d = 18.5$ in.

$$R_u = \frac{M_u}{bd^2} = \frac{48.1 \times 12{,}000}{12 \times (18.5)^2} = 140.5 \text{ psi} \quad \rho = 0.0027$$
$$A_s = \rho bd = 0.60 \text{ in}^2$$
$$\text{Min. shrinkage } A_s = 0.0018(12)(22) = 0.475 \text{ in.}^2$$
$$\text{Min. flexural } A_s = 0.0033(12)(18.5) = 0.733 \text{ in.}^2$$

Use no. 6 bars spaced at 7 in. ($A_s = 0.76$ in.^{2}). The development length for the no. 6 top bars equals $1.4l_d = 35$ in. Therefore, the bars must be extended 3 ft into the toe of the base.

Temperature and shrinkage reinforcement in the longitudinal direction is not needed in the heel or toe, but it may be preferable to use minimal amounts of reinforcement in that direction, say, no. 4 bars spaced at 12 in.

8. Design of the toe: The toe of the base acts as a cantilever beam subjected to upward pressures, as calculated in step 4. The factored soil pressure is obtained by multiplying the service load soil pressure by a load factor of 1.6, because it is primarily caused by the lateral forces. The critical section for the bending moment is at the front face of the stem. The critical section for shear is at a distance d from the front face of the stem, because the reaction in the direction of shear introduces compression into the toe.

Referring to Fig. 14.13, the toe is subjected to an upward pressure from the soil and downward pressure due to self-weight of the toe slab.

$$V_u = 1.6\left(\frac{3.13 + 2.62}{2}\right) \times 1.96 - 1.2\left(\frac{22}{12} \times 0.150\right) \times 1.96$$
$$= 837 \text{ K}$$

This is less than ϕV_c of 14.3 K calculated for the heel in step 7.

$$M_u = 1.6\left[\frac{2.22}{2} \times (3.5)^2 + (3.13 - 2.22) \times 3.5 \times 0.5\left(\frac{2}{3} \times 3.5\right)\right]$$

$$- 1.2\left[\left(\frac{22}{12} \times 0.150\right) \times \frac{(3.5)^2}{2}\right] = 25.7 \text{ K·ft}$$

$$R_u = \frac{M_u}{bd^2} = \frac{25.7 \times 12{,}000}{12 \times (18.5)^2} = 75 \text{ psi} \qquad \rho = 0.0017$$

$$A_s = 0.0017(12)(18.5) = 0.377 \text{ in.}^2$$

$$\text{Min. shrinkage } A_s = 0.0018(12)(22) = 0.475 \text{ in.}^2$$

$$\text{Min. flexural } A_s = 0.0033(12)(18.5) = 0.733 \text{ in.}^2$$

Use no. 6 bars spaced at 7 in., similar to the heel reinforcement. Development length of no. 6 bars equals 25 in. Extend the bars into the heel 25 in. The final reinforcement details are shown in Fig. 14.14.

9. Shear keyway between wall and footing: In the construction of retaining walls, the footing is cast first and then the wall is cast on top of the footing at a later date. A construction joint

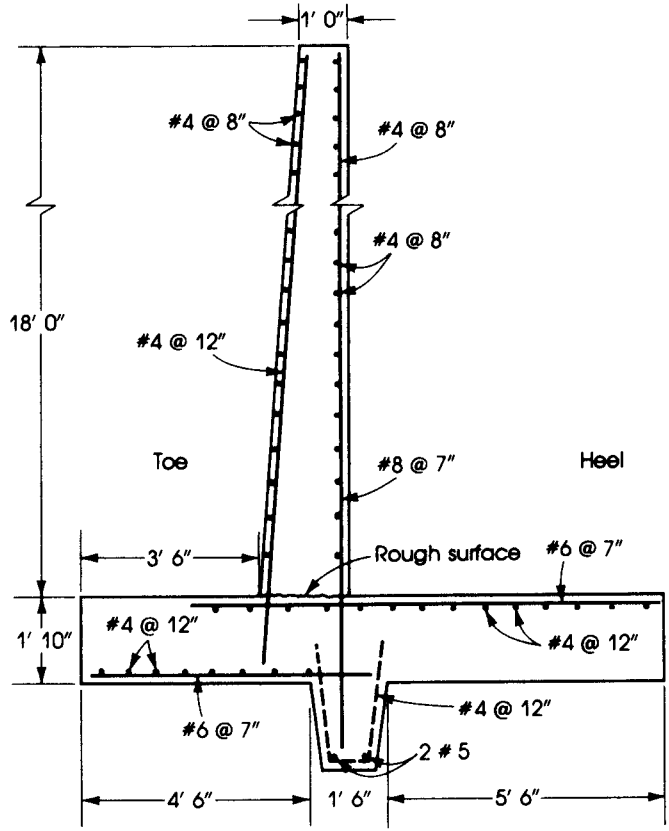


Figure 14.14 Example 14.2: reinforcement details.

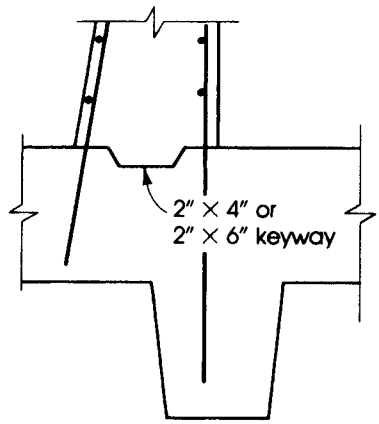


Figure 14.15 Example 14.2: keyway details.

is used at the base of the wall. The joint surface takes the form of a keyway, as shown in Fig. 14.15, or is left in a very rough condition (Fig. 14.14). The joint must be capable of transmitting the stem shear into the footing.

10. Proper drainage of the backfill is essential in this design, because the earth pressure used is for drained backfill. Weep holes should be provided in the wall, 4 in. in diameter and spaced at 5 ft in the horizontal and vertical directions.

14.11 BASEMENT WALLS

It is a common practice to assume that basement walls span vertically between the basement-floor slab and the first-floor slab. Two possible cases of design should be investigated for a basement wall.

First, when the wall only has been built on top of the basement floor slab, the wall will be subjected to lateral earth pressure with no vertical loads except its own weight. The wall in this case acts as a cantilever, and adequate reinforcement should be provided for a cantilever wall design. This case can be avoided by installing the basement and the first-floor slabs before backfilling against the wall.

Second, when the first-floor and the other floor slabs have been constructed and the building is fully loaded, the wall in this case will be designed as a propped cantilever wall subjected to earth pressure and to vertical load.

For an angle of internal friction of 35°, the coefficient of active pressure is $C_a = 0.271$. The horizontal earth pressure at the base is $\rho_a = C_a wh$. For w = 110 pcf and an average height of a basement of $h = 10$ ft, then

$$P_a = 0.271 \times 0.110 \times 10 = 0.3 \text{ ksf}$$

$$H_a = 0.271 \times 0.110 \times \frac{100}{2} = 1.49 \text{ K/ft of wall}$$

H_a acts at $h/3 = 10/3 = 3.33$ ft from the base. An additional pressure must be added to allow for a surcharge of about 200 psf on the ground behind the wall. The equivalent height to the

surcharge is

$$h_s = \frac{200}{110} = 1.82 \text{ ft}$$

$$\rho_s = C_a w h_s = 0.271 \times 0.110 \times 1.82 = 0.054 \text{ ksf}$$

$$H_s = (C_a w h_s) \times h = 0.054 \times 10 = 0.54 \text{ K/ft of wall}$$

H_s of the surcharge acts at $h/2 = 5ft$ from the base.

In the preceding calculations, it is assumed that the backfill is dry, but it is necessary to investigate the presence of water pressure behind the wall. The maximum water pressure occurs when the whole height of the basement wall is subjected to water pressure, and $\rho_w = wh = 62.5 \times 10 = 625$ psf.

$$H_w = \frac{wh^2}{2} = 0.625 \times 5 = 3.125 \text{ K/ft of wall}$$

The maximum pressure may not be present continuously behind the wall. Therefore, if the ground is intermittently wet, a percentage of the preceding pressure may be adopted, say, 50%:

$$\frac{P_w}{2} = \frac{0.625}{2} = 0.31 \text{ ksf}$$

$$H'_w = \frac{H_w}{2} = (0.5wh)\frac{h}{2} = \frac{3.125}{2} = 1.56 \text{ K/ft of wall}$$

H'_w acts at $h/3 = \frac{10}{3} = 3.33$ ft from the base. Water may be prevented from collecting against the wall by providing drains at the lower end of the wall.

In addition to drainage, a waterproofing or damp-proofing membrane must be laid or applied to the external face of the wall. The ACI Code, Section 14.5.3, specifies that the minimum thickness of an exterior basement wall and its foundation is 7.5 in. In general, the minimum thickness of bearing walls is $\frac{1}{25}$ of the supported height or length, whichever is shorter, or 4 in.

Example 14.3

Determine the thickness and necessary reinforcement for the basement retaining wall shown in Fig. 14.16. Given: Weight of backfill = 110 pcf, angle of internal friction = 35°, $f'_c = 3$ ksi, and $f_y = 60$ ksi.

Solution

1. The wall spans vertically and will be considered as fixed at the bottom end and propped at the top. Consider a span of $L = 9.75$ ft, as shown in Fig. 14.16. For these data, the different lateral pressures on a 1-ft length of the wall are as follows: Due to active soil pressure, $p_a = 0.3$ ksf and $H_a = 1.49$ K. Due to water pressure, $p_w = 0.31$ ksf and $H_w = 1.56$ K. Due to surcharge, $p_s = 0.054$ ksf and $H_s = 0.54$ K. H_a and H_w are due to triangular loadings, whereas H_S is due to uniform loading. Referring to Fig. 14.16 and using moment coefficients of a propped beam



Basement Wall

subjected to triangular and uniform loads, and a load factor = 1.6 (ACI Code, Appendix C)

$$M_u = 1.6(H_a + H_w)\frac{L}{7.5} + 1.6H_s\frac{L}{8}$$

$$= 1.6\left(\frac{3.05}{7.5} \times 9.75 + 0.54 \times \frac{9.75}{8}\right) = 7.41 \text{ K·ft}$$

$$R_B = 1.6\left(\frac{3.05}{3} + \frac{0.54}{2}\right) - \frac{7.41}{9.75} = 1.3 \text{ K}$$

$$R_A = 4.45 \text{ K}$$

Maximum positive bending moment within the span occurs at the section of 0 shear.

$$V_u = 1.3 - 1.6(0.054x) - 1.6\left(0.063\frac{x^2}{2}\right) = 0$$

$$x = 4.3 \text{ ft}$$

$$M_c = 1.3 \times 4.3 - 1.6\left[\frac{0.054}{2}(4.3)^2 + \frac{0.27}{2}\frac{(4.3)^2}{3}\right]$$

$$= +3.45 \text{ K·ft}$$

2. Assuming 0.01 steel ratio and $R_u = 332$ psi,

$$d = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{7.41 \times 12}{0.332 \times 12}} = 4.72 \text{ in.}$$

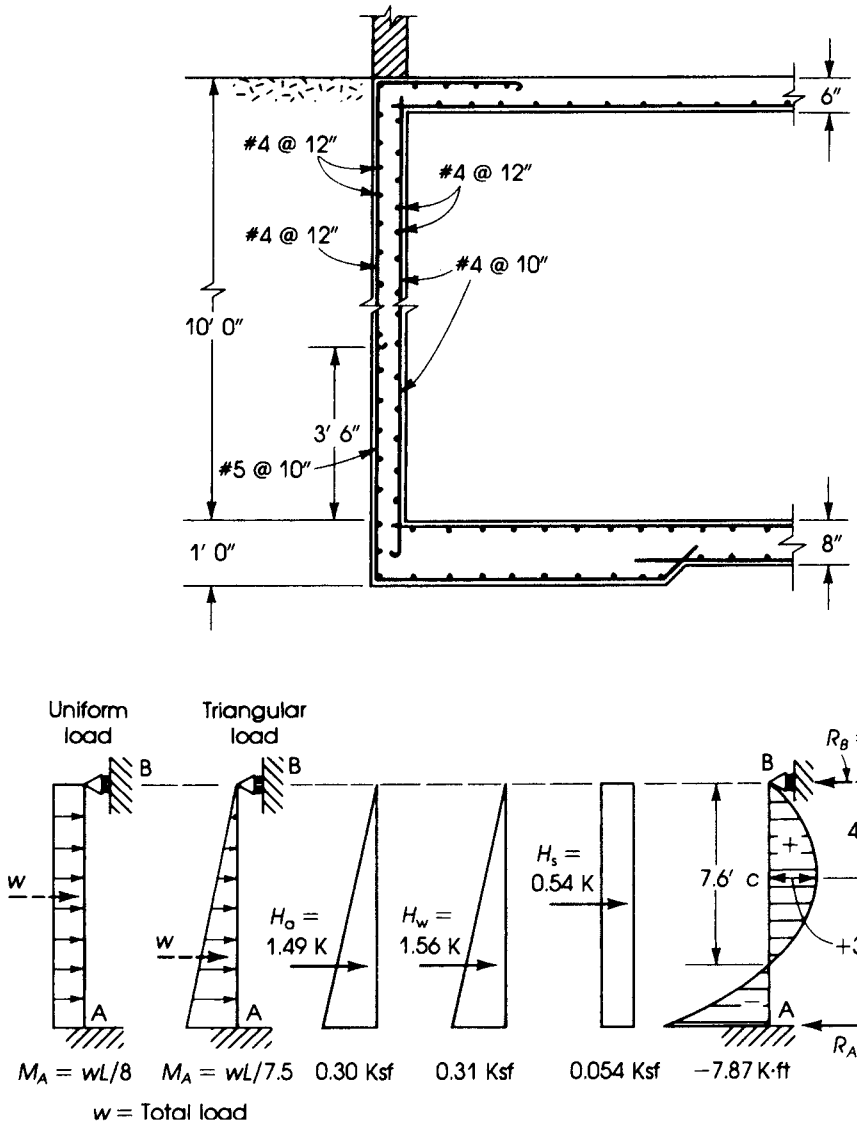


Figure 14.16 Example 14.3: basement wall.

Total depth = 4.72 + 1.5 (concrete cover) + 0.25 = 6.47 in. Use a $7\frac{1}{2}$ in. slab. $d = 5.75$ in.

$$R_u = \frac{M_u}{bd^2} = \frac{7.41 \times 12{,}000}{12 \times (5.75)^2} = 226 \text{ psi}$$

The steel ratio is $\rho = 0.0054$ and $A_s = 0.0054 \times 12 \times 5.75 = 0.369 \text{ in.}^2$

Minimum $A_s = 0.0015\,bh = 0.0015(12)(7.5) = 0.135 \text{ in.}^2$ (vertical bars)

Minimum A_s(flexure) $= 0.0033(12)(5.75) = 0.23 \text{ in.}^2$

Use no. 5 bars spaced at 10 in. ($A_s = 0.37 \text{ in}^2$).

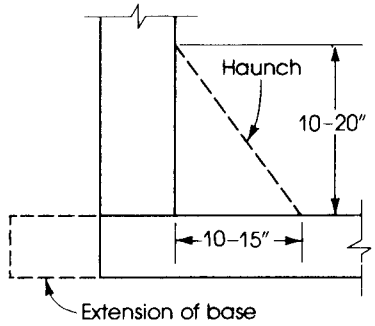


Figure 14.17 Example 14.3: adjustment of wall base.

3. For the positive moment, $M_c = 3.45$ K·ft:

$$R_u = \frac{3.45 \times 12{,}000}{12 \times (5.75)^2} = 105 \text{ psi} \quad \rho = 0.0020$$

$$A_s = 0.002 \times 12 \times 5.75 = 0.14 \text{ in.}^2 < 0.33 \text{ in.}^2$$

Use no. 4 bars spaced at 10 in. ($A_s = 0.24$ in^2).

4. Zero moment occurs at a distance of 7.6 ft from the top and 2.15 ft from the base. The development length of no. 5 bars is 14 in. Therefore, extend the main no. 5 bars to a distance of 2.15 + 1.2 = 3.35 ft, or 3.5 ft, above the base; then use no. 4 bars spaced at 12 in. at the exterior face. For the interior face, use no. 4 bars spaced at 10 in. throughout.
5. Longitudinal reinforcement: Use a minimum steel ratio of 0.0020 (ACI Code, Section 14.3), or $A_s = 0.0020 \times 7 \times 12 = 0.17$ in^2. Use no. 4 bars spaced at 12 in. on each side of the wall.
6. If the bending moment at the base of the wall is quite high, it may require a thick wall slab, for example, 12 in. or more. In this case a haunch may be adopted, as shown in Fig. 14.17. This solution will reduce the thickness of the wall, because it will be designed for the moment at the section exactly above the haunch.
7. The basement slab may have a thickness greater than the wall thickness and may be extended outside the wall by about 10 in. or more, as required.

SUMMARY

Sections 14.1–14.3

1. A retaining wall maintains unequal levels of earth on its two faces. The most common types of retaining walls are gravity, semigravity, cantilever, counterfort, buttressed, and basement walls.
2. For a linear pressure, the active and passive pressure intensities are

$$P_a = C_a w h \quad \text{and} \quad P_p = C_p w h$$

According to Rankine's theory,

$$C_a = \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) \quad \text{and} \quad C_p = \left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$$

Values of C_a and C_p for different values of ϕ and δ are given in Tables 14.2 and 14.3.

Sections 14.4–14.5

1. When soil is saturated, the submerged unit weight must be used to calculate earth pressure. The hydrostatic water pressure must also be considered.
2. A uniform surcharge on a retaining wall causes an additional pressure height, $h_s = w_s/w$.

Sections 14.6–14.8

1. A total frictional force, F, to resist sliding effect is

$$F = \mu R + H_p \tag{14.13}$$

$$\text{Factor of safety against sliding} = \frac{F}{H_{ah}} \geq 1.5 \tag{14.14}$$

2. Factor of safety against overturing is

$$\frac{M_b}{M_0} = \frac{\sum wx}{H_a h/3} \geq 2.0 \tag{14.15}$$

3. Approximate dimensions of a cantilever retaining wall are shown in Fig. 14.8.

Sections 14.9–14.10

1. Minimum reinforcement is needed in retaining walls.
2. To avoid hydrostatic pressure on a retaining wall, a drainage system should be used that consists of weep holes, perforated pipe, or any other adequate device.
3. Basement walls in buildings may be designed as propped cantilever walls subjected to earth pressure and vertical loads. This case occurs only if the first-floor slab has been constructed. A surcharge of 200 psf may be adopted.

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PROBLEMS

14.1 Check the adequacy of the retaining wall shown in Fig. 14.18 with regard to overturning, sliding, and the allowable soil pressure. Given: Weight of backfill. = 110 pcf, the angle of internal friction is $\phi = 30^\circ$, the coefficient of friction between concrete and soil is $\mu = 0.5$, allowable soil pressure = 3.5 ksf, and $f'_c = 3$ ksi.

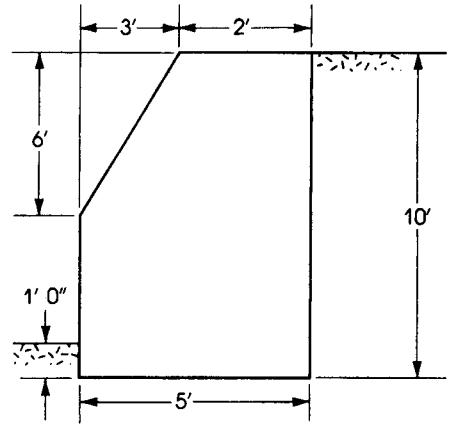


Figure 14.18 Problem 14.1: gravity wall.

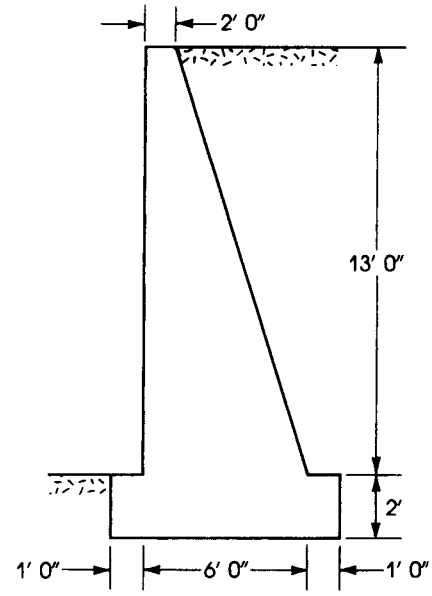


Figure 14.19 Problem 14.2: semigravity wall.

14.2 Repeat Problem 14.1 with Fig. 14.19.

14.3 For each problem in Table 14.4, determine the factor of safety against overturning and sliding. Also, determine the soil pressure under the wall footing and check if all calculated values are adequate (equal or below the allowable values). Given: Weight of soil = 110 pcf, weight of concrete = 150 pcf, and coefficient of friction between concrete and soil is 0.5 and between soil layers is 0.6. Consider that the allowable soil pressure of 4 ksf and the top of the backfill is level without surcharge. Neglect the passive soil resistance. See Fig. 14.20. ($\phi = 35°$.)

14.4 Repeat Problems 14.3*e*–*h*, assuming a surcharge of 300 psf.

Table 14.4 Problem

Problem No.	H	h_f	A	B	C	L
(a)	12	1.00	2.0	1.0	4.0	7
(b)	14	1.50	2.0	1.5	4.5	8
(c)	15	1.50	2.0	1.5	4.5	8
(d)	16	1.50	3.0	1.5	4.5	9
(e)	17	1.50	3.0	1.5	4.5	9
(f)	18	1.75	3.0	1.75	5.25	10
(g)	19	1.75	3.0	1.75	5.25	10
(h)	20	2.00	3.0	2.0	6.0	11
(i)	21	2.00	3.5	2.0	6.5	12
(j)	22	2.00	3.5	2.0	6.5	12

Refer to Fig. 14.20. All dimensions are in feet.

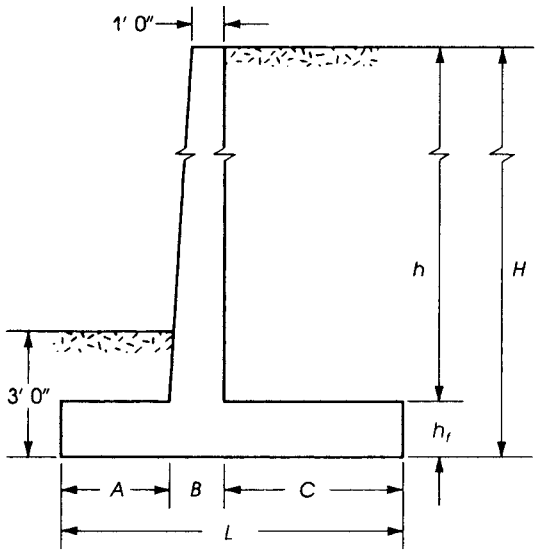


Figure 14.20 Problem 14.3.

14.5 Repeat Problems 14.3*e*–*h*, assuming that the backfill slopes at 10° to the horizontal. (Add key if needed.)

14.6 For Problems 14.3*e*–*h*, determine the reinforcement required for the stem, heel, and toe, and choose adequate bars and distribution. Use $f'_c = 3$ ksi and $f_y = 60$ ksi.

14.7 Determine the dimensions of a cantilever retaining wall to support a bank of earth 16 ft high. Assume that frost penetration depth is 4 ft. Check the safety of the retaining wall against overturning and sliding only. Given: Weight of backfill = 120 pcf, angle of internal friction = 33°, coefficient of friction between concrete and soil = 0.45, coefficient of friction between soil layers = 0.65, and allowable soil pressure = 4 ksf. Use a 1.5 × 1.5-ft key if needed.

14.8 A complete design is required for the retaining wall shown in Fig. 14.21. The top of the backfill is to be level without surcharge. Given: Weight of backfill soil = 110 pcf, angle of internal

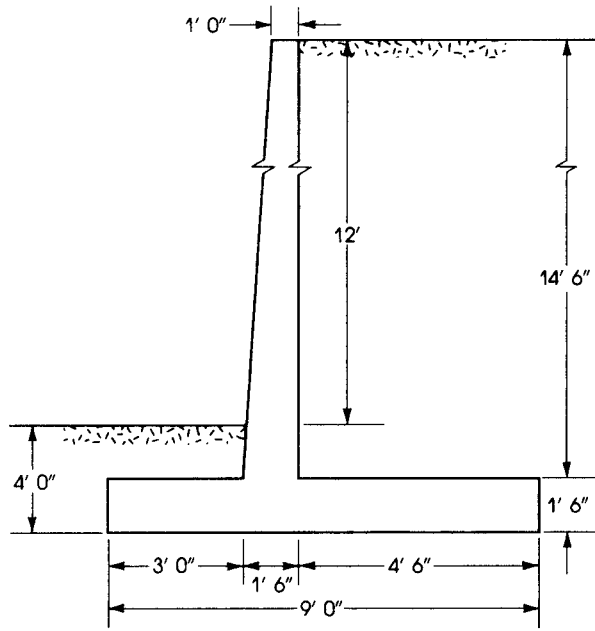


Figure 14.21 Problem 14.8: cantilever retaining wall.

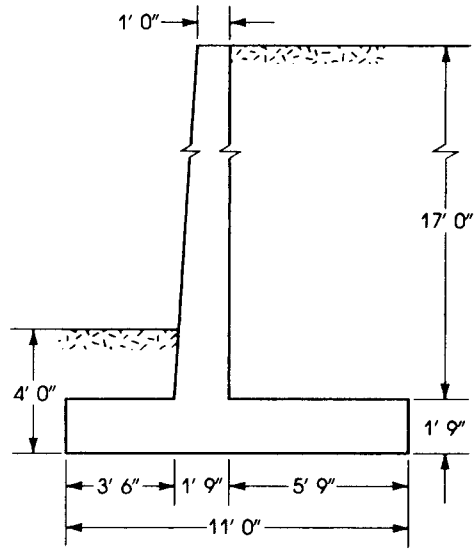


Figure 14.22 Problem 14.9: cantilever retaining wall.

friction = 35°, the coefficient of friction between concrete and soil is 0.55, and that between soil layers is 0.6. Use $f'_c = 3$ ksi, and $f_y = 60$ ksi, and an allowable soil pressure of 4 ksf.

14.9 Check the adequacy of the cantilever retaining wall shown in Fig. 14.22 for both sliding and over-turning conditions. Use a key of 1.5 × 1.5 ft if needed. Then determine reinforcement needed for the stem, heel, and toe, and choose adequate bars and distribution. Given: Weight of soil = 120 pcf,

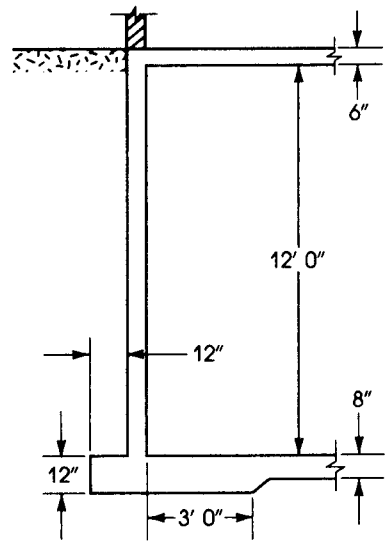


Figure 14.23 Problem 14.11: basement wall.

the angle of internal friction is $\phi = 35°$, the coefficient of friction between concrete and soil is 0.52 and that between soil layers is 0.70. Use $f'_c = 3$ ksi, $f_y = 60$ ksi, an allowable soil pressure of 4 ksf, and a surcharge of 300 psf.

14.10 Repeat Problem 14.9, assuming the backfill slopes at 30° to the horizontal.

14.11 Determine the thickness and necessary reinforcement for the basement wall shown in Fig. 14.23. The weight of backfill is 120 pcf and the angle of internal friction is $\phi = 30°$. Assume a surcharge of 400 psf and use $f'_c = 3$ ksi and $f_y = 60$ ksi.

14.12 Repeat Problem 14.11 using a basement clear height of 14 ft.

CHAPTER 15

DESIGN FOR TORSION



Apartment Building, Habitat 67, Montreal, Canada.

15.1 INTRODUCTION

Torsional stresses develop in a beam section when a moment acts on that section parallel to its surface. Such moments, called torsional moments, cause a rotation in the structural member and cracking on its surface, usually in the shape of a spiral. To illustrate torsional stresses, let a torque T be applied on a circular cantilever beam made of elastic homogeneous material, as shown in Fig. 15.1. The torque will cause a rotation of the beam. Point B moves to point B' at one end of the beam, whereas the other end is fixed. The angle θ is called the angle of twist. The plane $AO'\ OB$ will be distorted to the shape $AO'\ OB'$. Assuming that all longitudinal elements have the same length, the shear strain is

$$\gamma = \frac{(BB')}{L} = \frac{r\theta}{L}$$

where L is the length of the beam and r is the radius of the circular section.

In reinforced concrete structures, members may be subjected to torsional moments when they are curved in plan, support cantilever slabs, act as spandrel beams (end beams), or are part of a spiral stairway.

Structural members may be subjected to pure torsion only or, as in most cases, subjected simultaneously to shearing forces and bending moments. Example 15.1 illustrates the different forces that may act at different sections of a cantilever beam.

Example 15.1

Calculate the forces acting at sections 1, 2, and 3 of the cantilever beam shown in Fig. 15.2. The beam is subjected to a vertical force $P_1 = 15\,\text{K}$, a horizontal force $P_2 = 12\,\text{K}$ acting at C, and a horizontal force $P_3 = 20\,\text{K}$ acting at B and perpendicular to the direction of the force P_2.

Solution

Let N = normal force, V = shearing force, M = bending moment, and T = torsional moment. The forces are as follows.

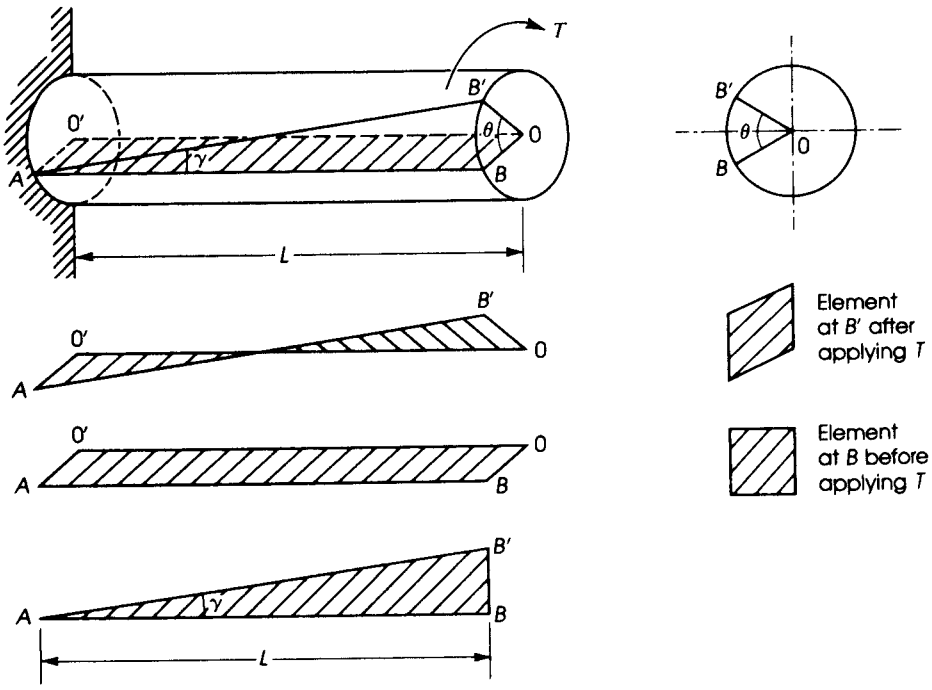


Figure 15.1 Torque applied to a cantilever beam.

Section	N (K)	M_x (K·ft)	M_y (K·ft)	V_x (K)	V_y (K)	T (K·ft)
1	0 (15 × 9)	−135 (12 × 9)	+108	+12	+15	0
2	−12 Compression	0	+108	+20	+15	135 (15 × 9)
3	−12 Compression	−180	+348	+20	+15	135 (15 × 9)

If P_1, P_2, and P_3 are factored loads ($P_u = 1.2P_D + 1.6P_L$), then the values in the table will be the factored design forces.

15.2 TORSIONAL MOMENTS IN BEAMS

It was shown in Example 15.1 that forces can act on building frames, causing torsional moments. If a concentrated load P is acting at point C in the frame ABC shown in Fig. 15.3*a*, it develops a torsional moment in beam AB of $T = PZ$ acting at D. When D is at midspan of AB, then the torsional design moment in AD equals that in DB, or $\frac{1}{2}T$. If a cantilever slab is supported by the beam AB in Fig. 15.3*b*, the slab causes a uniform torsional moment m_t along AB. This uniform torsional moment is due to the load on a unit width strip of the slab. If S is the width of the cantilever slab and w is load on the slab (psf), then $m_t = wS^2/2$ K·ft/ft of beam AB.

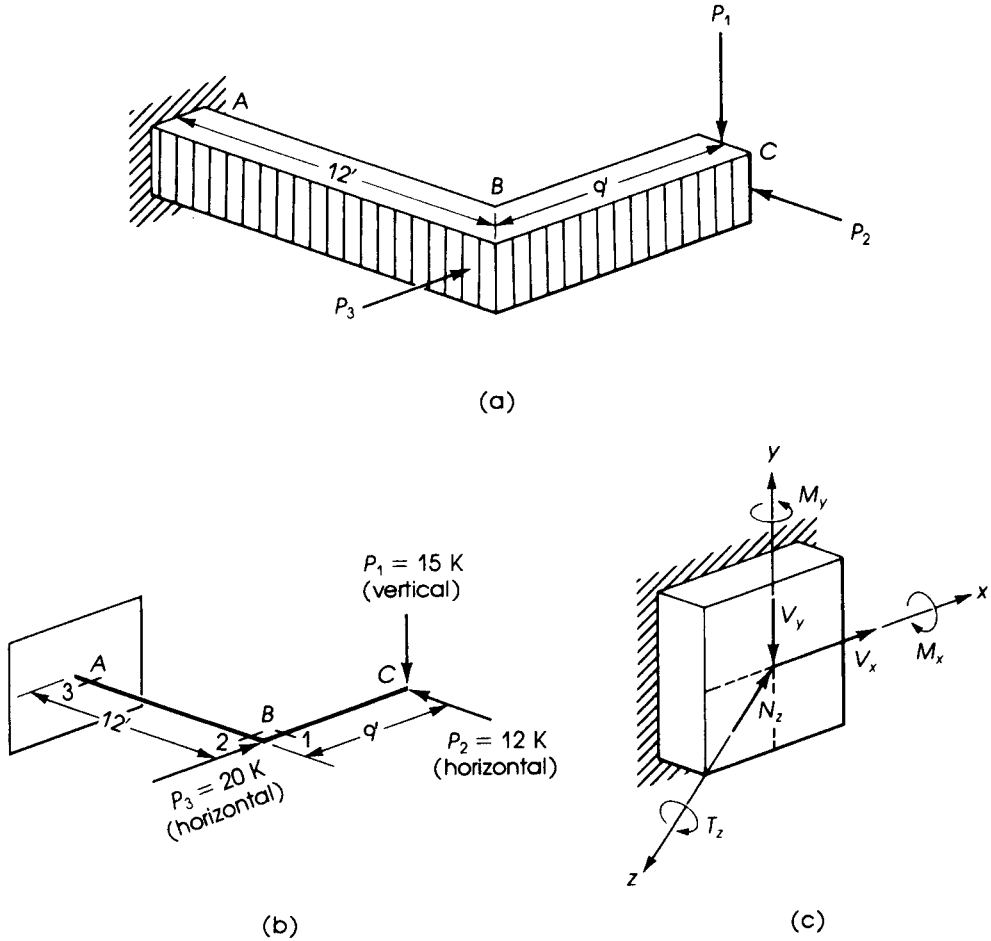


Figure 15.2 Example 15.1.

The maximum torsion design moment in beam AB is $T = \frac{1}{2}m_t L$ acting at A and B. Other cases of loading are explained in Table 15.1. In general, the distribution of torsional moments in beams has the same shape and numerically has the same values as the shear diagrams for beams subjected to a load m_t or T.

15.3 TORSIONAL STRESSES

Considering the cantilever beam with circular section of Fig. 15.1, the torsional moment T will cause a shearing force dV perpendicular to the radius of the section. From the conditions of equilibrium, the external torsional moment is resisted by an internal torque equal to and opposite to T. If dV is the shearing force acting on the area dA (Fig. 15.4), then the magnitude of the torque is $T = \int r\, dV$. Let the shearing stress be v; then

$$dV = v\, dA \quad \text{and} \quad T = \int rv\, dA$$

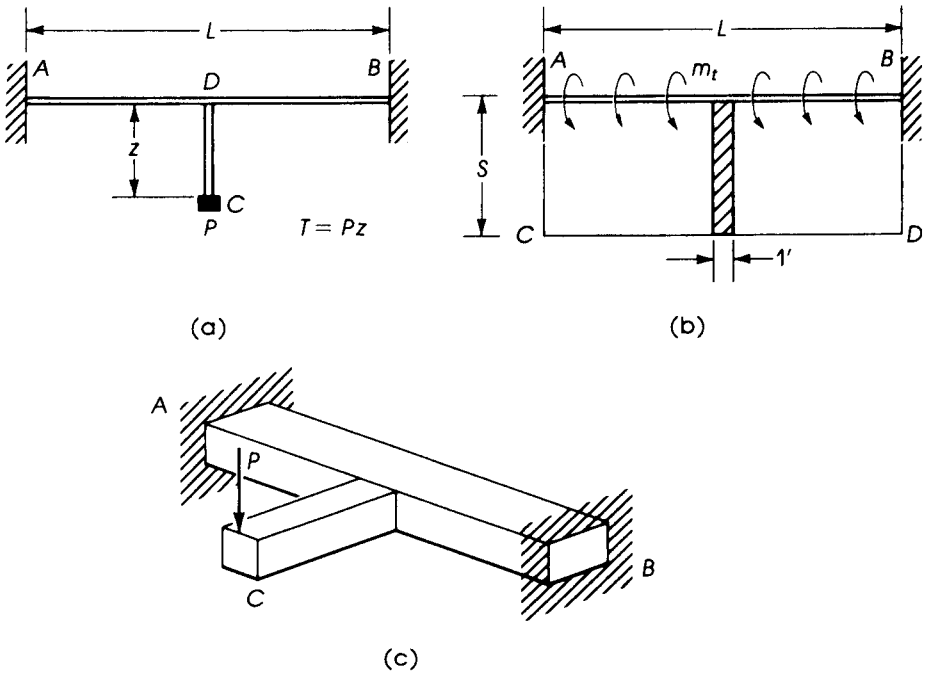


Figure 15.3 Torsional moments on *AB*.

The maximum elastic shear occurs at the external surface of the circular section at radius r with thickness dr; then the torque T can be evaluated by taking moments about the center 0 for the ring area:

$$dT = (2\pi r\, dr)vr$$

where $(2\pi r\, dr)$ is the area of the ring and v is the shear stress in the ring. Thus,

$$T = \int_0^R (2\pi r dr)vr = \int_0^R 2\pi r^2\, v\, dr \tag{15.1}$$

For a hollow section with internal radius R_1,

$$T = \int_{R1}^R 2\pi r^2\, v\, dr \tag{15.2}$$

For a solid section, using Eq. 15.1 and using $v = v_{\max}\, r/R$,

$$T = \int_0^R 2\pi r^2 \left(\frac{v_{\max} r}{R}\right) dr = \left(\frac{2\pi}{R}\right) v_{\max} \int_0^R r^3 dr$$

$$= \left(\frac{2\pi}{R}\right) v_{\max} \times \frac{R^4}{4} = \left(\frac{\pi}{2}\right) v_{\max} R^3$$

$$v_{\max} = \frac{2T}{\pi r^3} \tag{15.3}$$

Table 15.1 Torsion Diagrams

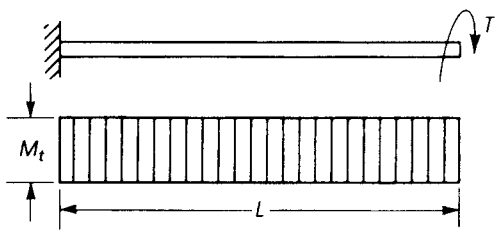
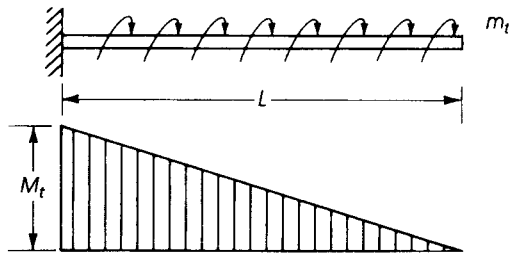
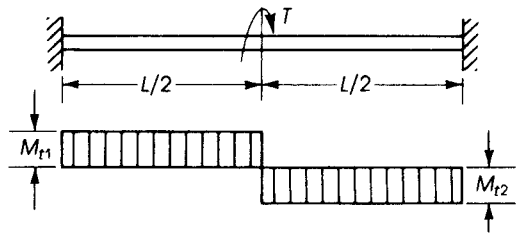
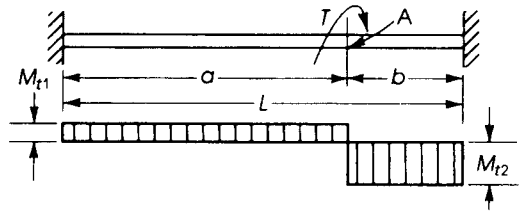
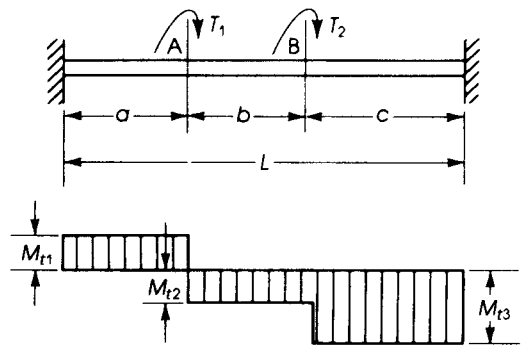
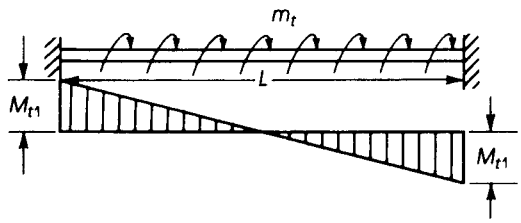
	At support $M_t = T$	For a circular section $J = \frac{\pi R^4}{2}$
	At support $M_t = m_t L$ m_t = uniform torque	
	$M_{t1} = M_{t2} = \frac{T}{2}$	
	$M_{t1} = \frac{Tb}{L}$ $M_{t2} = \frac{Ta}{L}$	
	$M_{t1} = \frac{T_1(b-c) + T_2 c}{L}$ $M_{t2} = \frac{T_2 c - T_1 a}{L}$ $M_{t3} = \frac{T_1 a - T_2(a+b)}{L}$ Note: When $a = b = c = L/\beta$ and $T_1 = T_2$ $= M_{t1} = -M_{t3} = T \cdot M_{t2} = 0$	

Table 15.1 *(continued)*

	$M_{t1} = \frac{m_t L}{2}$

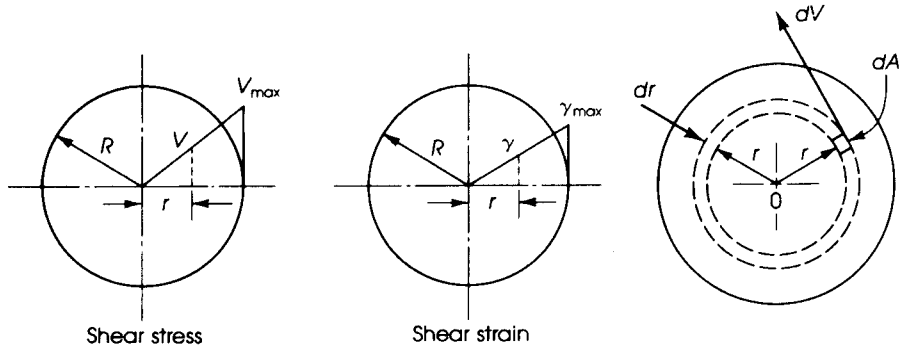


Figure 15.4 Torque in circular sections.

The polar moment of inertia of a circular section is $J = \pi R^4/2$. Therefore, the shear stress can be written as a function of the polar moment of inertia J as follows:

$$v_{max} = \frac{TR}{J} \tag{15.4}$$

15.4 TORSIONAL MOMENT IN RECTANGULAR SECTIONS

The determination of the stress in noncircular members subjected to torsional loading is not as simple as that for circular sections. However, results obtained from the theory of elasticity in dicate that the maximum shearing stress for rectangular sections can be calculated as follows:

$$v_{max} = \frac{T}{\alpha x^2 y} \tag{15.5}$$

where

T = the applied torque
x = short side of the rectangular section
y = the long side of the rectangular section
α = coefficient that depends on the ratio of y/x; its value is given in the following table.

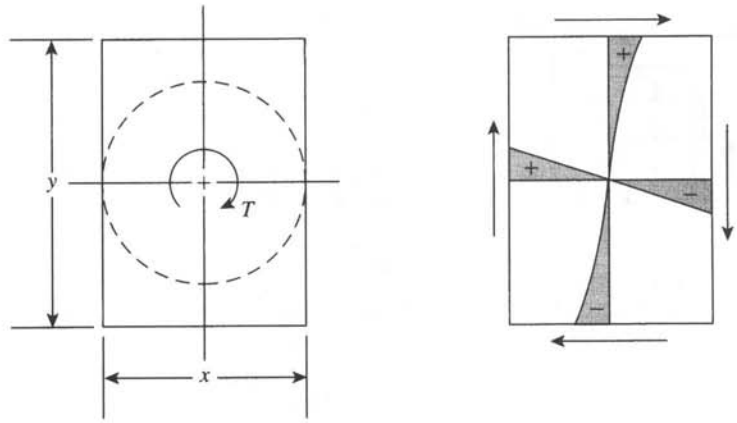


Figure 15.5 Stress distribution in rectangular sections due to pure torsion.

y/x	1.0	1.2	1.5	2.0	4	10
α	0.208	0.219	0.231	0.246	0.282	0.312

The maximum shearing stress occurs along the centerline of the longer side y (Fig. 15.5).

For members composed of rectangles, such as T-, L-, or I-sections, the value of α can be assumed equal to be $\frac{1}{3}$, and the section may be divided into several rectangular components having a long side y_i and a short side x_i. The maximum shearing stress can be calculated from

$$v_{\max} = \frac{3T}{\Sigma x_i^2 y_i} \tag{15.6}$$

where $\Sigma x_i^2 y_i$ is the value obtained from the rectangular components of the section. When $y/x \leq 10$, a better expression may be used:

$$v_{\max} = \frac{3T}{\sum x^2 y \left(1 - 0.63\frac{x}{y}\right)} \tag{15.7}$$

15.5 COMBINED SHEAR AND TORSION

In most practical cases, a structural member may be subjected simultaneously to both shear and torsional forces. Shear stresses will be developed in the section, as was explained in Chapter 8, with an average shear $= v_1$ in the direction of the shear force V (Fig. 15.6*a*). The torque T produces torsional stresses along all sides of the rectangular section $ABCD$ (Fig. 15.6*a*), with $v_3 > v_2$. The final stress distribution is obtained by adding the effect of both shear and torsion stresses to produce maximum value of $(v_1 + v_3)$ on side CD, whereas side AB will have a final stress of $(v_1 - v_3)$. Both sides AD and BC will be subjected to torsional stress v_2 only. The section must be designed for the maximum $v = (v_1 + v_3)$.

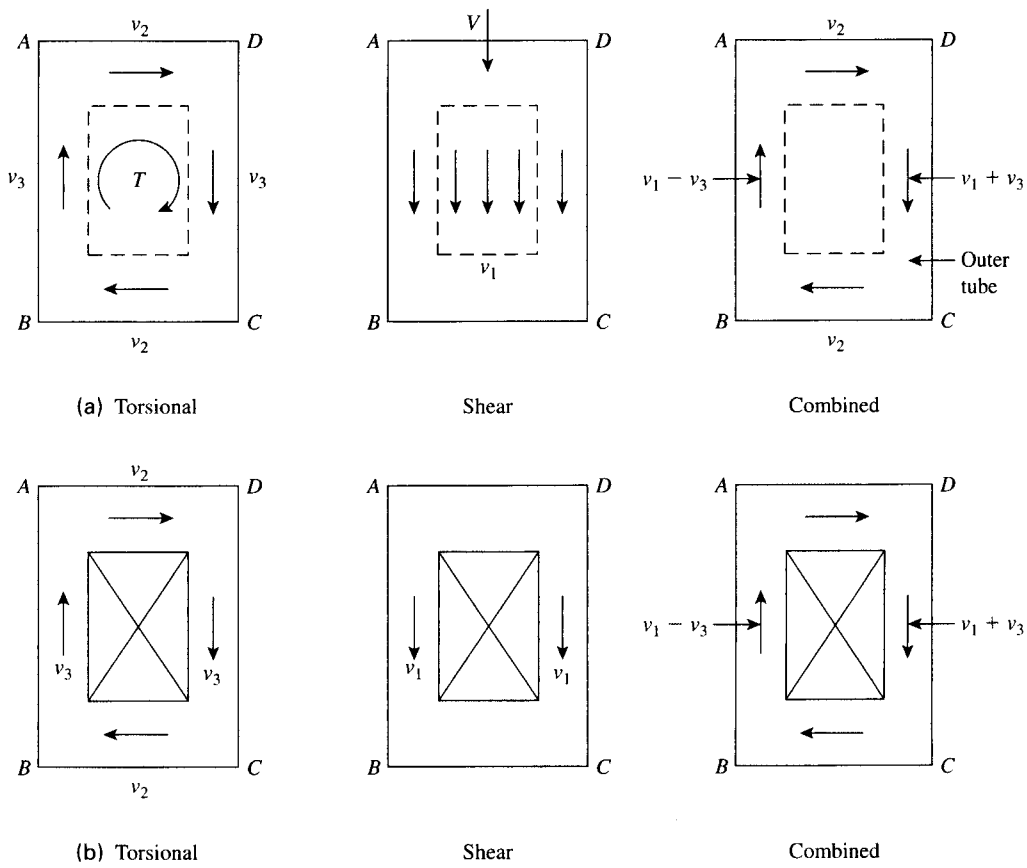


Figure 15.6 Combined shear and torsional stresses: (*a*) solid sections and (*b*) hollow sections.

15.6 TORSION THEORIES FOR CONCRETE MEMBERS

Various methods are available for the analysis of reinforced concrete members subjected to torsion or simultaneous torsion, bending, and shear. The design methods rely generally on two basic theories: the skew bending theory and the space truss analogy.

15.6.1 Skew Bending Theory

The skew bending concept was first presented by Lessig in 1959 [2] and was further developed by Goode and Helmy [3], Collins et al. in 1968 [4], and Below et al. in 1975 [5]. The concept was applied to reinforced concrete beams subjected to torsion and bending. Expressions for evaluating the torsional capacity of rectangular sections were presented by Hsu in 1968 [6,7] and were adopted by the ACI Code of 1971. Torsion theories for concrete members were discussed by Zia [8]. Empirical design formulas were also presented by Victor et al. in 1976 [9].

The basic approach of the skew bending theory, as presented by Hsu, is that failure of a rectangular section in torsion occurs by bending about an axis parallel to the wider face of the section y and inclined at about 45° to the longitudinal axis of the beam (Fig. 15.7). Based on

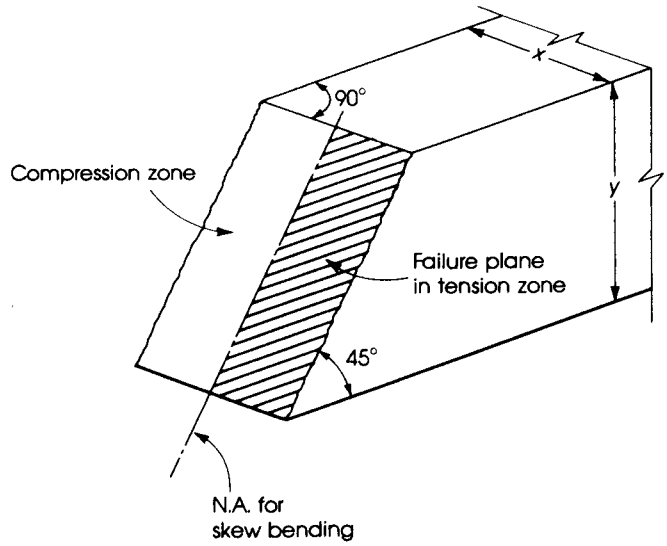


Figure 15.7 Failure surface due to skew bending.

this approach, the minimum torsional moment, T_n, can be evaluated as follows:

$$T_n = \left(\frac{x^2 y}{3}\right) f_r \tag{15.8}$$

where f_r is the modulus of rupture of concrete; f_r is assumed to be $5\sqrt{f_c'}$ in this case, as compared to $7.5\lambda\sqrt{f_c'}$ adopted by the ACI Code for the computation of deflection in beams.

The torque resisted by concrete is expressed as follows:

$$T_c = \left(\frac{2.4}{\sqrt{x}}\right) x^2 y \sqrt{f_c'} \tag{15.9}$$

and the torque resisted by torsional reinforcement is

$$T_s = \frac{\alpha_1 (x_1 y_1 A_t f_y)}{s} \tag{15.10}$$

Thus, $T_n = T_c + T_s$, where T_n is the nominal torsional moment capacity of the section.

15.6.2 Space Truss Analogy

The space truss analogy was first presented by Rausch in 1929 and was further developed by Lampert [10,11], who supported his theoretical approach with extensive experimental work. The Canadian Code provisions for the design of reinforced concrete beams in torsion and bending are based on the space truss analogy. Mitchell and Collins [12] presented a theoretical model for structural concrete in pure torsion. McMullen and Rangan [13] discussed the design concepts of rectangular sections subjected to pure torsion. In 1983, Solanki [14] presented a simplified design approach based on the theory presented by Mitchell and Collins.

The concept of the space truss analogy is based on the assumption that the torsional capacity of a reinforced concrete rectangular section is derived from the reinforcement and the concrete surrounding the steel only. In this case, a thin-walled section is assumed to act as a space truss

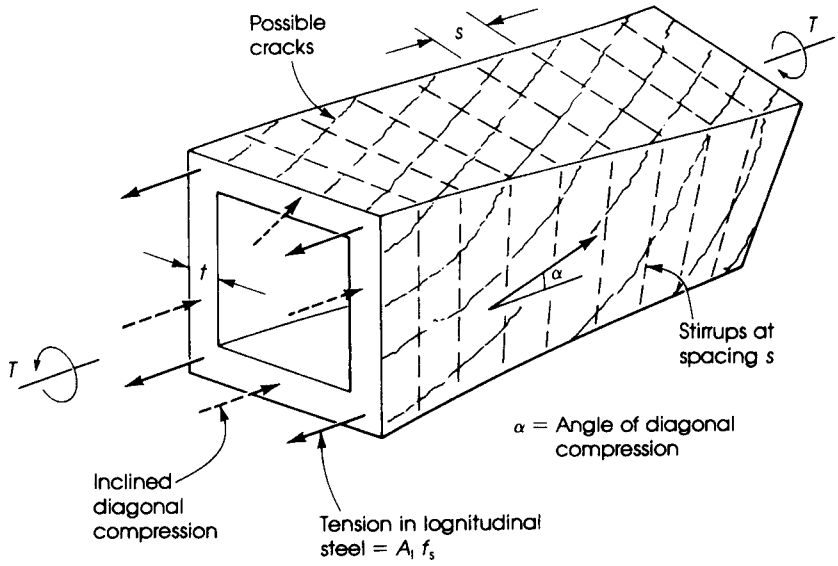


Figure 15.8 Forces on section in torsion (space truss analogy).

(Fig. 15.8). The inclined spiral concrete strips between cracks resist the compressive forces, whereas the longitudinal bars at the corners and stirrups resist the tensile forces produced by the torsional moment.

The behavior of a reinforced concrete beam subjected to pure torsion can be represented by an idealized graph relating the torque to the angle of twist, as shown in Fig. 15.9. It can be seen that prior to cracking, the concrete resists the torsional stresses and the steel is virtually unstressed. After cracking, the elastic behavior of the beam is not applicable, and hence a sudden

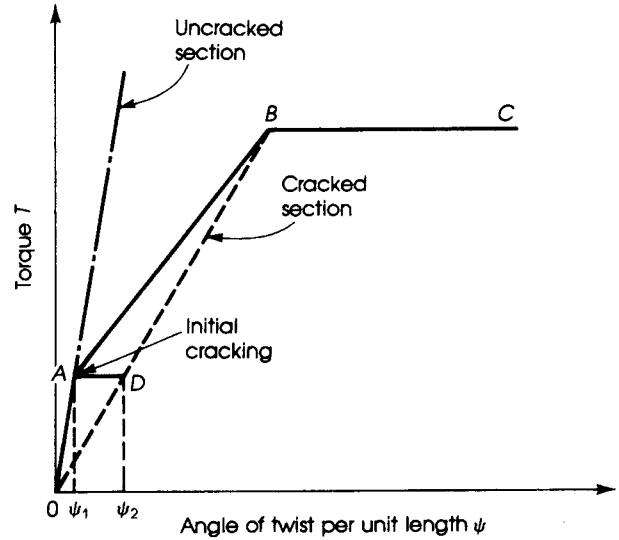


Figure 15.9 Idealized torque versus twist relationship.

change in the angle of twist occurs, which continues to increase until the maximum torsional capacity is reached. An approximate evaluation of the torsional capacity of a cracked section may be expressed as follows:

$$T_n = 2\left(\frac{A_t f_y}{s}\right) x_1 y_1 \tag{15.11}$$

where

A_t = of one leg of stirrups
s = spacing of stirrups

x_1 and y_1 = short and long distances, center to center of closed rectangular stirrups or corner bars

The preceding expression neglects the torsional capacity due to concrete. Mitchell and Collins [12] presented the following expression to evaluate the angle of twist per unit length ψ:

$$\psi = \left(\frac{P_0}{2A_0}\right)\left[\left(\frac{\varepsilon_1}{\tan\alpha}\right) + \left(\frac{P_h(\varepsilon_h \tan\alpha)}{P_0}\right) + \frac{2\varepsilon_d}{\sin\alpha}\right] \tag{15.12}$$

where

ε_1 = strain in the longitudinal reinforcing steel
ε_h = strain in the hoop steel (stirrups)
ε_d = concrete diagonal strain at the position of the resultant shear flow
P_h = hoop centerline perimeter
α = angle of diagonal compression = $(\varepsilon_d + \varepsilon_1)\Big/\left[\varepsilon_d + \varepsilon_h\left(\frac{P_h}{P_0}\right)\right]$
A_0 = area enclosed by shear, or
= torque/$2q$ where q = shear flow)
P_0 = perimeter of the shear flow path (perimeter of A_0).

The preceding twist expression is analogous to the curvature expression in flexure (Fig. 15.10):

$$\phi = \text{curvature} = \frac{\varepsilon_c + \varepsilon_s}{d_t} \tag{15.13}$$

where ε_c and ε_s are the strains in concrete and steel, respectively. A simple equation is presented by Solanki [14] to determine the torsional capacity of a reinforced concrete beam in pure torsion, based on the space truss analogy, as follows:

$$T_u = (2A_0)\left[\left(\frac{\sum A_s f_{sy}}{P_0}\right) \times \left(\frac{A_h f_{\text{hy}}}{s}\right)\right]^{1/2} \tag{15.14}$$

where A_0,P_0, and s are as explained before and

$\Sigma A_s f_{\text{sy}}$ = yield force of all the longitudinal steel bars
$A_h f_{\text{hy}}$ = yield force of the stirrups

The ACI Code adapted this theory to design concrete structural members subjected to torsion or shear and torsion in a simplified approach.

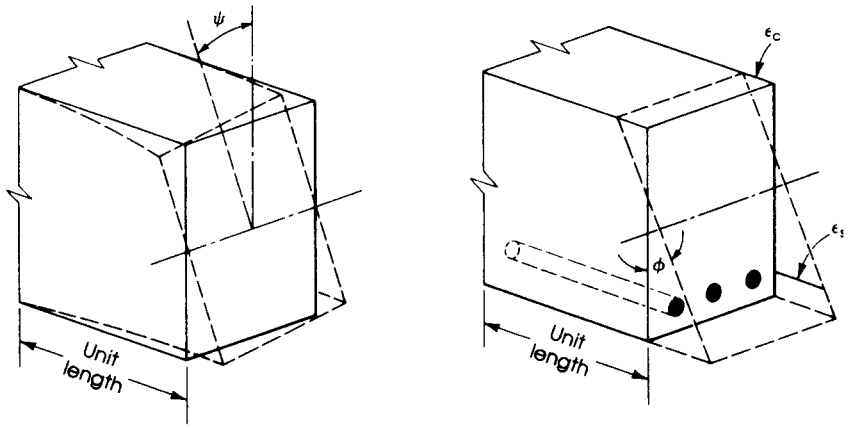


Figure 15.10 (*a*) Torsion and (*b*) flexure.

15.7 TORSIONAL STRENGTH OF PLAIN CONCRETE MEMBERS

Concrete structural members subjected to torsion will normally be reinforced with special torsional reinforcement. In case that the torsional stresses are relatively low and need to be calculated for plain concrete members, the shear stress, v_{tc}, can be estimated using Eq. 15.6:

$$v_{tc} = \frac{3T}{\phi \Sigma x^2 y} \leq 6\sqrt{f'_c}$$

and the angle of twist is $\theta = 3TL/x^3yG$, where T is the torque applied on the section (less than the cracking torsional moment) and G is the shear modulus and can be assumed to be equal to 0.45 times the modulus of elastic of concrete, E_c; that is, $G = 25{,}700\sqrt{f'_c}$. The torsional cracking shear, v_c, in plain concrete may be assumed equal to $6\sqrt{f'_c}$. Therefore, for plain concrete rectangular sections,

$$T_c = 2\phi x^2 y \sqrt{f'_c} \tag{15.15}$$

and for compound rectangular sections,

$$T_c = 2\phi \sqrt{f'_c} \Sigma x^2 y \tag{15.16}$$

15.8 TORSION IN REINFORCED CONCRETE MEMBERS (ACI CODE PROCEDURE)

15.8.1 General

The design procedure for torsion is similar to that for flexural shear. When the factored torsional moment applied on a section exceeds that which the concrete can resist, torsional cracks develop, and consequently torsional reinforcement in the form of closed stirrups or hoop reinforcement must be provided. In addition to the closed stirrups, longitudinal steel bars are provided in the corners of the stirrups and are well distributed around the section. Both types of reinforcement, closed stirrups and longitudinal bars, are essential to resist the diagonal tension forces caused by

torsion; one type will not be effective without the other. The stirrups must be closed, because torsional stresses occur on all faces of the section.

The reinforcement required for torsion must be added to that required for shear, bending moment, and axial forces. The reinforcement required for torsion must be provided such that the torsional moment strength of the section ϕT_n is equal to or exceeds the applied factored torsional moment T_u computed from factored loads.

$$\phi T_n \geq T_u \tag{15.17}$$

When torsional reinforcement is required, the torsional moment strength ϕT_n must be calculated assuming that all the applied torque, T_u, is to be resisted by stirrups and longitudinal bars with concrete torsional strength, $T_c = 0$. At the same time, the shear resisted by concrete, v_c, is assumed to remain unchanged by the presence of torsion.

15.8.2 Torsional Geometric Parameters

In the ACI Code, Section 11.5 the design for torsion is based on the space truss analogy, as shown in Fig. 15.8. After torsional cracking occurs, the torque is resisted by closed stirrups, longitudinal bars, and concrete compression diagonals. The concrete shell outside the stirrups becomes relatively ineffective and is normally neglected in design. The area enclosed by the centerline of the outermost closed stirrups is denoted by A_{oh}, the shaded area in Fig. 15.11. Because other terms are used in the design equations, they are introduced here first to make the equation easier to comprehend. Referring to Fig. 15.11, the given terms are defined as follows:

A_{cp} = enclosed by outside perimeter of concrete section, in.^{2}
P_{cp} = perimeter of concrete gross area, A_{cp}, in.
A_{oh} = area enclosed by centerline of the outermost closed transverse torsional reinforcement, in.^{2} (shaded area in Fig. 15.11)
A_0 = gross area enclosed by shear flow path and may be taken equal to 0.85 A_{oh} (A_0 may also be determined from analysis [18,19].)
P_h = perimeter of concrete of outermost closed transverse torsional reinforcement
θ = angle of compression diagonals between 30° and 60° (may be taken equal to 45° for reinforced concrete members)

In T- and L-sections, the effective overhang width of the flange on one side is limited to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness (ACI Code, Sections 11.5.1 and 13.2.4).

15.8.3 Cracking Torsional Moment, T_{cr}

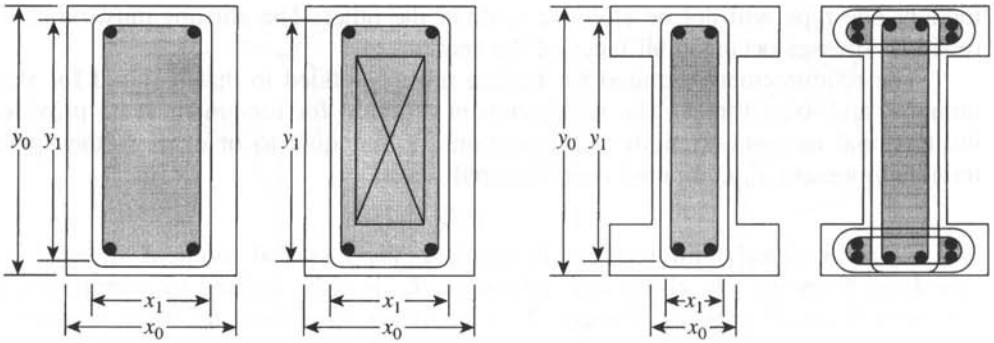
The cracking moment under pure torsion, T_{cr}, may be derived by replacing the actual section, prior to cracking, with an equivalent thin-walled tube, $t = 0.75\ A_{cp}/P_{cp}$, and an area enclosed by the wall centerline, $A_0 = 2\ A_{cp}/3$. When the maximum tensile stress (principal stress) reaches $4\lambda\sqrt{f'_c}$, cracks start to occur and the torque T in general is equal to

$$T = 2A_0\tau t \tag{15.18}$$

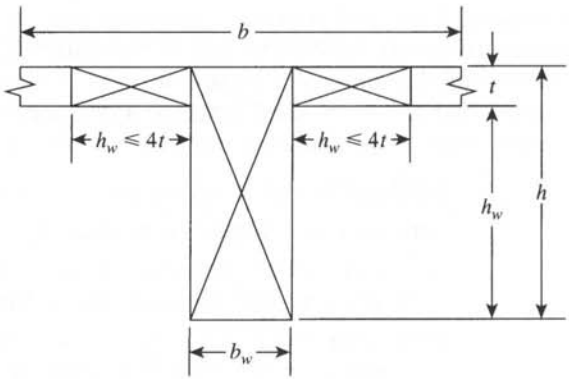
where τ = the torsional shear stress = $4\lambda\sqrt{f'_c}$ for torsional cracking.

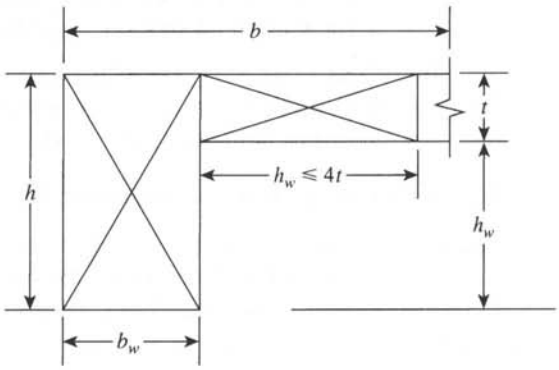
Replacing τ by $4\lambda\sqrt{f'_c}$,

$$T_{cr} = 4\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) = T_n \quad \text{and} \quad T_u = \phi T_{cr} \tag{15.19}$$



(a) $A_{cp} = x_0 y_0$, $A_{oh} = x_1 y_1$ = Shaded area to center of stirrups





(b)

Figure 15.11 (*a*) Torsional geometric parameters; (*b*) effective flange width for T- and L-sections and component rectangles.

Assuming that a torque less than or equal to $T_{cr}/4$ will not cause a significant reduction in the flexural or shear strength in a structural member, the ACI Code, Section 11.5.1, permits neglect of torsion effects in reinforced concrete members when the factored torsional moment $T_u \leq \phi T_{cr}/4$, or

$$T_u \leq \phi\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) = T_a \tag{15.20}$$

When T_u exceeds the value in Eq. 15.20, all T_u must be resisted by closed-stirrup and longitudinal bars. The torque, T_u, is calculated at a section located at distance d from the face of the support and $T_u = \phi T_n$, where $\phi = 0.75$.

Example 15.2

For the three sections shown in Fig. 15.12, and based on the ACI Code limitations, it is required to compute the following:

a. The cracking moment ϕT_{cr}

b. The maximum factored torque ϕT_n that can be applied to each section without using torsional web reinforcement

Assume $f'_c = 4$ ksi, $f_y = 60$ ksi, a 1.5-in. concrete cover, and no. 4 stirrups.

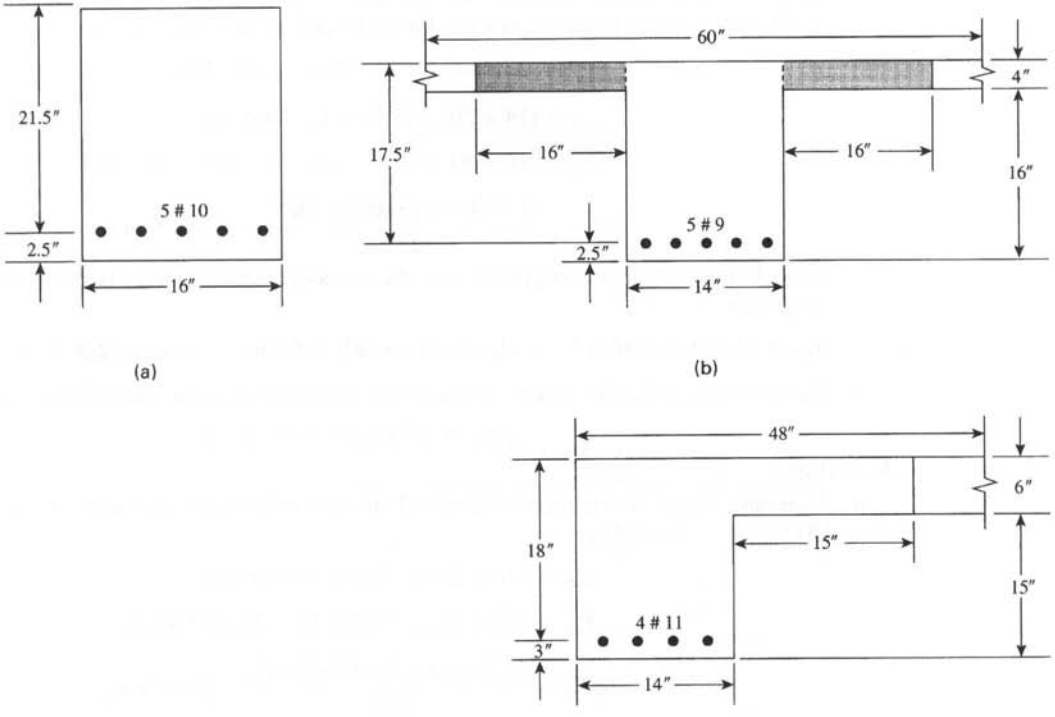


Figure 15.12 Example 15.2.

Solution

1. Section 1
   a. Cracking moment, ϕT_{cr}, can be calculated from Eq. 15.19.

$$\phi T_{cr} = \phi 4\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$$

   For this section, $A_{cp} = x_0 y_0$, the gross area of the section, where $x_0 = 16$ in. and $y_0 = 24$ in.

$$A_{cp} = 16(24) = 384 \text{ in.}^2$$

$$P_{cp} = \text{perimeter of the gross section}$$

$$= 2(x_0 + y_0) = 2(16 + 24) = 80 \text{ in.}$$

$$\phi T_{cr} = \frac{0.75(4)(1)\sqrt{4000}(384)^2}{80} = 349.7 \text{ K·in.}$$

   b. The allowable ϕT_n that can be applied without using torsional reinforcement is computed from Eq. 15.20:

$$T_a = \frac{\phi T_r}{4} = \frac{349.7}{4} = 87.4 \text{ K·in.}$$

2. Section 2
   a. First calculate A_{cp} and P_{cp} for this section and apply Eq. 15.19 to calculate ϕT_{cr}. Assuming flanges are confined with closed stirrups, the effective flange part to be used on each side of the web is equal to four times the flange thickness, or $4(4) = 16$ in $= h_w = 16$ in.

$$A_{cp} = \text{web area } (b_w h) + \text{area of effective flanges}$$

$$= (14 \times 20) + 2(16 \times 4) = 408 \text{ in.}^2$$

$$P_{cp} = 2(b + h) = 2(14 + 2 \times 16 + 20) = 132 \text{ in.}^2$$

$$\phi T_{cr} = \frac{0.75(4)(1)\sqrt{(4000)}(408)^2}{132} = 239.3 \text{ K·in.}$$

   Note: If the flanges are neglected and the torsional reinforcement is confined in the web only, then

$$A_{cp} = 14(20) = 280 \text{ in.}^2 \qquad P_{cp} = 2(14 + 20) = 68 \text{ in.} \qquad \phi T_{cr} = 219 \text{ K·in.}$$

   b. The allowable ϕT_n that can be applied without using torsional reinforcement is

$$\phi T_{cr}/4 = 239.3/4 = 59.8 \text{ K·in.}$$

3. Section 3
   a. Assuming flange is confined with closed stirrups, effective flange width is equal to $b_w = 15$ in. $< 4 \times 6 = 24$ in.

$$A_{cp} = (14 \times 21) + (15 \times 6) = 384 \text{ in.}^2$$

$$P_{cp} = 2(b + h) = 2(14 + 15 + 21) = 100 \text{ in.}$$

$$\phi T_{cr} = \frac{0.75(4)(1)\sqrt{(4000)}(384)^2}{100} = 279.8 \text{ K·in.}$$

   Note: If the flanges are neglected, then $A_{cp} = 294$ in.^{2}, $P_{cp} = 70$ in., and $\phi T_{cr} = 234.3$ K·in.

   b. The allowable $\phi T_n = \phi T_{cr}/4 = 279.8/4 = 70$ K·in.

15.8.4 Equilibrium Torsion and Compatibility Torsion

Structural analysis of concrete members gives the different forces acting on the member, such as normal forces, bending moments, shear forces, and torsional moments, as explained in the simple problem of Example 15.1. The design of a concrete member is based on failure of the member under factored loads. In statically indeterminate members, a redistribution of moments occurs before failure; consequently, design moments may be reduced, whereas in statically determinate members, such as a simple beam or a cantilever beam, no moment redistribution occurs.

In the design of structural members subjected to torsional moments two possible cases may apply after cracking.

1. The *equilibrium torsion case* occurs when the torsional moment is required for the structure to be in equilibrium and T_u cannot be reduced by redistribution of moments, as in the case of simple beams. In this case torsion reinforcement must be provided to resist all of T_u. Figure 15.13 shows an edge beam supporting a cantilever slab where no redistribution of moments will occur [18,19].
2. The *compatibility torsion case* occurs when the torsional moment, T_u, can be reduced by the redistribution of internal forces after cracking while compatibility of deformation is maintained in the structural member. Figure 15.14 shows an example of this case, where two transverse beams are acting on an edge beam producing twisting moments. At torsional

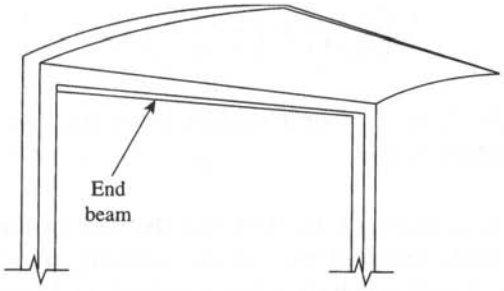


Figure 15.13 Design torque may not be reduced. Moment redistribution is not possible [19].

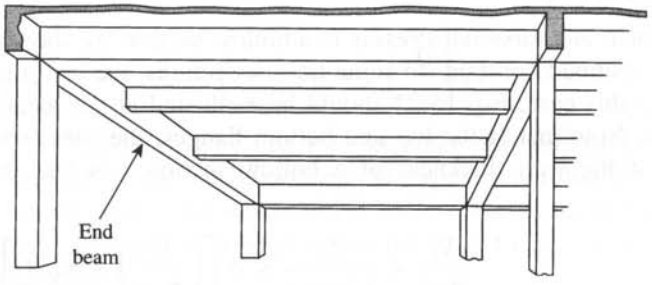


Figure 15.14 Design torque may be reduced in a spandrel beam. Moment redistribution is possible [19].

cracking, a large twist occurs, resulting in a large distribution of forces in the structure [18,19]. The cracking torque, T_{cr}, under combined flexure, shear, and torsion is reached when the principal stress in concrete is about $4\lambda\sqrt{f'_c}$. When $T_u > T_{cr}$, a torque equal to T_{cr} (Eq. 15.19), may be assumed to occur at the critical sections near the faces of the supports.

The ACI Code limits the design torque to the smaller of T_u from factored loads or ϕT_{cr} from Eq. 15.19.

15.8.5 Limitation of Torsional Moment Strength

The ACI Code, Section 11.5.3, limits the size of the cross-sectional dimension by the following two equations:

1. For solid sections,

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2} \le \phi\left[\left(\frac{V_c}{b_w d}\right) + 8\sqrt{f'_c}\right] \tag{15.21}$$

2. For hollow sections,

$$\left(\frac{V_u}{b_w d}\right) + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right) \le \phi\left[\left(\frac{V_c}{b_w d}\right) + 8\sqrt{f'_c}\right] \tag{15.22}$$

where $V_c = 2\lambda\sqrt{f'_c}b_w d$ = shear strength for normal-weight concrete. All other terms were defined in Section 15.8.2.

This limitation is based on the fact that the sum of the stresses due to shear and torsion (on the left-hand side) may not exceed the cracking stress plus $8\sqrt{f'_c}$. The same condition was applied to the design of shear without torsion in Chapter 8. The limitation is needed to reduce cracking and to prevent crushing of the concrete surface due to inclined shear and torsion stresses.

15.8.6 Hollow Sections

Combined shear and torsional stresses in a hollow section are shown in Fig. 15.6, where the wall thickness is assumed constant. In some hollow sections, the wall thickness may vary around the perimeter. In this case, Eq. 15.22 should be evaluated at the location where the left-hand side is maximum. Note that at the top and bottom flanges, the shear stresses are usually negligible. In general, if the wall thickness of a hollow section t is less than A_{oh}/P_h, then Eq. 15.22 becomes

$$\frac{V_u}{b_w d} + \frac{T_u}{1.7A_{oh}t} \le \phi\left[\left(\frac{V_c}{b_w d}\right) + 8\sqrt{f'_c}\right] \tag{15.23}$$

(ACI Code, Section 11.5.3).

15.8.7 Web Reinforcement

As was explained earlier, the ACI Code approach for the design of the members due to torsion is based on the space truss analogy in Fig. 15.8. After torsional cracking, two types of reinforcement are required to resist the applied torque, T_u: transverse reinforcement, A_t, in the form of closed stirrups, and longitudinal reinforcement, A_l, in the form of longitudinal bars. The ACI Code, Section 11.5.3, presented the following expression to compute A_t and A_l:

1. Closed stirrups A_t can be calculated as follows.

$$T_n = \frac{2A_0 A_t f_{yt} \cot\theta}{s} \tag{15.24}$$

where

$T_n = \dfrac{T_u}{\theta}$ and $\phi = 0.75$
A_t = area of one leg of the transverse closed stirrups
f_{yt} = yield strength of $A_t \leq 60$ ksi
s = spacing of stirrups

A_0 and θ were defined in Section 15.8.2. Equation 15.24 can be written as follows:

$$\frac{A_t}{s} = \frac{T_n}{2A_0 f_{yt} \cot\theta} \tag{15.25}$$

If $\theta = 45^\circ$, then $\cot\theta = 1.0$, and if $f_{yv} = 60$ ksi, then Eq. 15.25 becomes

$$\frac{A_t}{s} = \frac{T_n}{120A_0} \tag{15.26}$$

where T_n is in kip in. Spacing of stirrups, s, should not exceed the smaller of $P_h/8$ or 12 in. For hollow sections in torsion, the distance measured from the centerline of stirrups to the inside face of the wall shall not be less than $0.5\, A_{oh}/P_h$.

2. The additional longitudinal reinforcement, A_l, required for torsion should not be less than the following:

$$A_l = \left(\frac{A_t}{s}\right) P_h \left(\frac{f_{yt}}{f_y}\right) \cot^2\theta \tag{15.27}$$

If $\theta = 45^\circ$ and $f_{yt} = f_y = 60$ ksi for both stirrups and longitudinal bars, then Eq. 15.27 becomes

$$A_l = \left(\frac{A_t}{s}\right) P_h = 2\left(\frac{A_t}{s}\right)(x_1 + y_1) \tag{15.28}$$

P_h was defined in Section 15.8.2. Note that reinforcement required for torsion should be added to that required for the shear, moment, and axial force that act in combination with torsion. Other limitations for the longitudinal reinforcement, A_l, are as follows:

a. The smallest bar diameter of a longitudinal bar is that of no. 3 or stirrup spacing $s/24$, whichever is greater.

b. The longitudinal bars should be distributed around the perimeter of the closed stirrups with a maximum spacing of 12 in.

c. The longitudinal bars must be inside the stirrups with at least one bar in each corner of the stirrups. Corner bars are found to be effective in developing torsional strength and in controlling cracking.

d. Torsional reinforcement should be provided for a distance $(b_t + d)$ beyond the point theoretically required, where b_t is the width of that part of the cross-section containing the stirrups resisting torsion.

15.8.8 Minimum Torsional Reinforcement

Where torsional reinforcement is required, the minimum torsional reinforcement may be computed as follows (ACI Code, Section 11.5.5):

1. Minimum transverse closed stirrups for combined shear and torsion (see Section 8.6):

$$A_v + 2A_t \geq \frac{50 b_w s}{f_{yt}} \qquad \text{(for } f'_c < 4.5 \text{ ksi)}$$

$$\geq 0.75\sqrt{f'_c}\left(\frac{b_w s}{f_{yt}}\right) \qquad \text{(for } f'_c \geq 4.5 \text{ ksi)} \tag{15.29}$$

where

A_v = area of two legs of a closed stirrup determined from shear
A_t = area of one leg of closed stirrup determined from torsion
s = spacing of stirrups
f_{yt} = yield strength of closed stirrups $\leq$ 60 ksi

Spacing of stirrups, s, should not exceed $P_h/8$ or 12 in., whichever is smaller. This spacing is needed to control cracking width.

2. Minimum total area of longitudinal torsional reinforcement:

$$A_{l\,\min} = \left(\frac{5\sqrt{f'_c}A_{cp}}{f_y}\right) - \left(\frac{A_t}{s}\right) P_h \left(\frac{f_{yt}}{f_y}\right) \tag{15.30}$$

where A_t/s shall not be taken less than $25\, b_w/f_{yt}$.

The minimum A_l in Eq. 15.30 was determined to provide a minimum ratio of the volume of torsional reinforcement to the volume of concrete of about 1% for reinforced concrete subjected to pure torsion.

15.9 SUMMARY OF ACI CODE PROCEDURES

The design procedure for combined shear and torsion can be summarized as follows:

1. Calculate the factored shearing force, V_u, and the factored torsional moment, T_u, from the applied forces on the structural member. Critical values for shear and torsion are at a section distance d from the face of the support.
2. a. Shear reinforcement is needed when $V_u > \phi V_c/2$, where $V_c = 2\lambda\sqrt{f'_c}b_w d$.

b. Torsional reinforcement is needed when

$$T_u > \phi\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) \tag{15.20}$$

If web reinforcement is needed, proceed as follows.

3. Design for shear:

a. Calculate the nominal shearing strength provided by the concrete, V_c. Determine the shear to be carried by web reinforcement:

$$V_u = \phi V_c + V_s \text{ or } V_s = \frac{V_u - \phi V_c}{\phi}$$

b. Compare the calculated V_s with maximum permited value of $(8\sqrt{f'_c}b_w d)$ according to the ACI Code Section 11.4.7.9. If calculated V_s is less, proceed with the design; if not, increase the dimensions of the concrete section.

c. The shear web reinforcement is calculated as follows:

$$A_v = \frac{V_s s}{f_{yt} d}$$

where

A_v = area of two legs of the stirrup
s = spacing of stirrups.

The shear reinforcement per unit length of beam is

$$\frac{A_v}{s} = \frac{V_s}{f_{yt} d}$$

d. Check A_v/s calculated with the minimum A_v/s:

$$(\min)\frac{A_v}{s} = 0.75\sqrt{f'_c}\left(\frac{b_w}{f_{yt}}\right) \geq 50\left(\frac{b_w}{f_{yt}}\right)$$

The minimum A_v, specified by the code under the combined action of shear and torsion, is given in step 5.

4. Design for torsion:

a. Check if the factored torsional moment, T_u, causes equilibrium or compatibility torsion. For equilibrium torsion, use T_u. For compatibility torsion, the design torsional moment is the smaller of T_u from factored load and

$$T_{u2} = \phi 4\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) \tag{15.19}$$

b. Check that the size of the section is adequate. This is achieved by checking either Eq. 15.21 for solid sections or Eq. 15.22 for hollow sections. If the left-hand-side value is greater than $\phi(V_c/b_w d + 8\sqrt{f'_c})$, then increase the cross-section. If it is less than that value, proceed. For hollow sections, check if the wall thickness t is less than A_{oh}/P_h. If it is less, use Eq. 15.23 instead of Eq. 15.22; otherwise, use Eq. 15.22.

c. Determine the closed stirrups required from Eq. 15.25:

$$\frac{A_t}{s} = \frac{T_n}{2A_0 f_{yt} \cot\theta} \tag{15.25}$$

A_t/s should not be less than 25 b_w/f_{yt}. Also, the angle θ may be assumed to be 45°, $T_n = T_u/\phi$, and $\phi = 0.75$.

Assume $A_0 = 0.85\ A_{oh} = 0.85\ (x_1\ y_1)$, where x_1 and y_1 are the width and depth of the section to the centerline of stirrups; see Fig. 15.11. Values of A_0 and θ may be obtained from analysis [18]. For $\theta = 45°$ and $f_y = 60$ ksi,

$$\frac{A_t}{s} = \frac{T_n}{120A_0} \tag{15.26}$$

The maximum allowable spacing, s, is the smaller of 12 in. or $P_h/8$.

d. Determine the additional longitudinal reinforcement:

$$A_l = \left(\frac{A_t}{s}\right) P_h \left(\frac{f_{yt}}{f_y}\right) \cot^2\theta \tag{15.27}$$

but not less than

$$A_{l\,\min} = \left(\frac{5\sqrt{f'_c}A_{cp}}{f_y}\right) - \left(\frac{A_t}{s}\right) P_h \left(\frac{f_{yt}}{f_y}\right) \tag{15.30}$$

For $\theta = 45°$ and $f_{yt} = 60$ ksi, then $A_l = (A_t/s)P_h$. (15.28)

Bars should have a diameter of at least stirrup spacing, $s/24$, but not less than no. 3 bars. The longitudinal bars should be placed inside the closed stirrups with maximum spacing of 12 in. At least one bar should be placed at each corner of stirrups. Normally, one-third of A_l is added to the tension reinforcement, one-third at midheight of the section, and one-third at the compression side.

5. Determine the total area of closed stirrups due to V_u and T_u.

$$A_{vt} = (A_v + 2A_t) \geq \frac{50\ b_w s}{f_{yt}} \tag{15.29}$$

Choose proper closed stirrups with a spacing s as the smaller of 12 in. or $P_h/8$.

The stirrups should be extended a distance $(b_t + d)$ beyond the point theoretically no longer required, where b_t = width of cross-section resisting torsion.

Example 15.3: (Equilibrium Torsion)

Determine the necessary web reinforcement for the rectangular section shown in Fig. 15.15. The section is subjected to an factored shear $V_u = 48$ K and an equilibrium torsion $T_u = 360$ K·in at a section located at a distance d from the face of the support. Use normal-weight concrete with $f'_c = 4$ ksi, and $f_y = 60$ ksi.

Solution

The following steps explain the design procedure:

1. Design forces are $V_u = 48$ K and an equilibrium torsion $T_u = 400$ K·in.
2. **a.** Shear reinforcement is needed when $V_u > \phi V_c/2$.

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}bd = 0.75(2)(1)\sqrt{4000}(16)(20.5) = 31.1\text{ K}$$

$$V_u = 48\text{ K} > \frac{\phi V_c}{2} = 15.55\text{ K}$$

Shear reinforcement is required.

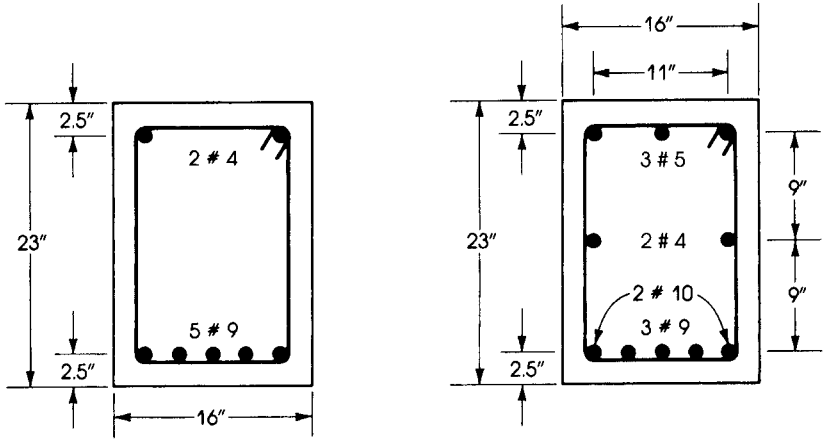


Figure 15.15 Example 15.3.

b. Torsional reinforcement is needed when

$$T_u > \phi\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) = T_a$$

$$A_{cp} = x_0 y_0 = 16(23) = 368 \text{ in.}^2$$

$$P_{cp} = 2(x_0 + y_0) = 2(16 + 23) = 78 \text{ in.} \qquad (15.20)$$

$$T_a = \frac{0.75(1)\sqrt{4000}(368)^2}{78} = 82.36 \text{ K·in.}$$

$$T_u = 360 \text{ K·in.} > 82.36 \text{ K·in.}$$

Torsional reinforcement is needed. Note that if T_u is less than 82.36 K·in., torsional reinforcement is not required, but shear reinforcement may be required.

3. Design for shear:

a. $V_u = \phi V_c + \phi V_s$, $\phi V_c = 35.26$ K. $48 = 31.1 + 0.75V_s$, $V_s = 22.5$ K

b. Maximum $V_s = 8\sqrt{f'_c}bd = 8\sqrt{4000}(16)(20.5) = 166 \text{ K} > V_s$.

c. $A_v/s = V_s/f_y d = 22.5/(60 \times 20.5) = 0.018$ in.^{2}/in. (two legs)

$A_v/2s = 0.018/2 = 0.009$ in.^{2}/in. (one leg)

4. Design for torsion:

a. Design $T_u = 360$ K·in. Determine sectional properties, assuming 1.5-in. concrete cover and no. 4 stirrups:

x_1 = width to center of stirrups $= 16 - 2(1.5 + 0.25) = 12.5$ in.

y_1 = depth to center of stirrups $= 23 - 2(1.5 + 0.25) = 19.5$ in.

Practically, x_1 can be assumed to be $b - 3.5$ in. and $y_1 = h - 3.5$ in.

$$A_{oh} = x_1 y_1 = (12.5 \times 19.5) = 244 \text{ in.}^2$$

$$A_0 = 0.85A_{oh} = 207.2 \text{ in.}^2$$

$$P_h = 2(x_1 + y_1) = 2(12.5 + 19.5) = 64 \text{ in.}$$

For $\theta = 45^\circ$ and $\cot\theta = 1.0$.

b. Check the adequacy of the size of the section using Eq. 15.21:

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2} \le \phi\left[\left(\frac{V_c}{b_w d}\right) + 8\sqrt{f_c'}\right]$$

$$\phi V_c = 31.1 \text{ K} \quad \text{and} \quad V_c = 41.5 \text{ K}$$

$$\text{Left-hand side} = \sqrt{\left(\frac{48{,}000}{16 \times 20.5}\right)^2 + \left(\frac{360{,}000 \times 64}{1.7(244)^2}\right)^2} = 271 \text{ psi}$$

$$\text{Right-hand side} = 0.75\left(\frac{41{,}500}{16 \times 20.5} + 8\sqrt{4000}\right) = 475 \text{ psi} > 271 \text{ psi}$$

The section is adequate.

c. Determine the required closed stirrups due to torsion from Eq. 15.25:

$$\frac{A_t}{s} = \frac{T_n}{2A_0 f_{yt} \cot\phi}$$

$$T_n = \frac{T_u}{\phi} = \frac{360}{0.75} = 480 \text{ K·in.} \quad \cot\theta = 1.0 \quad \text{and} \quad A_0 = 207.2 \text{ in.}^2$$

$$\frac{A_t}{s} = \frac{480}{2 \times 207.2 \times 60} = 0.019 \text{ in.}^2/\text{in.} \quad \text{(per one leg)}$$

d. Determine the additional longitudinal reinforcement from Eq. 15.27:

$$A_l = \left(\frac{A_t}{s}\right) P_h \left(\frac{f_{yt}}{f_y}\right) \cot^2\theta$$

$$\frac{A_t}{s} = 0.019, \quad P_h = 64 \text{ in.} \quad f_{yt} = f_y = 60 \text{ ksi} \quad \cot\theta = 1.0$$

$$A_l = 0.019(64) = 1.21 \text{ in.}^2$$

$$\text{Min. } A_l = 5\sqrt{f_c'} A_{cp}/f_y - \left(\frac{A_t}{s}\right) P_h \left(\frac{f_{yt}}{f_y}\right)$$

$$A_{cp} = 368 \text{ in.}^2 \quad \frac{A_t}{s} = 0.019$$

$$f_{yt} = f_{yl} = 60 \text{ Ksi}$$

$$\text{Min. } A_l = \left[\frac{5\sqrt{4000}(368)}{60{,}000}\right] - (0.019 \times 64 \times 1.0) = 0.72 \text{ in.}^2$$

$$A_l = 1.21 \text{ in.}^2 \text{ controls}$$

5. Determine total area of closed stirrups:

a. For one leg of stirrups, $A_{vt}/s = A_t/s + A_v/2s$.

$$\text{Required } A_{vt} = \frac{0.018}{2} + 0.019 = 0.028 \text{ in.}^2/\text{in.} \quad \text{(per one leg)}$$

Using no. 4 stirrups, area of one leg is 0.2 in.^{2}

$$\text{Spacing of stirrups} = \frac{0.2}{0.028} = 7.14 \text{ in.} \quad \text{or} \quad 7.0 \text{ in.}$$

b. Maximum $s = P_h/8 = \frac{64}{8} = 8$ in. or 12 in., whichever is smaller. The value of s used is 7.0 in. < 8 in.

c. Minimum $A_{vt}/s = 50b_w/f_{yt} = 50(16)/60{,}000 = 0.0133$ in.^{2}/in. This is less than 0.028 in.^{2}/in. provided.

6. To find the distribution of longitudinal bars, note that total $A_l = 1.21$ in.^{2} Use one-third at the top, or 1.21/3 = 0.4 in.^{2}, to be added to the compression steel A'_s. Use one-third, or 0.4 in.^{2}, at the bottom, to be added to the tension steel, and one-third, or 0.4 in.^{2}, at middepth.

a. The total area of top bars is 0.4(2 no. 4) + 0.4 = 0.8 in.^{2}; use three no. 5 bars (A_s = 0.91 in.^{2}).

b. The total area of bottom bars is 5 (five no. 9) + 0.4 = 5.4 in.^{2}; use three no. 9 and two no. 10 bars at the corners (total A_s = 5.53 in.^{2}).

c. At middepth, use two no. 4 bars (A_s = 0.4 in.^{2}).

Reinforcement details are shown in Fig. 15.15. Spacing of longitudinal bars is equal to 9 in., which is less than the maximum required of 12 in. The diameter of no. 4 bars used is greater than the minimum of no. 3 or stirrup spacing, or $s/24$ = 0.21 in.

Example 15.4: Compatibility Torsion

Repeat Example 15.3 if the factored torsional torque is a compatibility torsion.

Solution

Referring to the solution of Example 15.3,

1. Design forces are V_u = 48 K and compatibility torsion is 360 K·in.

2. Steps (a) and (b) are the same as in Example 15.3. Web reinforcement is required.

3. Step (c) is the same.

4. Design for torsion:
Because this is a compatibility torsion of 360 K in., the design T_u is the smaller of 360 K in. or ϕT_{cr} given in Eq. 15.19.

$$\phi T_{cr} = \phi 4\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) = \frac{0.75(4)(1)\sqrt{4000}(368)^2}{78} = 329.4 \text{ K·in.} \qquad (15.19)$$

Because ϕT_{cr} < 360 K·in., use T_u = 329.4 K·in. Repeat all the steps of Example 15.3 using T_u = 329.3 K·in. to determine that the section is adequate.

$$\frac{A_t}{s} = 0.018 \text{ in.}^2\text{/in.} \qquad \text{(one leg)}$$

$$A_l = 0.018(64) = 1.152 \text{ in.}^2$$

Use 1.2 in.^{2} > min. A_l.

5. Required $A_{vt} = 0.018/2 + 0.018 = 0.027$ in.^{2} in. (one leg).

$$s = \frac{0.2}{0.027} = 7.4 \text{ in.}$$

Use 7 in. Choose bars, stirrups, and spacing similar to Example 15.3.

Example 15.5: L-Section with Equilibrium Torsion

The edge beam of a building floor system is shown in Fig. 15.16. The section at a distance d from the force of the support is subjected to V_u = 53 K and an equilibrium torque T_u = 240 K·in. Design the necessary web reinforcement using f'_c = 4 ksi and f_y = 60 ksi for all steel bars and stirrups.

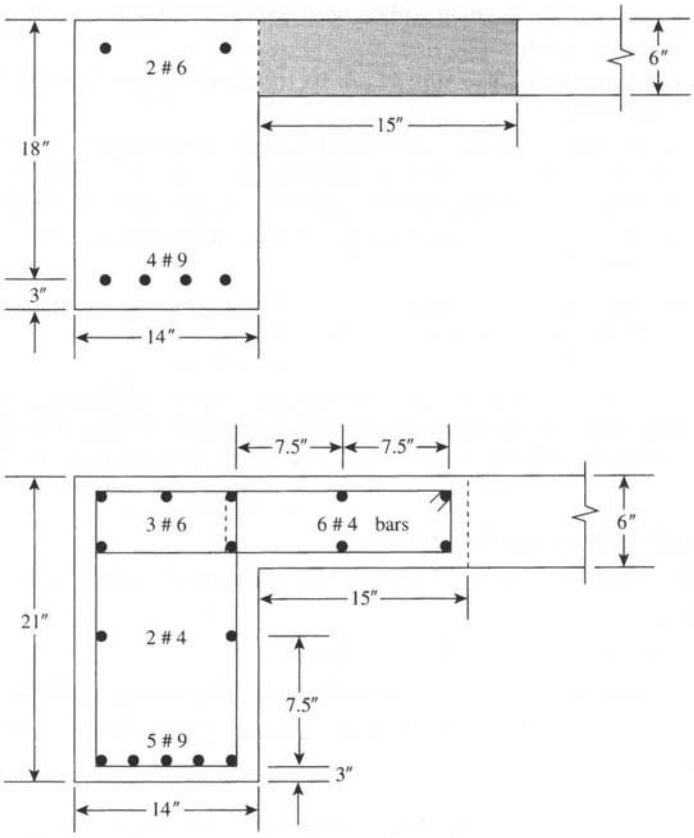


Figure 15.16 Example 15.5.

Solution

1. Design forces are $V_u = 60\,\text{K}$ and $T_u = 270\,\text{K·in.} = 22.5\,\text{K·ft.}$
2. **a.** Shear reinforcement is needed when $V_u > \phi\, V_c/2$.

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}\,b_w d = 0.75(2)(1)\sqrt{4000}(14)(18) = 23.9\,\text{K}$$

$$V_u > \frac{\phi V_c}{2} = 11.95\,\text{K}$$

Shear reinforcement is required.

b. Check if torsional reinforcement is needed. Assuming that flange is contributing to resist torsion, the effective flange length is $h_w = 15\,\text{in.} < 4 \times 6 = 24\,\text{in.}$

$$x_0 = 14\text{ in.} \quad \text{and} \quad y_0 = 21\text{ in.}$$

$$A_{cp} = (14 \times 21)(\text{web}) + (15 \times 6)(\text{flange}) = 384\text{ in.}^2$$

$$P_{cp} = 2(21 + 29) = 100\text{ in.}$$

$$T_a\ (\text{Eq. } 15.20) = \frac{0.75(1)\sqrt{4000}(384)^2}{100} = 70\,\text{K·in.}$$

$$T_u > T_a$$

Torsional reinforcement is required.

3. Design for shear:

a.

$$V_u = \phi V_c + \phi V_s$$

$$53 = 23.9 + 0.75V_s$$

$$V_s = 38.8\,\text{K}$$

b. Maximum $V_s = 8\sqrt{f'_c}b_w d = 127.5\,\text{K} > V_s$

c.

$$A_v/s = \frac{V_s}{f_y d} = \frac{38.8}{60 \times 18} = 0.036\ \text{in.}^2/\text{in.} \qquad \text{(two legs)}$$

$$A_v/2s = \frac{0.036}{2} = 0.018\ \text{in.}^2/\text{in.}$$

4. Design for torsion: $T_u = 240\,\text{K·in.}$

a. Determine section properties assuming a concrete cover of 1.5 in. and no. 4 stirrups.

Web $x_1 = b - 3.5\ \text{in.} = 14 - 3.5 = 10.5\ \text{in.}$ $\quad y_1 = h - 3.5 = 21 - 3.5 = 17.5\ \text{in.}$

Flange $x_1 = 15\ \text{in.}$ (stirrups extend to the web) $\quad y_1 = 6 - 3.5 = 2.5\ \text{in.}$

$A_{oh} = (15 \times 2.5) + (10.5 \times 17.5) = 221\ \text{in.}^2$ $\quad A_0 = 0.85A_{oh} = 188\ \text{in.}^2$

$P_h = 2(15 + 2.5) + 2(10.5 + 17.5) = 91\ \text{in.}$ $\quad \theta = 45^\circ$ $\quad \cot\theta = 1.0$

b. Check the adequacy of the section using Eq. 15.21: $V_u = 53\,\text{K}$, $\phi\ V_c = 23.9\,\text{K}$ $V_c = 31.9\,\text{K}$ $T_u = 240\,\text{K·in.}$

$$\text{left-hand side} = \sqrt{\left(\frac{53{,}000}{14 \times 18}\right)^2 + \left[\frac{240{,}000 \times 91}{1.7(184)^2}\right]^2} = 434\,\text{psi}$$

$$\text{right-hand side} = 0.75\left[\frac{31{,}900}{14 \times 18} + 8\sqrt{4000}\right] = 475\,\text{psi}$$

The section is adequate.

c. Determine the torsional closed stirrups, A_t/s, from Eq. 15.25:

$$\frac{A_t}{s} = \frac{T_n}{2A_0 f_{yt}} = \frac{240}{0.75 \times 2 \times 188 \times 60} = 0.014\ \text{in.}^2/\text{in.} \qquad \text{(for one leg)}$$

d. Calculate the additional longitudinal reinforcement from Eq. 15.28 (for $f_y = 60\,\text{ksi}$ and $\cot\theta = 1.0$):

$$A_l = \left(\frac{A_t}{s}\right) P_h = 0.014(91) = 1.28\ \text{in.}^2$$

$A_{l\,\text{min}}$ (from Eq. 15.30) is

$$A_l = \left[\frac{5\sqrt{4000}(384)}{60{,}000}\right] - (0.014 \times 91) = 0.75\ \text{in.}^2$$

The contribution of the flange may be neglected with slight difference in results, and less labor cost.

5. Determine the total area of the closed stirrups.

a. For one leg, $A_{vt}/s = A_t/s + A_v/2s$.

$$\text{Required } A_{vt} = 0.014 + 0.018 = 0.032\ \text{in.}^2/\text{in.} \quad \text{(per leg)}$$

Choose no. 4 closed stirrups, area $= 0.2\,\text{in.}^2$

$$\text{Spacing of stirrups} = \frac{0.2}{0.032} = 6.25\ \text{in.}$$

Use 6 in.

b. Max. $s = P_h/8 = 91/8 = 11.4$ in. Use $s = 6$ in., as calculated.

c. $A_{vt}/s = 50b_w/f_{yt} = 50(14)/60{,}000 = 0.017$ in.^{2}/in., which is less than the 0.032 in.^{2}/in. used. Use no. 4 closed stirrups spaced at 6 in.

6. Find the distribution of longitudinal bars. Total A_l is 1.28 in.^{2} Use one-third, or 0.43 in.^{2}, at the top, at the bottom, and at middepth.

a. Total top bars $= 0.88 + 0.43 = 1.31$ in.^{2}; use three no. 6 bars (1.32 in.^{2}).

b. Total bottom bars $= 4.0 + 0.43 = 4.43$ in.^{2}; use five no. 9 bars (5.0 in.^{2}).
Total A_l used $= (1.32 - 0.88) + (5 - 4) = 1.44$ in.^{2}

c. Use two no. 4 bars at middepth (0.40 in.^{2}). Reinforcement details are shown in Fig. 15.16. Spacing of longitudinal bars is at 7.5 in. < 12 in. The diameter of no. 4 bars used is 0.5 in., which is greater than no. 3 or stirrup spacing, $s/24 = \frac{6}{24} = 0.25$ in. Add no. 4 longitudinal bars on all corners of closed stirrups in beam web and flange.

SUMMARY

Sections 15.1–15.7

1. Torsional stresses develop in a beam when a moment acts on the beam section parallel to its surface.
2. In most practical cases, a structural member may be subjected to combined shear and torsional moments.
3. The design methods for torsion rely generally on two basic theories: the skew bending theory and the space truss theory. The ACI Code adopted the space truss theory.

Sections 15.8–15.9

A summary of the relative equations in U.S. customary units and SI units is given here.

Note that $(1.0\sqrt{f'_c})$ in psi is equivalent to $(0.08\sqrt{f'_c})$ in MPa N/mm^2, 1 in. $\approx$ 25 mm, and $f_{yt} \leq 400$ MPa.

Equation	U.S. Customary Units	SI Units
15.16	$T_c = 2\phi\sqrt{f'_c}\sum x^2 y$	$T_c = 0.17\phi\sqrt{f'_c}\sum x^2 y$
15.17	$\phi T_n \geq T_u$	Same
15.19	$T_{cr} = 4\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$	$T_{cr} = (\lambda\sqrt{f'_c}/3)(A_{cp}^2/P_{cp})$
15.20	$T_u = \phi\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$	$T_u \leq \phi\lambda(\sqrt{f'_c}/12)(A_{cp}^2/P_{cp})$
15.21	$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2} \leq \phi\left[\left(\frac{V_c}{b_w d}\right) + 8\sqrt{f'_c}\right]$	(U.S.)
	$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2} \leq \phi\left[\left(\frac{V_c}{b_w d}\right) + (2\sqrt{f'_c}/3)\right]$	(SI)
15.24	$T_n = \frac{2A_0 A_t f_{yt}\cot\theta}{s}$	Same

(Note that f_{yt} is in MPa, S is in mm, A_0 and A_t are in mm^2, and T_n is in kN m.)

Equation	U.S. Customary Units	SI Units
15.25	$\frac{A_t}{s} = \frac{T_n}{2A_0 f_{yt} \cot\theta}$	Same
15.27	$A_t = \frac{A_t P_h (f_{yt}/f_y) \cot^2\theta}{s}$	Same
15.29	$A_v + 2A_t \geq \frac{50 b_w s}{f_{yt}}$	$(A_v + 2A_t) \geq 0.35 b_w s / f_{yt}$
15.30	$A_{l\,\min} = \left[\frac{5\sqrt{f'_c} A_{cp}}{f_y}\right] - (A_t/s) P_h (f_{yt}/f_y)$	$A_{l\,\min} = [(5\sqrt{f'_c} A_{cp})/12 f_y] - \left(\frac{A_t}{s}\right) P_h \left(\frac{f_{yt}}{f_y}\right)$

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PROBLEMS

For each problem, compute the cracking moment ϕT_{cr} and the maximum factored torque ϕT_n that can be applied without using torsional web reinforcement. Use $f'_c = 4$ksi and $f_y = 60$ ksi.

15.1 A rectangular section with $b = 16$ in. and $h = 24$ in.

15.2 A rectangular section with $b = 12$ in. and $h = 20$ in.

15.3 A T-section with $b = 48$ in., $b_w = 12$ in., $t = 4$ in., and $h = 25$ in. Assume flanges are confined with closed stirrups.

15.4 A T-section with $b = 60$ in., $b_w = 16$ in., $t = 4$ in., and $h = 30$ in. Assume flanges are confined with closed stirrups.

15.5 An inverted L-section with $b = 32$ in., $b_w = 14$ in., $t = 6$ in., and $h = 24$ in. The flange does not have closed stirrups.

15.6 An inverted L-section with $b = 40$ in., $b_w = 12$ in., $t = 6$ in., and $h = 30$ in. The flange contains confined closed stirrups.

15.7 Determine the necessary web reinforcement for a simple beam subjected to an equilibrium factored torque $T_u = 220$ K·in. and $V_u = 36$ K. The beam section has $b = 14$ in., $h = 22$ in., and $d = 19.5$ in., and is reinforced on the tension side by four no. 9 bars. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

15.8 Repeat Problem 15.7 using $f'_c = 5$ ksi and $f_y = 60$ ksi.

15.9 The section of an edge (spandrel) beam is shown in Fig. 15.17. The critical section of the beam is subjected to an equilibrium torque $T_u = 300$ K·in and a shear $V_u = 60$ K. Determine the necessary web reinforcement using $f'_c = 4$ ksi and $f_y = 60$ ksi. Consider that the flange is not reinforced with closed stirrups.

15.10 Repeat Problem 15.9. Considering that the flange is effective and contains closed stirrups.

15.11 The T-section shown in Fig. 15.18 is subjected to a factored shear $V_u = 28$ K and a factored equilibrium torque $T_u = 300$ K·in. and $M_u = 250$ K·ft. Design the necessary flexural and web reinforcement. Use $f'_c = 4$ ksi and $f_y = 60$ ksi.

15.12 Repeat Problem 15.11 if $V_u = 36$ K, $T_u = 360$ K·in., $M_u = 400$ K·ft., and $h = 24$ in.

15.13 Repeat Problem 15.11 using $f'_c = 3$ ksi and $f_y = 60$ ksi.

15.14 Repeat Problem 15.11 if T_u is a compatibility torsion.

15.15 Repeat Problem 15.13 if T_u is a compatibility torsion.

15.16 Repeat Problem 15.7 if T_u is a compatibility torsion.

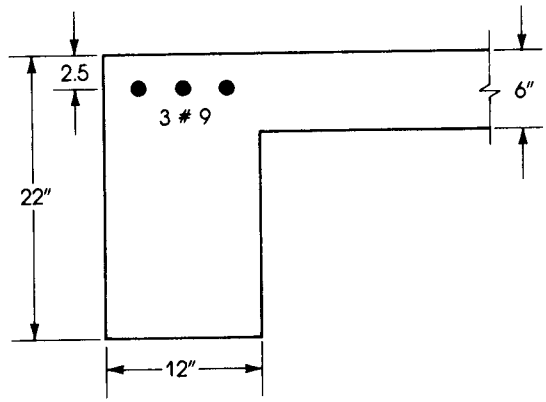


Figure 15.17 Problem 15.9.

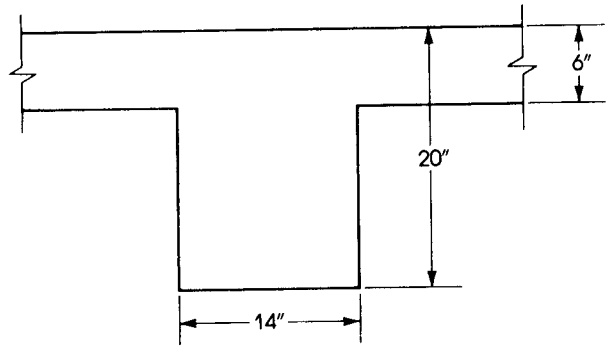


Figure 15.18 Problem 15.11.

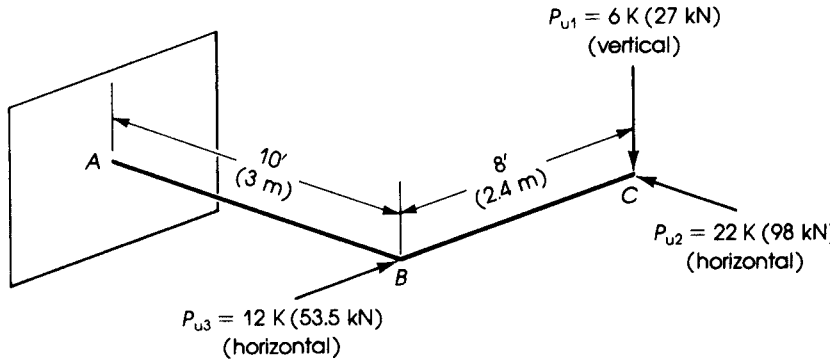


Figure 15.19 Problem 15.17.

15.17 The cantilever beam shown in Fig. 15.19 is subjected to the factored load shown.

a. Draw the axial and shearing forces and the bending and torsional moment diagrams.

b. Design the beam section at A using a steel percentage less than or equal to ρ_{max} for bending moment. Use $b = 16$ in. (300 mm), $f'_c = 4$ ksi, and $f_y = 60$ ksi.

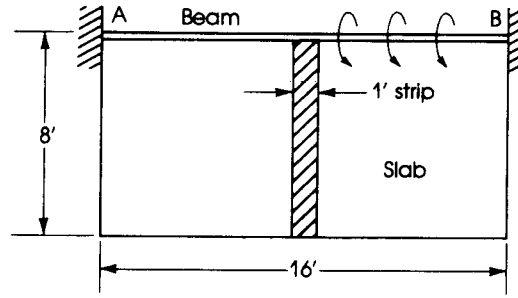


Figure 15.20 Problem 15.18.

15.18 The size of the slab shown in Fig. 15.20 is 16 by 8 ft; it is supported by the beam AB, which is fixed at both ends. The uniform dead load on the slab (including its own weight) equals 100 psf, and the uniform live load equals 80 psf. Design the section at support A of beam AB using $f'_c = 4$ ksi, $f_y = 60$ ksi, $b_w = 14$ in., $h = 20$ in., a slab thickness of 5 in., and the ACI Code requirements.

CHAPTER 16

CONTINUOUS BEAMS AND FRAMES



Reinforced concrete parking structure, Minneapolis, Minnesota.

16.1 INTRODUCTION

Reinforced concrete buildings consist of different types of structural members, such as slabs, beams, columns, and footings. These structural members may be cast in separate units as precast concrete slabs, beams, and columns or with the steel bars extending from one member to the other, forming a monolithic structure. Precast units are designed as structural members on simple supports unless some type of continuity is provided at their ends. In monolithic members, continuity in the different elements is provided, and the structural members are analyzed as statically indeterminate structures.

The analysis and design of continuous one-way slabs were discussed in Chapter 9, and the design coefficients and reinforcement details were shown in Figs. 9.8 and 9.9. In one-way floor systems, the loads from slabs are transferred to the supporting beams, as shown in Fig. 16.1*a*. If the factored load on the slab is w_u psf, the uniform load on beams AB and BC per unit length is $w_u s$ plus the self-weight of the beam. The uniform load on beams DE and EF is $w_u s/2$ plus the self-weight of the beam. The load on column B equals $W_u LS$, whereas the loads on columns E, A, and D are $W_u LS/2$, $W_u SL/2$, and $W_u LS/4$, respectively.

In two-way rectangular slabs supported by adequate beams on four sides, the floor loads are transferred to the beam from tributary areas bounded by 45° lines, as shown in Fig. 16.1*b*. Part of the floor loads are transferred to the long beams AB, BC, DE, and EF from trapezoidal areas, whereas the rest of the floor loads are transferred to the short beams AD, BE, and CF from triangular areas. In square slabs, loads are transferred to all surrounding beams from triangular floor areas. Interior beams carry loads from both sides, whereas end beams carry loads from one side only. Beams in both directions are usually cast monolithically with the slabs; therefore, they should be analyzed as statically indeterminate continuous beams. The beams transfer their loads in turn to the supporting columns. The load on column B equals $W_u LS$, while the loads on columns E, A, and D are $W_u LS/2$, $W_u SL/2$, and $W_u LS/4$, respectively. The tributary area for each column extends from the centerlines of adjacent spans in each direction.

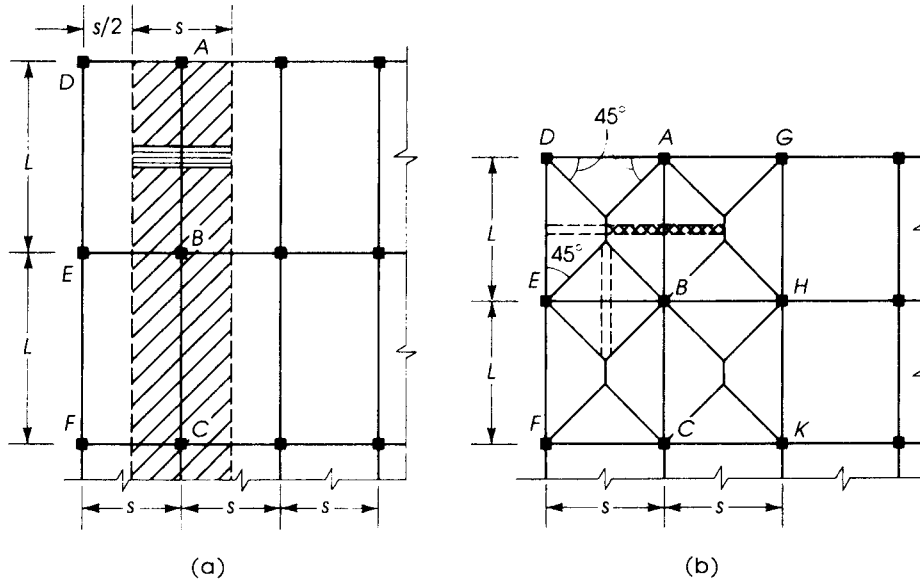


Figure 16.1 Slab loads on supporting beams: (*a*) one-way direction, $L/s > 2$; and (*b*) two-way direction, $L/s \leq 2$.

16.2 MAXIMUM MOMENTS IN CONTINUOUS BEAMS

16.2.1 Basic Analysis

The computation of bending moments and shear forces in reinforced concrete continuous beams is generally based on the elastic theory. When reinforced concrete sections are designed using the strength design method, the results are not entirely consistent with the elastic analysis. However, the ACI Code does not include provisions for a plastic design or limit design of reinforced concrete continuous structures except in allowing moment redistribution, as is explained later in this chapter.

16.2.2 Loading Application

The bending moment at any point in a continuous beam depends not only on the position of loads on the same span, but also on the loads on the other spans. In the case of dead loads, all spans must be loaded simultaneously, because the dead load is fixed in position and magnitude. In the case of moving loads or occasional live loads, the pattern of loading must be considered to determine the maximum moments at the critical sections. Influence lines may be used to determine the position of the live load to calculate the maximum and minimum moments. However, in this chapter, simple rules based on load-deflection curves are used to determine the loading pattern that produces maximum moments.

16.2.3 Maximum and Minimum Positive Moments within a Span

The maximum positive bending moment in a simply supported beam subjected to a uniform load w K/ft is at midspan, and $M = wl_2/8$. If one or both ends are continuous, the restraint at the continuous end will produce a negative moment at the support and slightly shift the

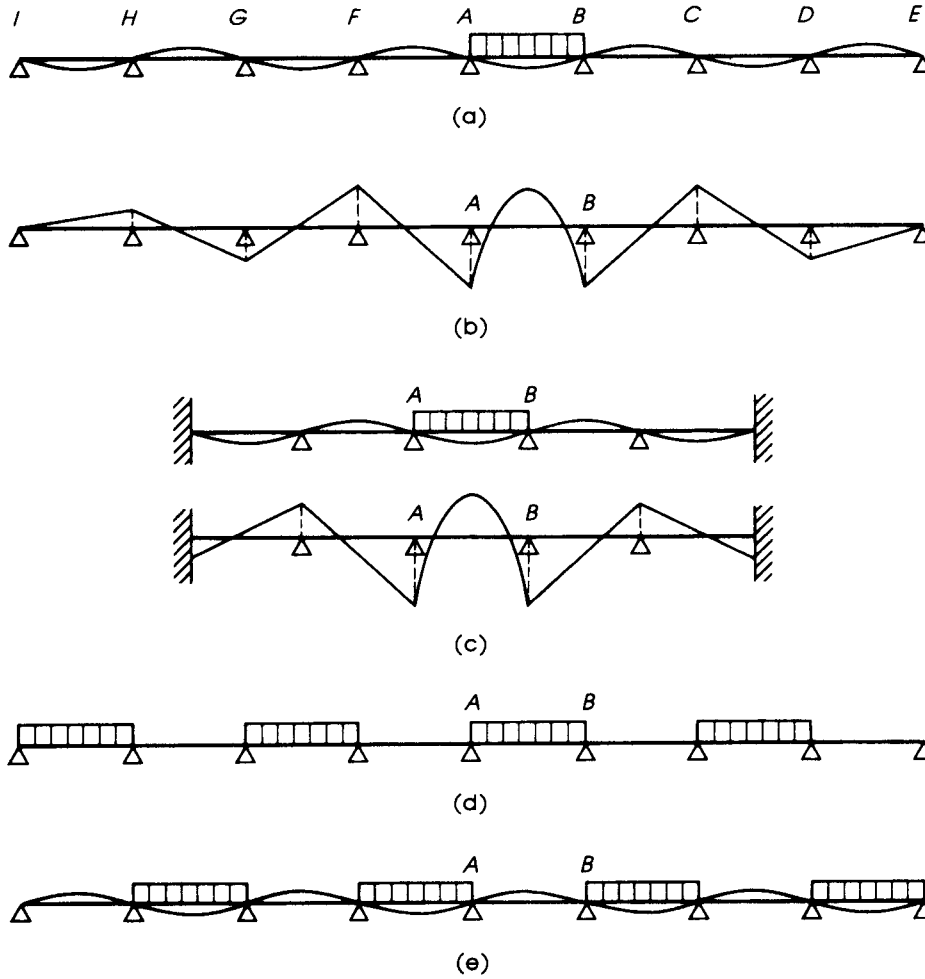


Figure 16.2 Loadings for maximum and minimum moment within span *AB*.

location of the maximum positive moment from midspan. The deflected shape of the continuous beam for a single-span loading is shown in Fig. 16.2*a*; downward deflection indicates a positive moment and upward deflection indicates a negative moment. If all spans deflected downward are loaded, each load will increase the positive moment at the considered span *AB* (Fig. 16.2*d*). Therefore, to calculate the maximum positive moment within a span, the live load is placed on that span and on every alternate span on both sides. The factored live load moment, calculated as explained before, must be added to the factored dead-load moment at the same section to obtain the maximum positive moment.

The bending moment diagram due to a uniform load on *AB* is shown in Fig. 16.2*b*. The deflections and the bending moments decrease rapidly with the distance from the loaded span *AB*. Therefore, to simplify the analysis of continuous beams, the moments in any span can be computed by considering the loaded span and two spans on either side of the considered span *AB*, assuming fixed supports at the far ends (Fig. 16.2*c*).

If the spans adjacent to span AB are loaded, the deflection curve will be as shown in Fig. 16.2*e*. The deflection within span AB will be upward, and a negative moment will be produced in span AB. This negative moment must be added to the positive moment due to dead load to obtain the final bending moment. Therefore, to calculate the minimum positive moment (or maximum negative moment) within a span AB, the live load is placed on the adjacent spans and on every alternate span on both sides of AB (Fig. 16.2*e*).

16.2.4 Maximum Negative Moments at Supports

In this case, it is required to determine the maximum negative moment at any support, say, support A (Fig. 16.3). When span AB is loaded, a negative moment is produced at support A. Similarly, the loading of span AF will produce a negative moment at A. Therefore, to calculate the maximum negative moment at any support, the live load is placed on the two adjacent spans and on every alternate span on both sides (Fig. 16.3).

In the structural analysis of continuous beams, the span length is taken from center to center of the supports, which are treated as knife-edge supports. In practice, the supports are always made wide enough to take the loads transmitted by the beam, usually the moments acting at the face of supports. To calculate the design moment at the face of the support, it is quite reasonable to deduct a moment equal to $V_u c/3$ from the factored moment at the centerline of the support, where V_u is the factored shear and c is the column width.

16.2.5 Moments in Continuous Beams

Continuous beams and frames can be analyzed using approximate methods or computer programs, which are available commercially. Other methods, such as the displacement and force methods of analysis based on the calculation of the stiffness and flexibility matrices, may also be adopted. Slope deflection and moment-distribution methods may also be used. These methods are explained in books dealing with the structural analysis of beams and frames. However, the ACI Code, Section 8.3, gives approximate coefficients for calculating the bending moments and shear forces in continuous beams and slabs. These coefficients were given in Chapter 9. The moments obtained using the ACI coefficients will be somewhat larger than those arrived at by exact analysis. The limitations stated in the use of these coefficients must be met.

Example 16.1

The slab-beam floor system shown in Fig. 16.4 carries a uniform live load of 130 psf and a dead load that consists of the slab's own weight plus 80 psf. Using the ACI moment coefficients, design a typical interior continuous beam and draw detailed sections. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, beam width (b) = 12 in., 12-by-12 in. columns, and a slab thickness of 5.0 in.

Solution

1. Design of slabs: The floor slabs act as one-way slabs, because the ratio of the long to the short side is greater than 2. The design of a typical continuous slab was discussed in Example 9.4.

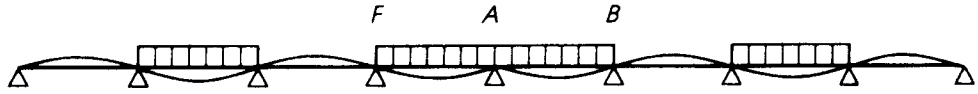


Figure 16.3 Loading for maximum negative moment at support *A*.

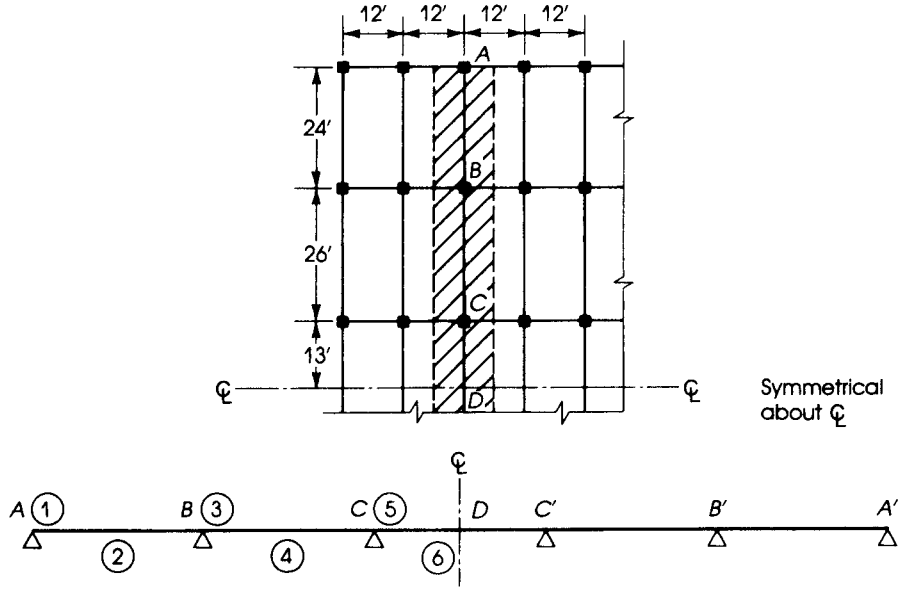


Figure 16.4 Example 16.1.

2. Loads on slabs:

$$\text{Dead load} = \frac{5}{12} \times 150 + 80 = 142.5 \text{ psf}$$

$$\text{Live load} = 130 \text{ psf}$$

$$\text{Factored load } (w_u) = 1.2(142.5) + 1.6(130) = 379 \text{ psf}$$

Loads on beams: A typical interior beam ABC carries slab loads from both sides of the beam, with a total slab width of 12 ft.

$$\text{Factored load on beam} = 12 \times 379 + 1.2 \times \text{(self-wseight of beam web)}$$

The depth of the beam can be estimated using the coefficients of minimum thickness of beams shown in Table A.6. For $f_y = 60$ ksi, the minimum thickness of the first beam AB is $L/18.5 = (24 \times 12)/18.5 = 15.6$ in. Assume a total depth of 22 in. and a web depth of $22 - 5 = 17$ in. Therefore, the factored load on beam $ABCD$ is

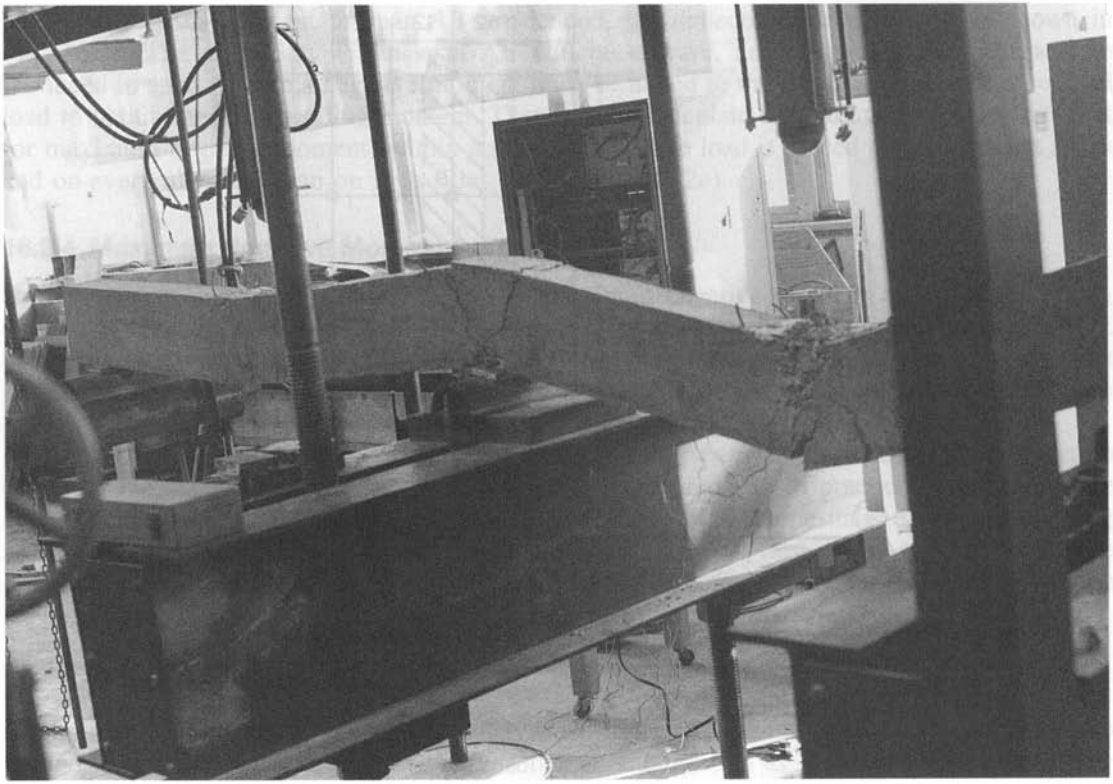
$$w_u = 12 \times 379 + 1.2\left(\frac{17 \times 12}{144} \times 150\right) = 4804 \text{ lb/ft}$$

Use 4.8 K/ft.

3. Moments in beam ABC: Moment coefficients are shown in Fig. 9.8. The beam is continuous on five spans and symmetrical about the centerline at D. Therefore, it is sufficient to design half of the beam $ABCD$, because the other half will have similar dimensions and reinforcement. Because the spans AB and BC are not equal and the ratio $\frac{26}{24}$ is less than 1.2, the ACI moment coefficients can be applied to this beam. Moreover, the average of the adjacent clear span is used to calculate the negative moments at the supports.

Moments at critical sections are calculated as follows (Fig. 16.4):

$$M_u = \text{coefficient} \times w_u l_n^2$$



Test on a continuous reinforced concrete beam. Plastic hinges developed in the positive and negative maximum moment regions.

Location	1	2	3	4	5	6
Moment coefficient	$-\frac{1}{16}$	$+\frac{1}{14}$	$-\frac{1}{10}$	$+\frac{1}{16}$	$-\frac{1}{11}$	$+\frac{1}{16}$
M_u (K·ft)	−158.7	181.4	−276.5	187.5	−272.7	187.5

4. Determine beam dimensions and reinforcement.

a. Maximum negative moment is −276.5 K·ft. Using $\rho_{max} = 0.016$, $R_u = 740$ psi.

$$R_{u\,max} = 820 \text{ psi} \qquad \rho_{max} = 0.01806 \text{ (Table 4.1)} \qquad \phi = 0.9$$

$$d = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{276.5 \times 12}{0.74 \times 12}} = 19.3 \text{ in.}$$

For one row of reinforcement, total depth is 19.3 + 2.5 = 21.8 in., say, 23 in., and actual d is 20.5 in. $A_s = 0.016 \times 12 \times 19.3 = 3.7 \text{ in.}^2$; use four no. 9 bars in one row. Note that total depth used here is 23 in., which is more than the 22 in. assumed to calculate the weight of the beam. The additional load is negligible, and there is no need to revise the calculations.

b. The sections at the supports act as rectangular sections with tension reinforcement placed within the flange. The reinforcements required at the supports are as follows:

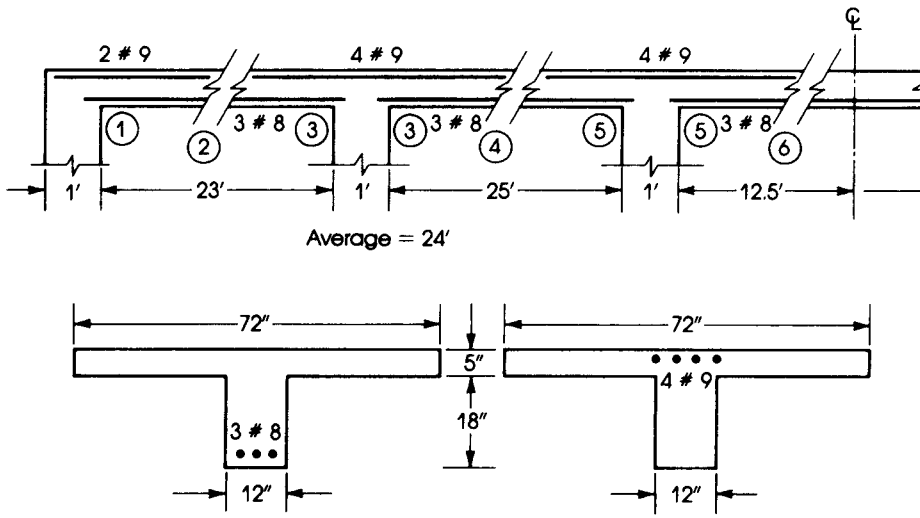


Figure 16.5 Example 16.1: reinforcement details.

Location	1	3	5
M_u(K·ft)	−158.7	−276.5	−272.7
$R_u = \frac{M_u}{bd^2}$ (psi)	378	658	649
ρ(%)	0.77	1.48	1.45
A_s(in.^{2})	1.9	3.7	3.6
No. 9 bars	2	4	4

c. For the midspan T-sections, $M_u = +187.5$ K·ft. For $a = 1.0$ in. and flange width = 72 in.,

$$A_s = \frac{M_u}{\phi f_y\left(d - \frac{a}{2}\right)} = \frac{187.5 \times 12}{0.9 \times 60(20.5 - 1/2)} = 2.1 \text{ in.}^2$$

$$\text{Check } a\text{: } a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.1 \times 60}{0.85 \times 3 \times 72} = 0.7 \text{ in.}$$

Revised a gives $A_s = 2.07$ in.^{2} Therefore, use three no. 8 bars ($A_s = 2.35$ in.^{2}) for all midspan sections. Reinforcement details are shown in Fig. 16.5.

5. Design the beam for shear, as explained in Chapter 8.

6. Check deflection and cracking, as explained in Chapter 6.

16.3 BUILDING FRAMES

A building frame is a three-dimensional structural system consisting of straight members that are built monolithically and have rigid joints. The frame may be one bay long and one story high, such as the portal frames and gable frames shown in Fig. 16.6*a*, or it may consist of multiple

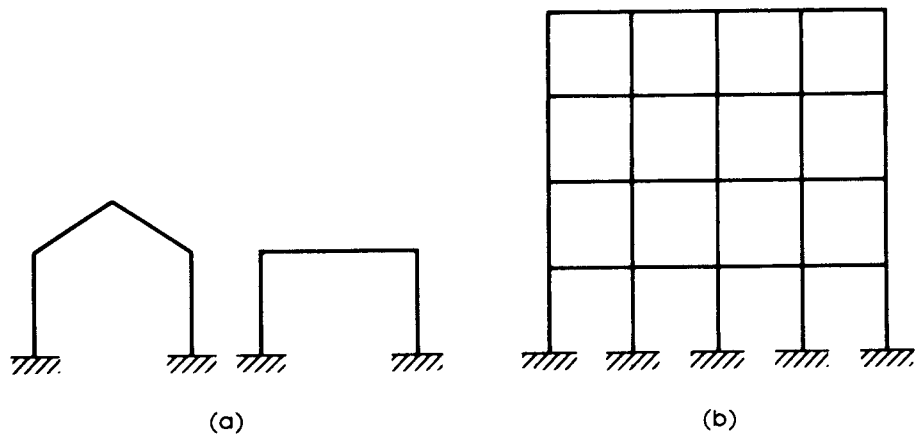


Figure 16.6 (*a*) Gable and portal frames (schematic) and (*b*) multibay, multistory frame.

bays and stories, as shown in Fig. 16.6*b*. All members of the frame are considered continuous in the three directions, and the columns participate with the beams in resisting external loads. Besides reducing moments due to continuity, a building frame tends to distribute the loads more uniformly on the frame. The effects of lateral loads, such as wind and earthquakes, are also spread over the whole frame, increasing its safety. For design purposes, approximate methods may be used by assuming a two-dimensional frame system.

A frame subjected to a system of loads may be analyzed by the equivalent frame method. In this method, the analysis of the floor under consideration is made assuming that the far ends of the columns above and below the slab level are fixed (Fig. 16.7). Usually, the analysis is performed using the moment–distribution method.

In practice, the size of panels, distance between columns, number of stories, and the height of each story are known because they are based upon architectural design and utility considerations. The sizes of beams and columns are estimated first, and their relative stiffnesses based on the gross concrete sections are used. Once the moments are calculated, the sections assumed previously are checked and adjusted as necessary. More accurate analysis can be performed using computers, which is recommended in the structural analysis of statically indeterminate structures with several redundants. Methods of analysis are described in many books on structural analysis.

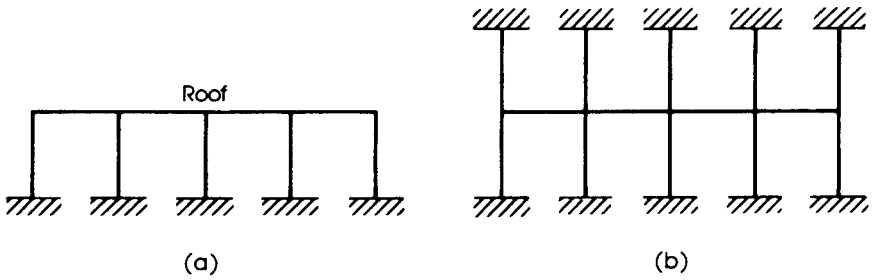


Figure 16.7 Assumption of fixed column ends for frame analysis.

16.4 PORTAL FRAMES

A portal frame consists of a reinforced concrete stiff girder poured monolithically with its supporting columns. The joints between the girder and the columns are considered rigidly fixed, with the sum of moments at the joint equal to 0. Portal frames are used in building large-span halls, sheds, bridges, and viaducts. The top member of the frame may be horizontal (portal frame) or inclined (gable frame) (Fig. 16.8). The frames may be fixed or hinged at the base.

A statically indeterminate portal frame may be analyzed by the moment-distribution method or any other method used to analyze statically indeterminate structures. The frame members are designed for moments, shear, and axial forces, whereas the footings are designed to carry the forces acting at the column base.

Girders and columns of frames may be of uniform or variable depths, as shown in Fig. 16.8. The forces in a single-bay portal frame of uniform sections may be calculated as follows.

16.4.1 Two Hinged Ends

The forces in the members of a portal frame with two hinged ends [2] can be calculated using the following expressions (Fig. 16.9).

For the case of a uniform load on top member BC, let

$$K = 3 + 2\left(\frac{I_2}{I_1} \times \frac{h}{L}\right)$$

where

I_1 and I_2 = column and beam moments of inertia
h and L = height and span of frame

The bending moments at joints B and C are

$$M_B = M_C = -\frac{wL^2}{4K}$$

$$\text{Maximum positive moment at midspan } BC = \frac{wL^2}{8} + M_B$$

The horizontal reaction at A is $H_A = M_B/h = H_D$. The vertical reaction at A is $V_A = WL/2 = V_D$. For a uniform load on half the beam BC, Fig. 16.9*b*: $M_B = M_C = -WL_2/8K$, $H_A = H_D = M_B/h$, $V_A = 3WL/8$, and $V_D = WL/8$.

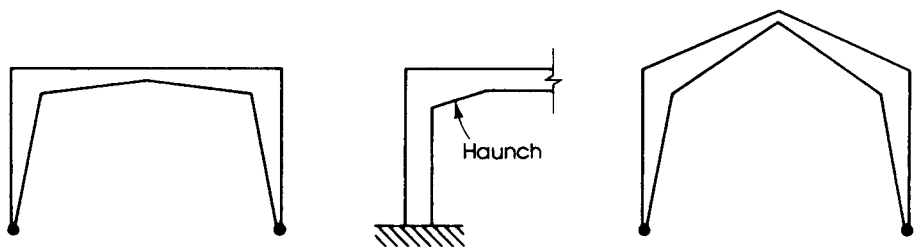


Figure 16.8 Portal and gable frames.

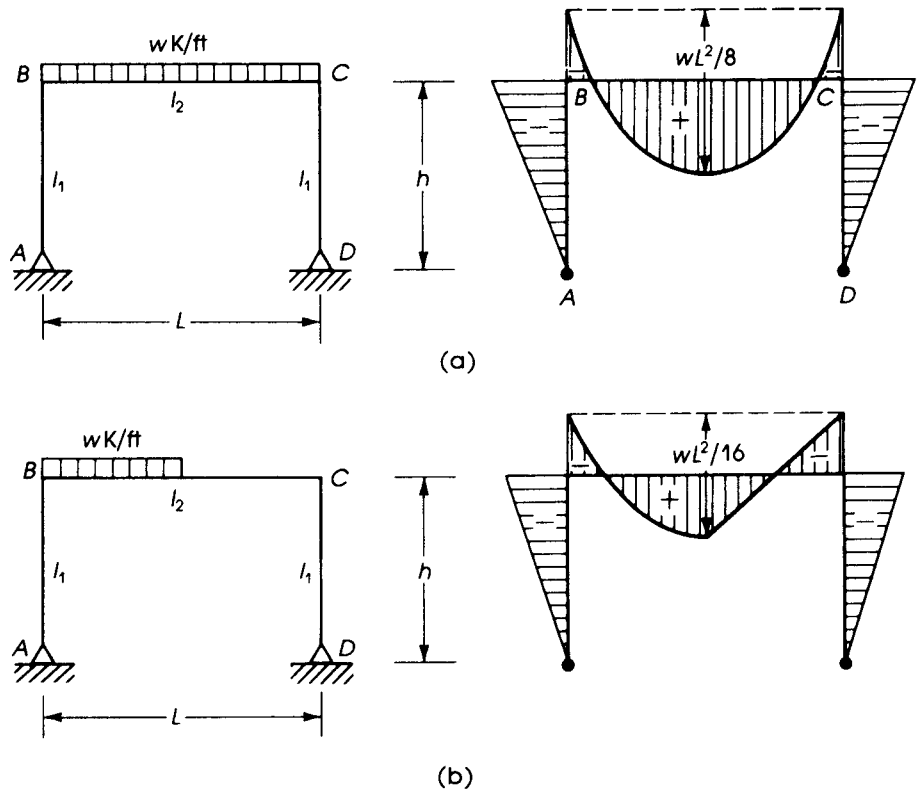


Figure 16.9 Portal frame with two hinged ends. Bending moments are drawn on the tension side.

16.4.2 Two Fixed Ends

The forces in the members of a portal frame with two fixed ends [2] can be calculated as follows (Fig. 16.10).

For a uniform load on top member BC, let

$$K_1 = 2 + \left(\frac{I_2}{I_1} \times \frac{h}{L}\right)$$

$$M_B = M_C = -\frac{wL^2}{6K_1}$$

$$M_A = M_D = \frac{M_B}{2} \qquad M \text{ (midspan)} = \frac{wL^2}{8} + M_B$$

$$H_A = H_D = \frac{3M_A}{h} \quad \text{and} \quad V_A = V_D = \frac{wL}{2}$$

For a uniform load on half the top member BC, let

$$K_2 = 1 + 6\left(\frac{I_2}{I_1} \times \frac{h}{L}\right)$$

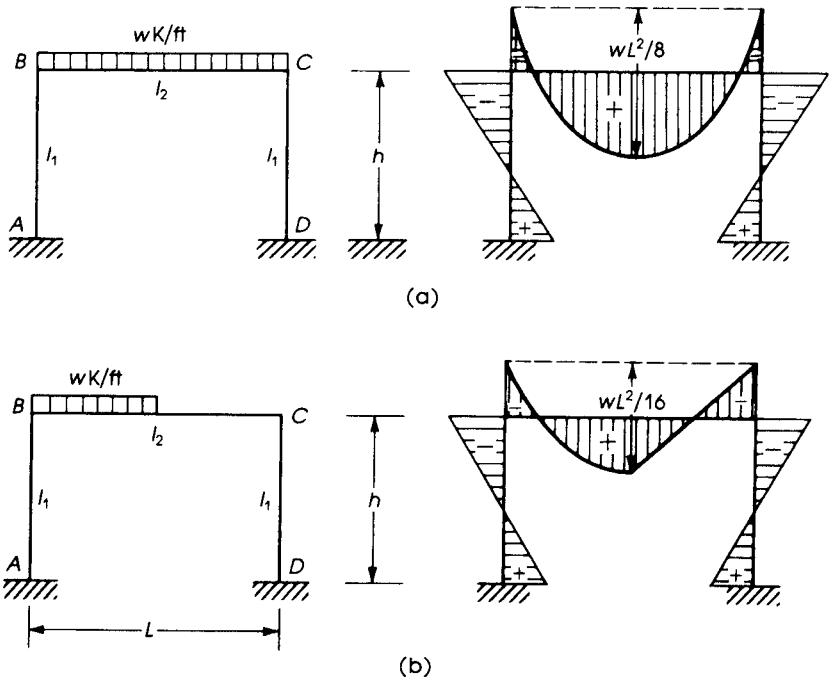


Figure 16.10 Portal frame with fixed ends. Bending moments are drawn on the tension side.

Then

$$M_A = \frac{wL^2}{8}\left(\frac{1}{3K_1} - \frac{1}{8K_2}\right) \qquad M_B = \frac{wl^2}{8}\left(\frac{2}{3K_1} + \frac{1}{8K_2}\right)$$

$$M_C = \frac{wL^2}{8}\left(\frac{2}{3K_1} - \frac{1}{8K_2}\right) \qquad M_D = \frac{wL^2}{8}\left(\frac{1}{3K_1} - \frac{1}{8K_2}\right)$$

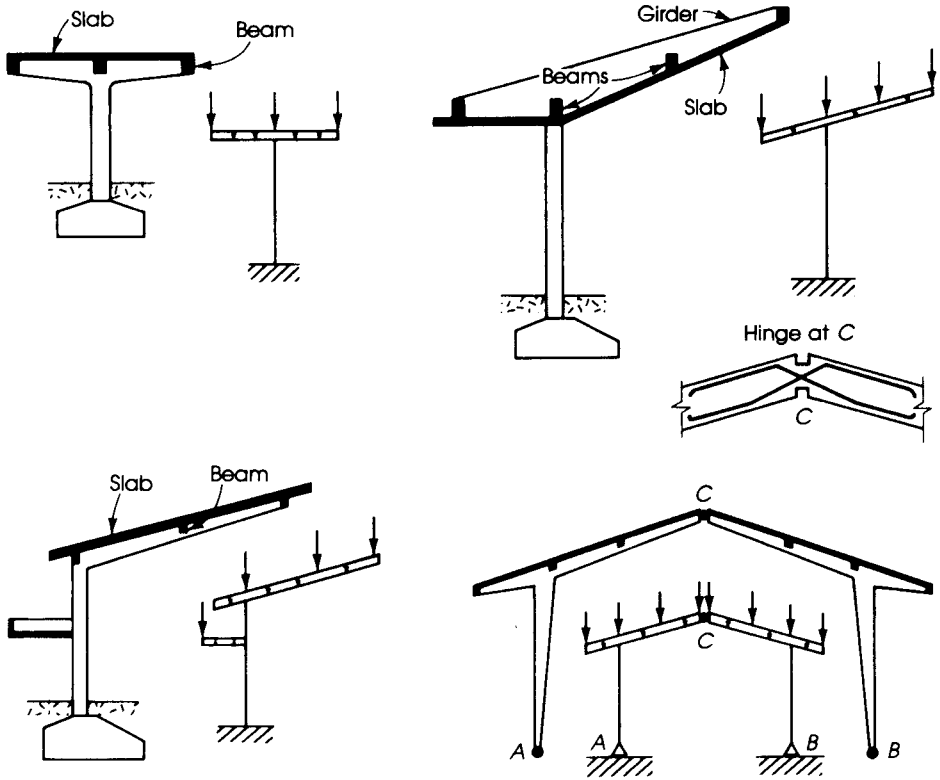
$$H_A = H_D = \frac{wl^2}{8} \times \frac{1}{K_1 h}$$

$$V_A = \frac{wL}{2} - V_D \quad \text{and} \quad V_D = \frac{wL}{8}\left(1 - \frac{1}{4K_2}\right)$$

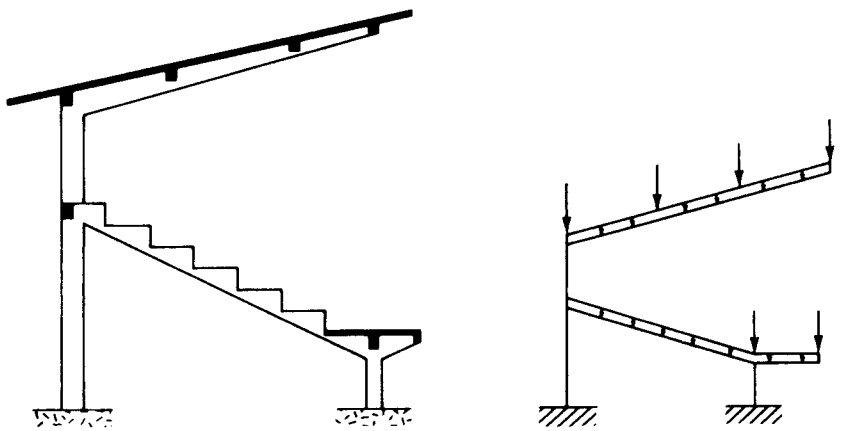
16.5 GENERAL FRAMES

The main feature of a frame is its rigid joints, which connect the horizontal or inclined girders of the roof to the supporting structural members. The continuity between the members tends to distribute the bending moments inherent in any loading system to the different structural elements according to their relative stiffnesses. Frames may be classified as

1. Statically determinate frames (Fig. 16.11*a*)
2. Statically indeterminate frames (Fig. 16.12)
3. Statically indeterminate frames with ties (Fig. 16.13).



(a)



(b)

Figure 16.11 (*a*) Statically determinate frames and (*b*) reinforced concrete stadium.

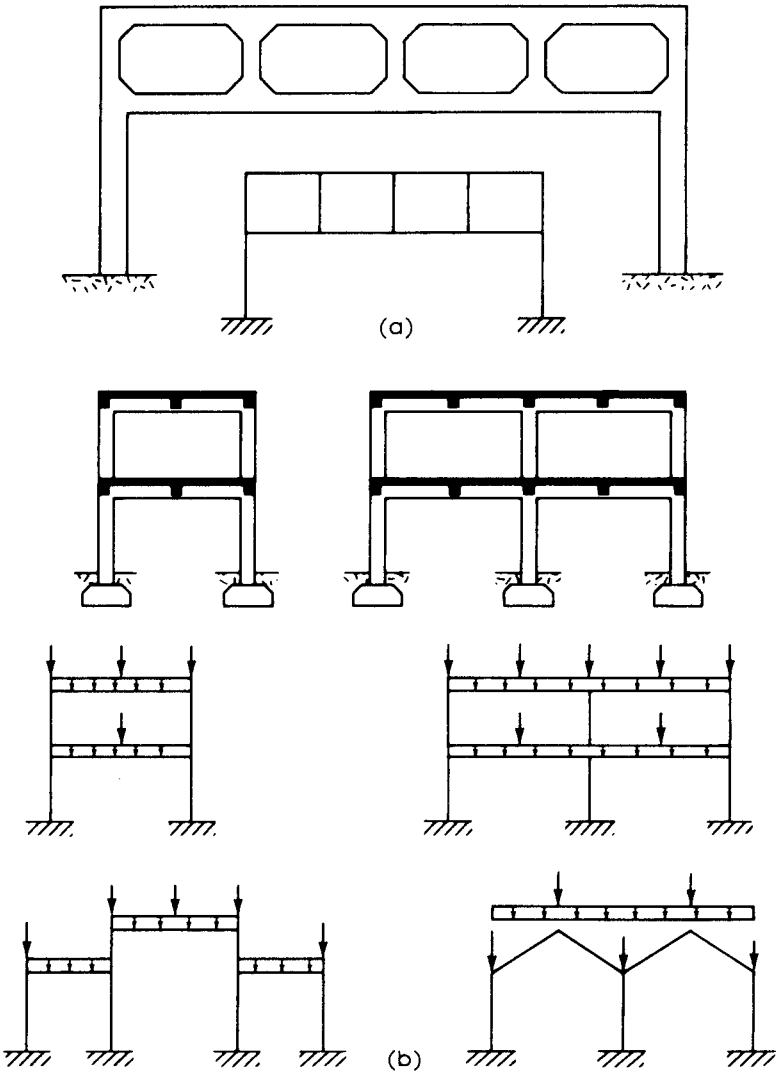


Figure 16.12 (*a*) Vierendeel girder and (*b*) statically indeterminate frames.

Different methods for the analysis of frames and other statically indeterminate structures are described in books dealing with structural analysis. Once the bending moments, shear, and axial forces are determined, the sections can be designed as the examples in this book are. Analysis may also be performed using computer programs.

16.6 DESIGN OF FRAME HINGES

The main types of hinges used in concrete structures are Mesnager hinges, Considére hinges, and lead hinges [19]. The description of each type is given next.

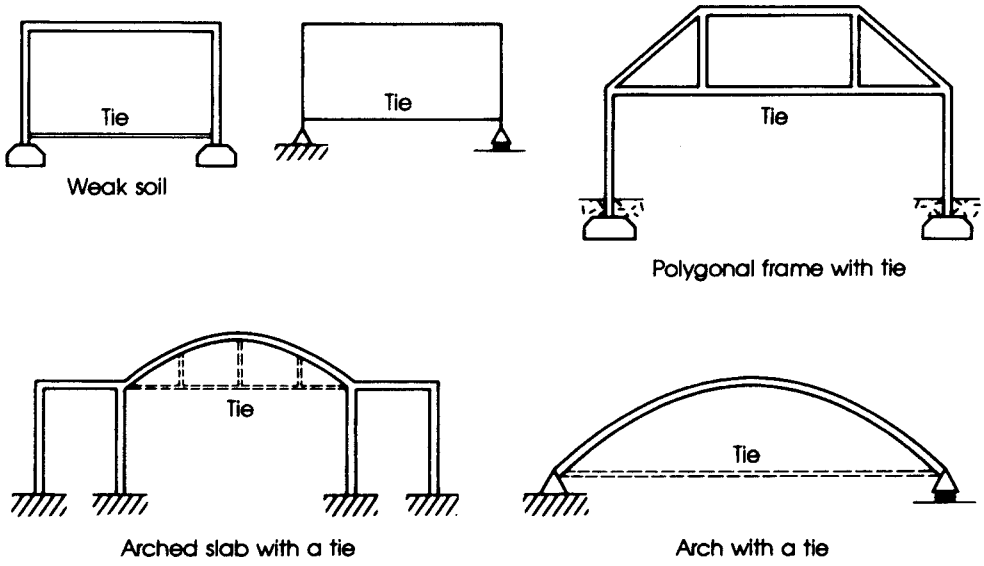


Figure 16.13 Structures with ties.

16.6.1 Mesnager Hinge

The forces that usually act on a hinge are a horizontal force, H, and a vertical force, P. The resultant of the two forces, R, is transferred to the footing through the crossing bars A and B shown in Fig. 16.14. The inclination of bars A and B to the horizontal varies between 30° and 60°, with a minimum distance a, measured from the lower end of the frame column, equal to $8D$, where D is the diameter of the inclined bars. The gap between the frame column and the top of the footing y varies between 1 in. and $1.3h'$, where h' is the width of the concrete section at the hinge level. A practical gap height ranges between 2 and 4 in. The rotation of the frame ends is taken by the hinges, and the gap is usually filled with bituminous cork or similar flexible material. The bitumen protects the cork in contact with the soil from deterioration. The crossing bars A and B are subjected to compressive stresses that must not exceed one-third the yield strength of the steel bars f_y under service loads or 0.55 f_y under factored loads. The low stress is assumed because any rotation at the hinge tends to bend the bars and induces secondary flexural stresses. It is generally satisfactory to keep the compression stresses low rather than to compute secondary stresses. The areas of bars A and B are calculated as follows:

$$\text{Area of bars } A: \quad A_{s_1} = \frac{R_1}{0.55 f_y} \tag{16.1}$$

$$\text{Area of bars } B: \quad A_{s_2} = \frac{R_2}{0.55 f_y} \tag{16.2}$$

where R_1 and R_2 are the components of the resultant R in the direction of the inclined bars A and B using factored loads. The components R_1 and R_2 are usually obtained by statics as follows:

$$H + R_2 \sin\theta = R_1 \sin\theta \quad \text{and} \quad R_2 = R_1 - \frac{H}{\sin\theta} \tag{16.3a}$$

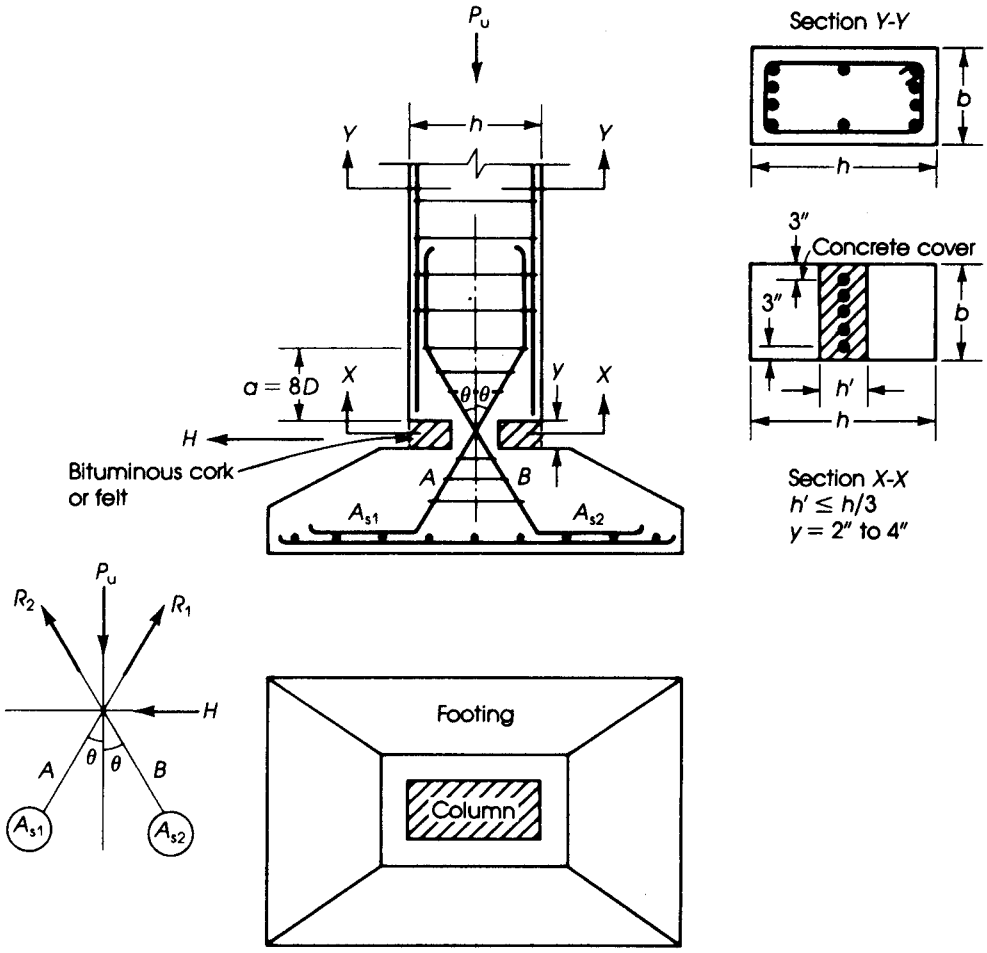


Figure 16.14 Hinge details.

Also, $(R_1 + R_2) \cos \theta = P_u$, so

$$R_1 = \frac{P_u}{\cos \theta} - R_2 = \frac{P_u}{\cos \theta} - \left[R_1 - \frac{H}{\sin \theta}\right] \tag{16.3b}$$

$$R_1 = \frac{1}{2}\left[\frac{P_u}{\cos \theta} + \frac{H}{\sin \theta}\right]$$

The inclined hinge bars transmit their force through the bond along the embedded lengths in the frame columns and footings. Consequently, the bars exert a bursting force, which must be resisted by ties. The ties should extend a distance $a = 8D$ (the larger bar diameter of bars A and B) in both columns and footings. The bursting force F can be estimated as

$$F = \frac{P_u}{2} \tan \theta + \frac{Ha}{0.85d} \tag{16.4}$$

If the contribution of concrete is neglected, then the area of tie reinforcement, A_{st}, required to resist F is

$$A_{st} = \frac{F}{\phi f_y} = \frac{F}{0.85 f_y} \tag{16.5}$$

The stress in the ties can also be computed as follows:

$$f_s(\text{ties}) = \frac{\frac{P_u}{2}\tan\theta + \frac{Ha}{0.85d}}{0.005ab + A_{st}\ (\text{ties})} \leq 0.85\, f_y \tag{16.6}$$

where

A_{st} = area of ties within a distance $a = 8D$
d = effective depth of column section
b = width of column section

This type of hinge is used for moderate forces and limited by the maximum number of inclined bars that can be placed within the column width.

16.6.2 Considère Hinge

The difference between the *Considère hinge* and the Mesnager one is that the normal force P_u is assumed to be transmitted to the footing by one or more short, spirally reinforced columns extending deep into the footing, whereas the horizontal force H is assumed to be resisted by the inclined bars A and B (Fig. 16.15). The load capacity of the spirally reinforced short column

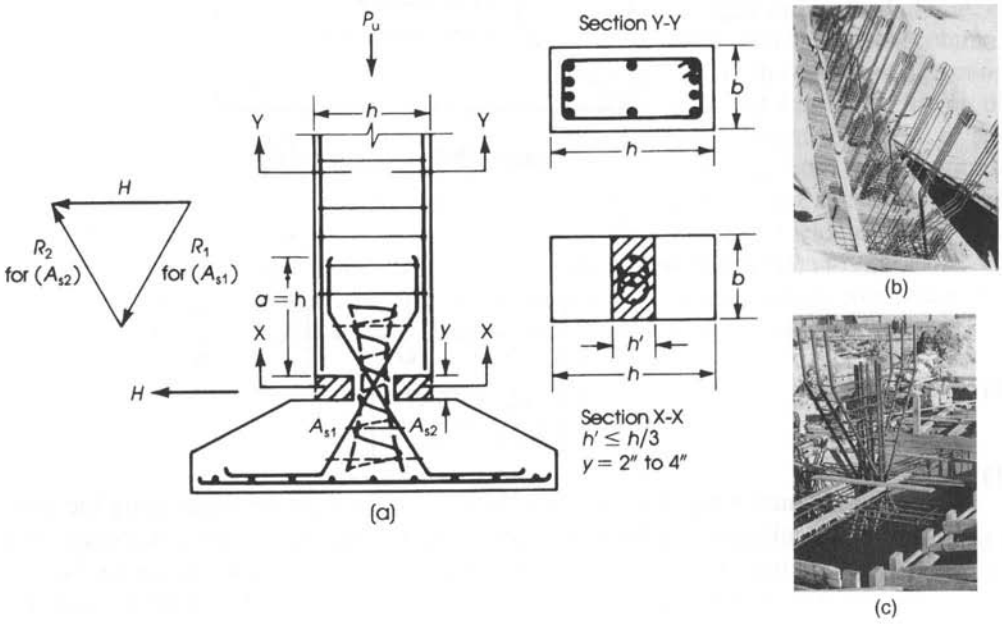


Figure 16.15 (*a*) Considère hinge, (*b*) Mesnager hinges for a series of portal frames, and (*c*) Considère hinge.

may be calculated using Eq. 10.7, neglecting the factor 0.85 for minimum eccentricity.

$$P_u = \phi P_n = 0.75[0.85 f'_c(A_g - A_{st}) + A_{st} f_y] \tag{16.7}$$

where A_g is the area of concrete hinge section, or bh', and A_{st} is the area of longitudinal bars within the spirals. Ties should be provided in the column up to a distance equal to the long side of the column section h.

16.6.3 Lead Hinges

Lead hinges are sometimes used in reinforced concrete frames. In this type of hinge, a lead plate, usually 0.75 to 1.0 in. thick, is used to transmit the normal force, P_u, to the footing. The horizontal force H is resisted by vertical bars placed at the center of the column and extended to the footing (Fig. 16.16). At the base of the column, the axial load P_u should not exceed the bearing strength specified by the ACI Code, Section 10.14, of $\phi(0.85 f'_c A_1)$, where $\phi = 0.65$ and $A_1 = bh'$. The area of the vertical bars is $A_s = H/0.6 f_y$, where H = factored horizontal force.

Example 16.2

An 84- by 40-ft hall is to be covered by reinforced concrete slabs supported on hinged-end portal frames spaced at 12 ft on centers (Fig. 16.17). The frame height is 15 ft, and no columns are allowed within the hall area. The dead load on the slabs is that due to self-weight plus 75 psf from roof finish. The live load on the slab is 85 psf. Design a typical interior frame using normal-weight concrete with $f'_c = 4$ ksi and $f_y = 60$ ksi for the frame and a column width of $b = 16$ in.

Solution

The main structural design of the building will consist of the following:

- Design of one-way slabs
- Analysis of the portal frame
- Design of the frame girder due to moment
- Design of the frame girder due to shear
- Design of columns
- Design of hinges
- Design of footings

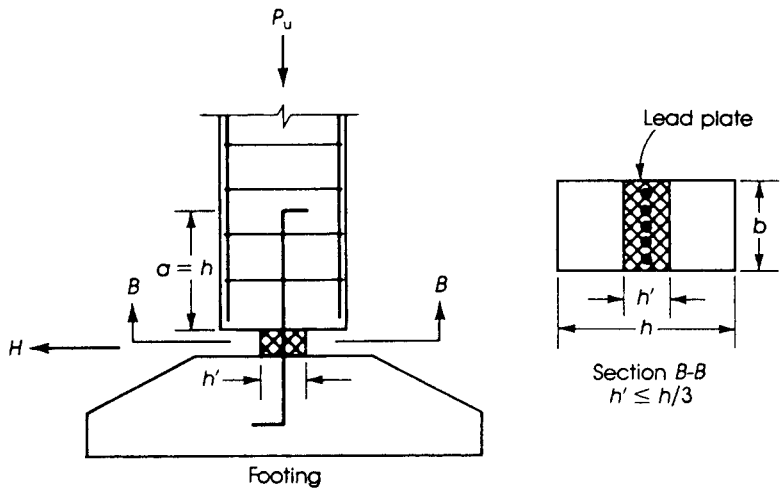


Figure 16.16 Lead hinge.

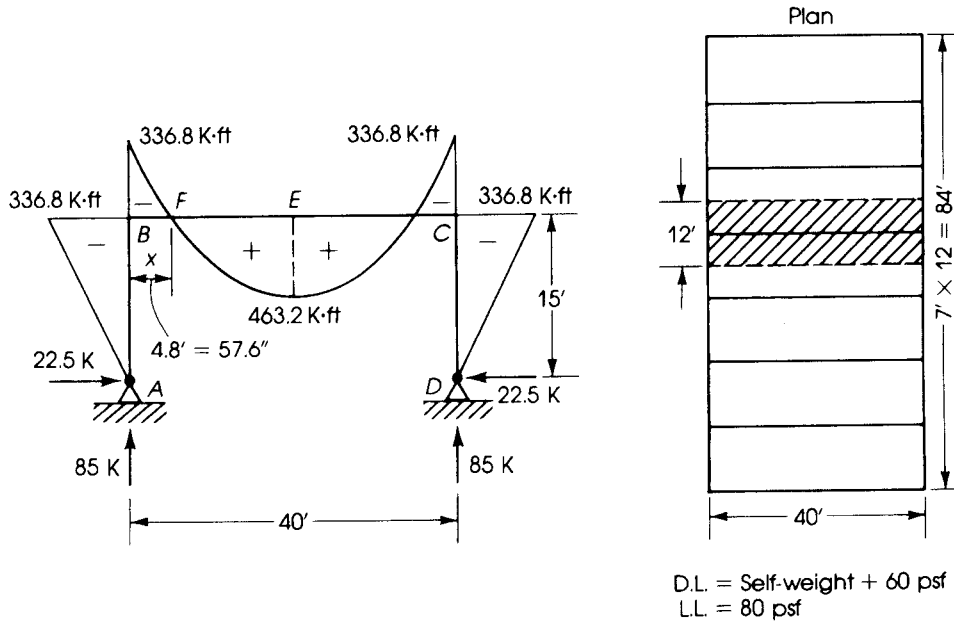


Figure 16.17 Example 16.2: design of portal frame.

1. One-way roof slab: The minimum thickness of the first slab is $L/30$, because one end is continuous and the other end is discontinuous (Table A.6 in Appendix A).

$$\text{Minimum depth} = \frac{12 \times 12}{30} = 4.8 \text{ in.}$$

Assume a slab thickness of 5.0 in. and design the slab following the steps of Example 9.5.

2. Analysis of an interior portal frame:

a. The loads on slabs are

$$\text{Dead load on slabs} = 75 + \left(\frac{5}{12} \times 150\right) = 137.5 \text{ psf}$$

$$\text{Factored load on slabs} = 1.2 \times 137.5 + 1.6 \times 85 = 301 \text{ psf}$$

b. Determine loads on frames: The interior frame carries a load from a 12-ft slab in addition to its own weight. Assume that the depth of the beam is $L/24 = (40 \times 12)/24 = 20$ in. Use a projection below the slab of 16 in., giving a total beam depth of 21 in.

$$\text{Dead load from self-weight of beam} = \left(\frac{16}{12}\right)^2 \times 150 = 267 \text{ lb/ft}$$

$$\text{Total factored load on frame} = 301 \times 12 + 1.2 \times 267$$

$$= 3932 \text{ lb/ft}$$

$$w_u = 4.0 \text{ K/ft}$$

c. Determine the moment of inertia of the beam and columns sections. The beam acts as a T-section. The effective width of slab acting with the beam is the smallest of span/4 = 40 × 12/4 = 120 in., $16h_s + b_w = 16 \times 5 + 16 = 96$, or 12 ft × 12 = 144 in. Use $b = 96$ in.

The centroid of the section from the top fibers is

$$y = \frac{96 \times 5 \times 2.5 + 16 \times 16 \times 13}{96 \times 5 + 16 \times 16} = 6.2 \text{ in.}$$

$$I_b(\text{beam}) = \left[\frac{96}{12}(5)^3 + 96 \times 5(3.7)^2\right] + \left[\frac{16}{12}(16)^3 + 16 \times 16(6.8)^2\right]$$

$$= 24{,}870 \text{ in.}^4$$

It is a common practice to consider an approximate moment of inertia of a T-beam as equal to twice the moment of inertia of a rectangular section having the total depth of the web and slab:

$$I_b(\text{beam}) = 2 \times \frac{16}{12}(21)^3 = 24{,}696 \text{ in.}^4$$

(For an edge beam, approximate $I = 1.5 \times bh_3/12$.) Assume a column section 16 by 20 in. (having the same width as the beam).

$$I_c(\text{column}) = \frac{16}{12}(20)^3 = 10{,}667 \text{ in.}^4$$

d. Let the factor

$$K = 3 + 2\left(\frac{I_b}{L} \times \frac{h}{I_c}\right) = 3 + 2\left(\frac{24{,}870}{40} \times \frac{15}{10{,}667}\right) = 4.75$$

Referring to Fig. 16.17 and for a uniform load $w_u = 4.0$ K/ft on BC,

$$M_B = M_C = -\frac{w_u L^2}{4K} = -\frac{4.0(40)^2}{4 \times 4.75} = -336.8 \text{ K·ft}$$

The maximum positive bending moment at midspan of BC equals

$$w_u \frac{L^2}{8} + M_B = \frac{4.0(40)^2}{8} - 336.8 = 463.2 \text{ K·ft}$$

The horizontal reaction at A is

$$H_a = H_D = \frac{M_B}{h} = \frac{336.8}{15} = 22.5 \text{ K}$$

The vertical reaction at A is

$$V_A = V_D = \frac{w_u L}{2} + \text{weight of column}$$

$$V_A = 4.0 \times \frac{40}{2} + \frac{20}{12} \times \frac{16}{12} \times 0.150 \times 15 \text{ ft} = 85.0 \text{ K}$$

The bending moment diagram is shown in Fig. 16.17.

e. To consider the sidesway effect on the frame, the live load is placed on half the beam BC, and the moments are calculated at the critical sections. This case is not critical in this example.

f. The maximum shear at the two ends of beam BC occurs when the beam is loaded with the factored load w_u, but the maximum shear at midspan occurs when the beam is loaded with half the live load and with the full dead load:

$$V_u \text{ at support} = 4.0 \times \frac{40}{2} = 80.0 \text{ K}$$

$$V_u \text{ at midspan} = W_l \frac{L}{8} = (1.7 \times 80 \times 12) \times \frac{40}{8}$$

$$= 8160 \text{ lb} = 8.16 \text{ K}$$

g. The axial force in each column is $V_A = V_D = 85.0\,\text{K}$.

h. Let the point of zero moment in BC be at a distance x from B; then

$$M_B = w_u L \frac{x}{2} - w_u \frac{x^2}{2}$$

$$336.8 = 4.0\left(\frac{40x}{2} - \frac{x^2}{2}\right) \quad \text{or} \quad x^2 - 40x + 168.4 = 0$$

$$x = 4.8\text{ ft} = 57.6\text{ in. from } B$$

3. Design of girder BC:

a. Design the critical section at midspan. $M_u = 463.2\,\text{K·ft}$, web width is flange width is $b_w = 16\,\text{in.}$, flange width is $b = 96\,\text{in.}$, and $d = 21 - 3.5 = 17.5\,\text{in.}$ (assuming two rows of steel bars). Check if the section acts as a rectangular section with effective $b = 96\,\text{in.}$ Assume $a = 1.0\,\text{in}$; then

$$A_s = \frac{M_u}{\phi f_y\left(d - \frac{a}{2}\right)} = \frac{463.2 \times 12}{0.9 \times 60\left(17.5 - \frac{1.0}{2}\right)} = 6.05\text{ in.}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{6.05 \times 60}{0.85 \times 4 \times 96} = 1.1\text{ in.} < 5.0\text{ in.}$$

The assumed a equals approximately the calculated a. The section acts as a rectangular section; therefore, use six no. 9 bars. Check $b_{\min}$ (to place bars in one row):

$$b_{\min} = 11\left(\frac{9}{8}\right) + 2\left(\frac{3}{8}\right) + 3 = 16.13\text{ in.} > 16\text{ in.}$$

Place bars in two rows, as shown in Fig. 16.18.

b. Design the critical section at joint B: $M_u = 336.8\,\text{K·ft}$, $b = 16\,\text{in.}$, and $d = 21 - 2.5 = 18.5\,\text{in.}$ (for one row of steel bars). The slab is under tension, and reinforcement bars are placed on top of the section.

$$R_u = \frac{M_u}{bd^2} = \frac{336.8 \times 12{,}000}{16(18.5)^2} = 738\text{ psi}$$

From tables in Appendix A, $\rho = 0.016 < \rho_{\max} = 0.018$. (Tension-controlled section, $\phi = 0.9$)

$$A_s = 0.016 \times 16 \times 18.5 = 4.73\text{ in.}^2$$

Use five no. 9 bars in one row.

4. Design the girder BC due to shear:

a. The critical section is at a distance d from the face of the column with a distance from the column centerline of $10 + 18.5 = 28.5\,\text{in.} = 2.4\,\text{ft}$. Thus,

$$V_u\text{ (at distance } d) = 80 - 4 \times 2.4 = 70.4\text{ K}$$

b. The shear strength provided by concrete is

$$\phi V_c = \phi(2\lambda\sqrt{f'_c})b_w d$$

$$\phi V_c = \frac{0.75 \times 2 \times (1)}{1000} \times \sqrt{4000} \times 16 \times 18.5 = 28.1\text{ K}$$

The shear force to be provided by web reinforcement is

$$\phi V_s = V_u - \phi V_c = 70.4 - 28.1 = 42.3\text{ K}$$

$$V_s = \frac{42.3}{0.75} = 56.4\text{ K}$$

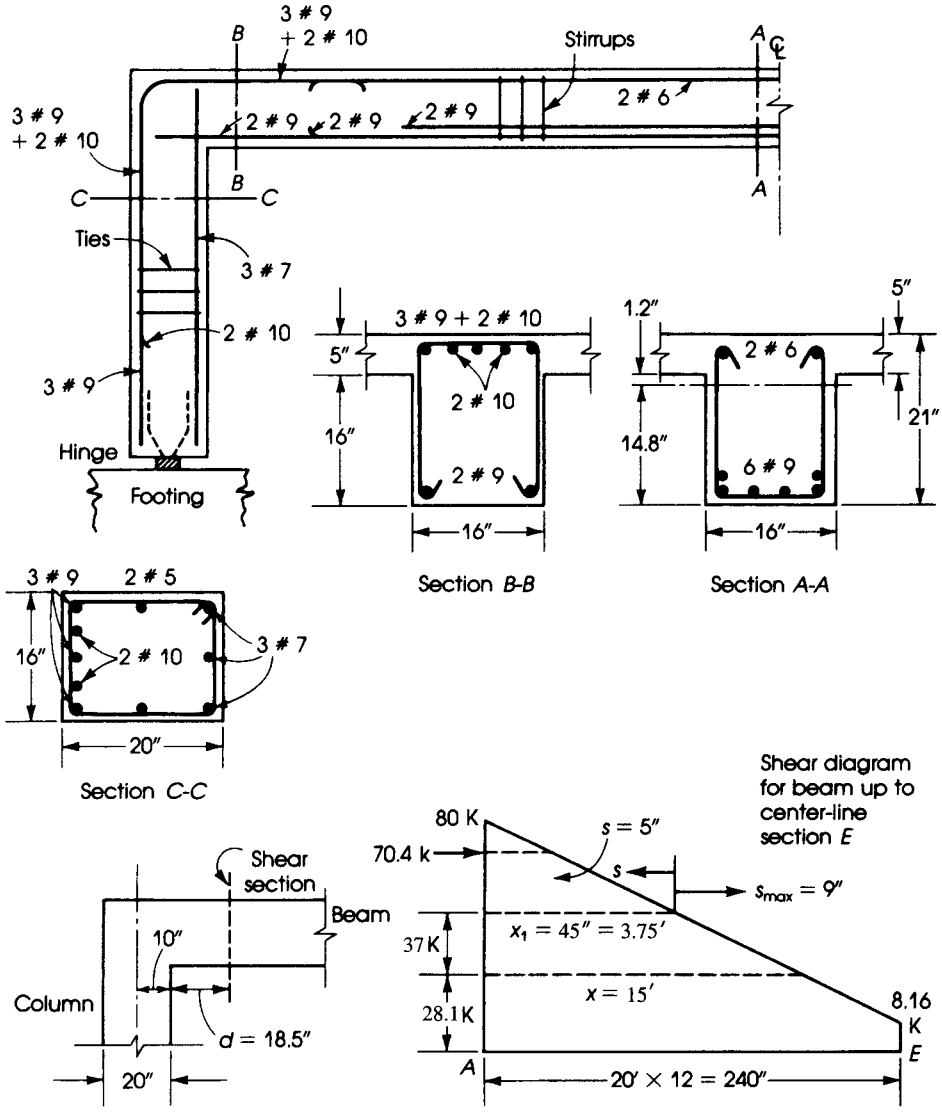


Figure 16.18 Example 16.2: reinforcement details of frame sections.

c. Choose no. 4 stirrups and $A_v = 2 \times 0.20 = 0.40\,\text{in.}^2$ Thus,

$$S = \frac{A_v f_y d}{V_s} = \frac{0.40 \times 60 \times 18.5}{56.4} = 7.8 \text{ in., say 7 in.}$$

d. Maximum spacing of no. 4 stirrups is

$$S_{\max} = \frac{d}{2} = \frac{18.5}{2} = 9.25 \text{ in.} \quad \text{say, 9 in.}$$

or

$$S_{\max} = \frac{A_v f_y}{50 b_w} = \frac{0.40 \times 60{,}000}{50 \times 16} = 30 \text{ in.}$$

Check for maximum spacing of $d/2$: $V_s \leq 4\sqrt{f'_c}b_w d$ or

$$V_s \leq 4\sqrt{4000} \times \frac{16 \times 18.5}{1000} = 74.9 \text{ K}$$

The value V_s of 56.4 is less than 74.9 K, so use $S_{\max} = 9$ in.

$$V_s \text{ (for } S_{\max} = 9 \text{ in.)} = \frac{A_v f_y d}{S} = \frac{0.40 \times 360 \times 18.5}{9} = 49.3 \text{ K}$$

$$\phi V_s = 0.75 \times 49.3 = 37 \text{ K}$$

The distance from the face of the column where $S_{\max} = 9$ in. can be used is equal to 45 in. = 3.75 ft (from the triangle of shear forces).

e. Distribution of stirrups:

First stirrups at $S/2 = 3.0$ in.

7 stirrups at 7 in. = 49.0 in

19 stirrups at 9 in. = 171.0 in. (Total = 223 in.)

The distance from the face of the column to the centerline of the beam is 240 − 10 = 230 in. Use the same distribution for the second half of the beam, and place one stirrup at midspan.

5. Design the column section at joint B: $M_u = 336.8$ K·ft, $P_u = 80$ K, $b = 16$ in., and $h = 20$ in.

a. Assuming that the frame under the given loads will not be subjected to sidesway, then the effect of slenderness may be neglected, and the column can be designed as a short column when

$$\frac{Kl_u}{4} < 34 - \frac{12M_1}{M_2} \quad \text{(see Section 12.5)}$$

$$M_1 = 0 \quad \text{and} \quad M_2 = 336.8 \text{ K·ft}$$

Let $K = 0.8$ (Fig. 12.2), $L_u = 15 - 21/(2 \times 12) = 14.125$ ft, and $r = 0.3h = 0.3 \times 20 = 6$ in; then

$$\frac{KL_u}{r} = 0.8 \times \frac{14.125 \times 12}{6} = 22.6 < 34$$

If K is assumed equal to 1.0, then

$$\frac{KL_u}{r} = 28.25 < 34$$

Therefore, design the member as a short column.

b. The design procedure is similar to Examples 11.16 and 11.3.

$$\text{Eccentricity } (e) = \frac{M_u}{P_u} = \frac{336.8 \times 12}{80} = 50.5 \text{ in.}$$

This is a large eccentricity, and it will be assumed that the section is in the transition region, $\phi < 0.9$.

$$d = 20 - 2.5 = 17.5 \text{ in.}$$

c. Because $e = 50.5$ in. is much greater than d, determine approximate A_s and A'_s from the M_u only and then check the final section by statics, as was explained in Example 11.3. For $M_u = 336.8$ K·ft, $b = 16$ in., $h = 20$ in., and $d = 17.5$ in., $R_u = M_u/bd^2 = 336.8(12{,}000)/16(17.5)^2 = 825$ psi.

$$\rho = 0.0183 \quad \text{and} \quad A_s = \rho bd = 0.0183(16)(17.5) = 5.12 \text{ in.}^2$$

Choose three no. 9 and two no. 10 bars and $A_s = 5.53\,\text{in.}^2$ Choose $A'_s = A_s/3 = 5.13/3 = 1.7\,\text{in.}^2$ and three no. 7 bars ($A'_s = 1.8\,\text{in.}^2$) (Fig. 16.18). When the eccentricity, e, is quite large, it is a common practice to use $A'_s = A_s/3$ or $A_s/2$ instead of $A_s = A'_s$.

d. Check the load capacity of the final section using $A_s = 5.53\,\text{in.}^2$ and $A'_s = 1.8\,\text{in.}^2$, similar to Example 11.3, according to the following steps:

i. $P_n = C_c + C_s - T$

$$C_c = 0.85 f'_c ab = 0.85(4)(16)a = 54.4a$$

$$C_s = A'_s(f'_s - 0.85 f'_c) = 1.8(60 - 0.85 \times 4) = 101.8\ \text{K}$$

$$T = A_s f_y = 5.53(60) = 331.8\ \text{K}$$

$$P_n = 54.4a + 101.8 - 331.8 = (43.4a - 230) \qquad \text{(I)}$$

ii. Take moments about A_s:

$$P_n = \frac{1}{e'}\left[C_c\left(d - \frac{a}{2}\right) + C_s(d - d')\right]$$

$e' = e + d''$, where d'' is the distance from A_s to the plastic centroid of the section. The plastic centroid occurs at 11.1 in. from the extreme compression fibers and $d'' = d - x = 6.4\,\text{in.}$ (refer to Example 11.1).

$$e' = 50.5 + 6.4 = 56.9\ \text{in.}$$

$$P_n = \frac{1}{56.9}\left[54.4a\left(17.5 - \frac{a}{2}\right) + 101.8(15)\right]$$

$$= 16.73a - 0.478a^2 + 26.86 \qquad \text{(II)}$$

iii. Equate Eqs. I and II and solve to get $a = 6.313\,\text{in.}$ and $P_n = 113.5\,\text{K}$. Check $f'_s = 87(c - d')/c \leq f_y$: $c = a/0.85 = 7.43\,\text{in.}$ and $f'_s = 87(7.43 - 2.5)/7.43 = 58\,\text{ksi}$, which is close to the 60 ksi assumed in the calculations. Choose no. 3 ties spaced at 16 in.

iv. Check ϕ: $d_t = 17.5\,\text{in.}$

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right)0.003 = 0.00407$$

$$\phi = 0.65 + (\varepsilon_t - 0.002)\left(\frac{250}{3}\right) = 0.823$$

$$\phi P_n = 0.823(113.5) = 93.3\ \text{K} > 80\ \text{K}$$

The section is adequate.

6. Check the adequacy of the column section at midheight, 7.5 ft from A: $M_u = 336.8/2 = 168.4\,\text{K·ft}$.

$$P_u = 80 + 2.5\ (\text{half the column weight}) = 82.5\ \text{K}$$

Use $A_s =$ three no. 9 bars and $A'_s =$ three no. 7 bars. In an approach similar to step 5, $\phi P_n = 122\,\text{K} > 82.5\,\text{K}$ (no. 10 bars can be terminated, and they have to be extended a development length below the midheight of the column).

7. Design the hinge at A: $M_u = 0$, $H = 22.5\,\text{K}$, $P_u = 85\,\text{K}$.

a. Choose a Mesnager hinge. Using Eqs. 16.3*a* and 16.3*b*, $R_1 = 72\,\text{K}$ and $R_2 = 27\,\text{K}$. (Refer to Fig. 16.19 with $\theta = 30°$.)

$$A_{s1} = \frac{R_1}{0.55 f_y} = \frac{72}{0.55 \times 60} = 2.2\ \text{in.}^2$$

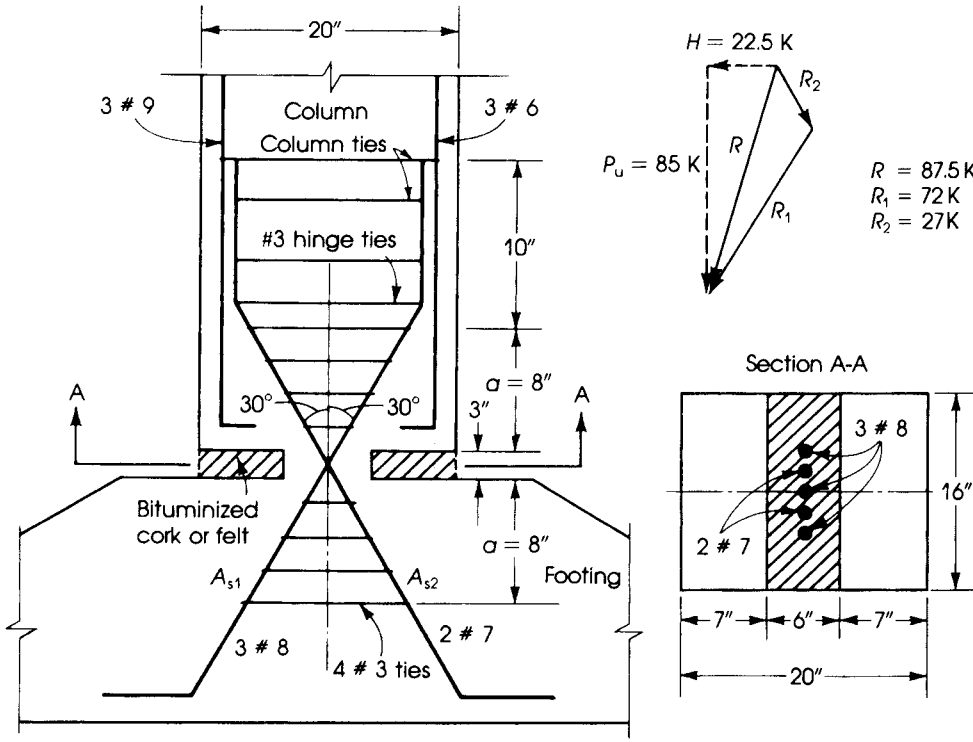


Figure 16.19 Example 16.2: hinge details.

Choose three no. 8 bars ($A_s = 2.35\,\text{in.}^2$).

$$A_{s2} = \frac{R_2}{0.55 \times f_y} = \frac{27}{0.55 \times 60} = 0.82\ \text{in.}^2$$

Choose two no. 7 bars ($A_s = 1.2\,\text{in.}^2$). Arrange the crossing bars by placing one no. 8 bar and then one no. 7 bar, as shown in Fig. 16.19 (or use five no. 8 bars.)

b. Lateral ties should be placed along a distance $a = 8 -$ bar diameter $= 8.0$ in. within the column and footing. The bursting force is

$$F = \frac{P_u}{2}\tan\theta + \frac{Ha}{0.85d}$$

For $\theta = 30^\circ$, $d = 17.5$ in., and $a = 8.0$ in.,

$$F = \frac{85}{2}\tan 30^\circ + \frac{22.5 \times 8}{0.85 \times 17.5} = 36.6\ \text{K}$$

$$\text{Area of ties} = \frac{36.6}{0.85 \times 60} = 0.72\ \text{in.}^2$$

If no. 3 closed ties (two branches) are chosen, then the area of one tie is $2 \times 0.11 = 0.22\,\text{in.}^2$ The number of ties is $0.72/0.22 = 3.27$, say, four ties spaced at $\frac{8}{3} = 2.7$ in., as shown in Fig. 16.19.

8. Design the footing: If the height of the footing is assumed to be h', then the forces acting on the footing are the axial load P and a moment $M = H/h'$. The soil pressure is calculated from

Eq. 13.14 of Chapter 13:

$$q = +\frac{P}{A} \pm \frac{Mc}{I} \leq \text{allowable soil pressure}$$

The design procedure of the footing is similar to that of Example 13.7.

16.7 INTRODUCTION TO LIMIT DESIGN

16.7.1 General

Limit state design of a structure falls into three distinct steps:

1. Determination of the factored design load, obtained by multiplying the dead and live loads by load factors. The ACI Code adopted the load factors given in Chapter 3.
2. Analysis of the structure under factored loads to determine the factored moments and forces at failure or collapse of the structure. This method of analysis has proved satisfactory for steel design; in reinforced concrete design, this type of analysis has not been fully adopted by the ACI Code because of the lack of ductility of reinforced concrete members. The Code allows only partial redistribution of moments in the structure based on an empirical percentage, as will be explained later in this chapter.
3. Design of each member of the structure to fail at the factored moments and forces determined from structural analysis. This method is fully established now for reinforced concrete design and the ACI Code permits the use of the strength design method, as was explained in earlier chapters.

16.7.2 Limit Design Concept

Limit design in reinforced concrete refers to the redistribution of moments that occurs throughout a structure as the steel reinforcement at a critical section reaches its yield strength. The ultimate strength of the structure can be increased as more sections reach their strength capacity. Although the yielding of the reinforcement introduces large deflections, which should be avoided under service loads, a statically indeterminate structure does not collapse when the reinforcement of the first section yields. Furthermore, a large reserve of strength is present between the initial yielding and the collapse of the structure.

In steel design, the term *plastic design* is used to indicate the change in the distribution of moments in the structure as the steel fibers, at a critical section, are stressed to their yield strength. The development of stresses along the depth of a steel section under increasing load is shown in Fig. 16.20. Limit analysis of reinforced concrete developed as a result of earlier research on steel structures and was based mainly on the investigations of Prager [4], Beedle [5], and J. B. Baker [6]. A. L. L. Baker [7] worked on the principles of limit design, whereas Cranston [8] tested portal frames to investigate the rotation capacity of reinforced concrete plastic hinges. However, more research work is needed before limit design can be adopted by the ACI Code.

16.7.3 Plastic Hinge Concept

The curvature, ϕ of a member increases with the applied bending moment M. For an underreinforced concrete beam, the typical moment–curvature and the load–deflection curves are shown

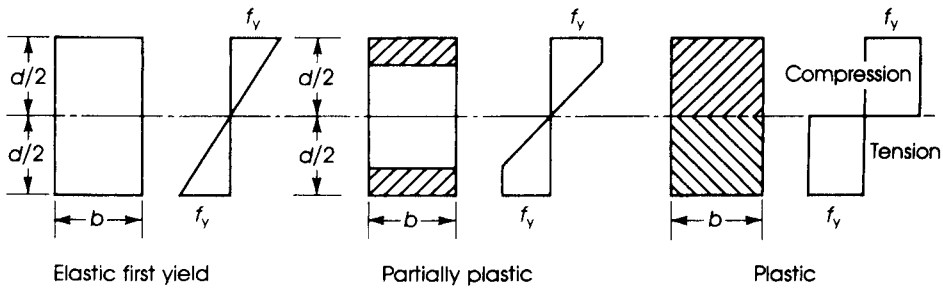


Figure 16.20 Distribution of yield stresses in a yielding steel rectangular section.

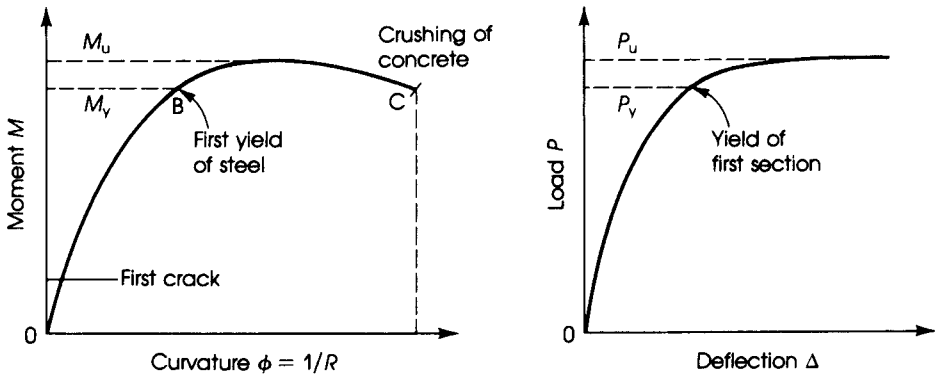


Figure 16.21 Yielding behavior of an under reinforced concrete beam.

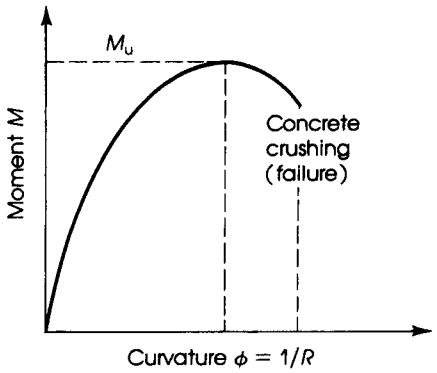


Figure 16.22 Yielding behavior of an overreinforced concrete beam.

in Fig. 16.21. A balanced or an overreinforced concrete beam is not permitted by the ACI Code, because it fails by the crushing of concrete and shows a small curvature range at factored moment (Fig. 16.22).

The significant part of the moment–curvature curve in Fig. 16.21 is that between B and C, in which M_u remains substantially constant for a wide range of values of ϕ. In limit analysis,

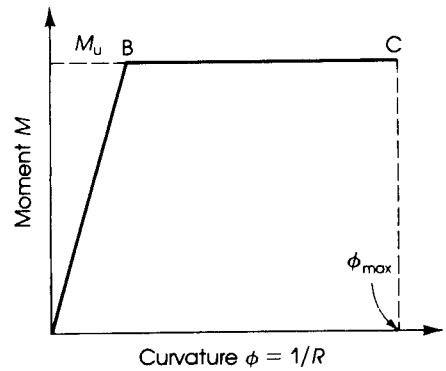


Figure 16.23 Idealized moment–curvature behavior of reinforced concrete beams.

the moment–curvature curve can be assumed to be of the idealized form shown in Fig. 16.23, where the curvature, ϕ, between B and C is assumed to be constant, forming a plastic hinge. Because concrete is a brittle material, there is usually considered to be a limit at which the member fails completely at maximum curvature at C.

Cranston [8] reported that in normally designed reinforced concrete frames, ample rotation capacity is available, and the maximum curvature at point C will not be reached until the failure or collapse of the frame. Therefore, when the member carries a moment equal to its factored moment, M_u, the curvature continues to increase between B and C without a change in the moment, producing a plastic hinge. The increase in curvature allows other parts of the statically indeterminate structure to carry additional loading.

16.8 THE COLLAPSEC MECHANISM

In limit design, the moment strength of a reinforced concrete member is reached when it is on the verge of collapse. The member collapses when there are sufficient numbers of plastic hinges to transform it into a mechanism. The required number of plastic hinges, n, depends upon the degree of redundancy, r, of the structure. The relation between n and r to develop a mechanism is

$$n = 1 + r \tag{16.8}$$

For example, in a simply supported beam no redundants exist, and $r = 0$. Therefore, the beam becomes unstable and collapses when one plastic hinge develops at the section of maximum moment, as shown in Fig. 16.24*a*. Applications to beams and frames are also shown in Fig. 16.24.

16.9 PRINCIPLES OF LIMIT DESIGN

Under working loads, the distribution of moments in a statically indeterminate structure is based on elastic theory, and the whole structure remains in the elastic range. In limit design, where factored loads are used, the distribution of moments at failure, when a mechanism is reached, is different from that distribution based on elastic theory. This change reflects moment redistribution.

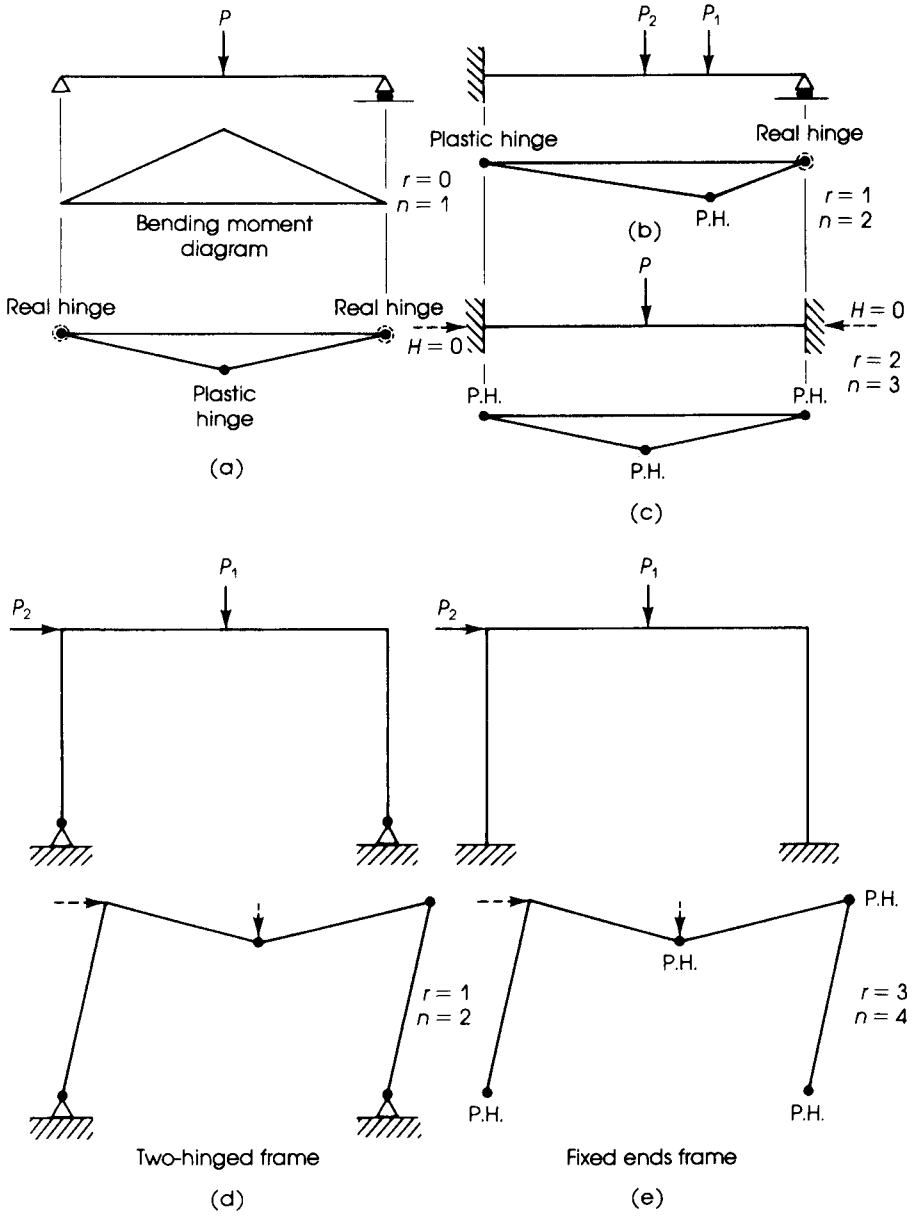


Figure 16.24 Development of plastic hinges (P.H.).

For limit design to be valid, four conditions must be satisfied.

1. *Mechanism condition:* Sufficient plastic hinges must be formed to transform the whole or part of the structure into a mechanism.
2. *Equilibrium condition:* The bending moment distribution must be in equilibrium with the applied loads.

3. *Yield condition:* The factored moment must not be exceeded at any point in the structure.
4. *Rotation condition:* Plastic hinges must have enough rotation capacity to permit the development of a mechanism.

Only the first three conditions apply to plastic design, because sufficient rotation capacity exists in ductile materials as steel. The fourth condition puts more limitations on the limit design of reinforced concrete members as compared to plastic design.

16.10 UPPER AND LOWER BOUNDS OF LOAD FACTORS

A structure on the verge of collapse must have developed the required number of plastic hinges to transform it into a mechanism. For arbitrary locations of the plastic hinges on the structure, the collapse loads can be calculated, which may be equal to or greater than the actual loads. Because the calculated loads cannot exceed the true collapse loads for the structure, then this approach indicates an upper or kinematic bound of the true collapse loads [10]. Therefore, if all possible mechanisms are investigated, the lowest M_u will be caused by the actual loads. Horne [11] explained the upper bound by assuming a mechanism and then calculating the external work, W_e, done by the applied loads and the internal work, W_i, done at the plastic hinges. If $W_e = W_i$, then the applied loads are either equal to or greater than the collapse loads.

If any arbitrary moment diagram is developed to satisfy the static equilibrium under the applied loads at failure, then the applied loads are either equal to or less than the true collapse loads. For different moment diagrams, different factored loads can be obtained. Higher values of the lower, or static, bound are obtained when the moments at several sections for the assumed moment diagram reach the collapse moment. Horne [11] explained the lower bound by assuming different moment distributions to obtain the one that is in equilibrium with the applied loads and satisfies the yield condition all over the structure. In this case, the applied loads are either equal to or less than the collapse loads.

16.11 LIMIT ANALYSIS

For the analysis of structures by the limit design procedure, two methods can be used, the virtual work method and the equilibrium method. In the virtual work method, the work done by the factored load, P_u (or w_u), to produce a given virtual deflection, Δ, is equated to the work absorbed at the plastic hinges. The external work done by loads is $W_e = \Sigma(w_u\ \Delta)$ or $\Sigma\ (P_u\Delta)$. The work absorbed by the plastic hinges is internal work $= Wi = \Sigma(M_u\theta)$.

Example 16.3

The beam shown in Fig. 16.25 carries a concentrated load at midspan. Calculate the collapse moment at the critical sections.

Solution

1. The beam is once statically indeterminate ($r = 1$), and the number of plastic hinges needed to transform the beam into a mechanism is $n = 1 + 1 = 2$ plastic hinges, at A and C. The first plastic hinge develops at A, and the beam acts as a simply supported member until a mechanism is reached.

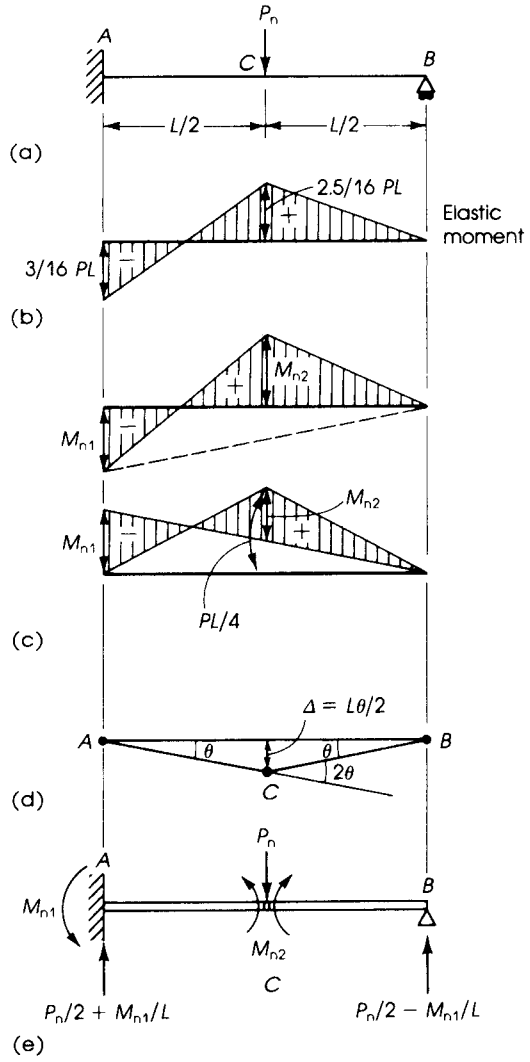


Figure 16.25 Example 16.3: $P_u = \phi P_n$ and $M_u = \phi M_n$.

2. If a rotation θ occurs at the plastic hinge at the fixed end, A, the rotation at the sagging hinge is $C = 2\theta$. The deflection of C under the load is $(L/2)\theta$ (Fig. 16.25).

$$W_e = \text{external work} = \sum P_u \Delta = P_u \left(\frac{L\theta}{2}\right)$$

$$W_i = \text{internal work} = \sum M_u \theta = M_{u_1}(\theta) + M_{u_2}(2\theta)$$

If the two sections at A and C have the same dimensions and reinforcement, then $M_{u_1} = M_{u_2} = M_u$, and $W_i = 3M_u\theta$. Equating W_e and W_i,

$$M_{u_1} + 2M_{u_2} = P_u \frac{L}{2} = 3M_u \quad \text{and} \quad M_u = \frac{P_u L}{6}$$

Example 16.4

Calculate the collapse moments at the critical sections for the beam shown in Fig. 16.26 due to a uniform load w_u.

Solution

1. The number of plastic hinges is two.
2. For a deflection at $C = 1.0$, the rotation at A, θ_A, is $1/a$; $\theta_B = 1/b$, and

$$\theta_c = \theta_A + \theta_B = \frac{1}{a} + \frac{1}{b} = \frac{a+b}{ab} = \frac{L}{ab}$$

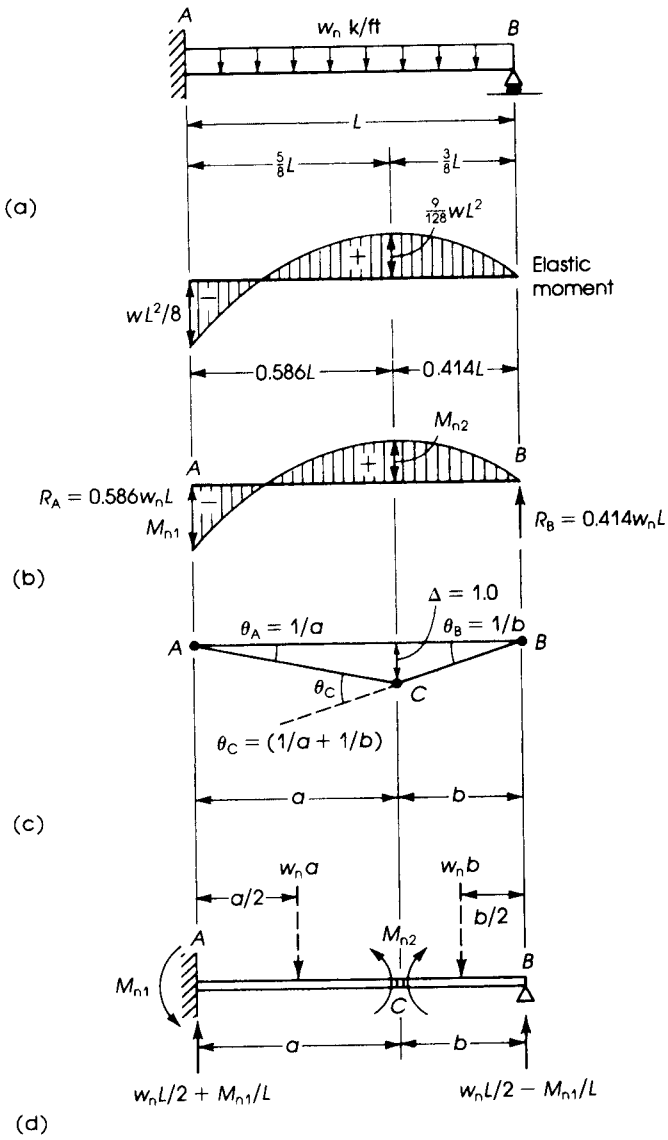


Figure 16.26 Example 16.4: $M_u = \phi M_n$ and $w_u = \phi w_n$.

3. External work is

$$W_e = \sum w_u \Delta = w_u \left(\frac{1 \times L}{2}\right) = \frac{w_u L}{2}$$

Internal work is

$$W_i = \sum \mathrm{m} M_u \theta = M_{u_1}\theta_A + M_{u_2}\theta_c$$

$$= M_{u_1}\left(\frac{1}{a}\right) + M_{u_2}\left(\frac{1}{a} + \frac{1}{b}\right)$$

Equating W_e and W_i,

$$w_u = \frac{2}{L}\left(\frac{M_{u_1}}{a} + \frac{M_{u_2}}{a} + \frac{M_{u_2}}{L-a}\right) \tag{16.9}$$

If both moments are equal, then

$$w_u = \frac{2M_u}{L}\left[\frac{2}{a} + \frac{1}{(L-a)}\right] = \frac{2M_u}{L}\left[\frac{(2L-a)}{a(L-a)}\right] \tag{16.10}$$

4. To determine the position of the plastic hinge at C that produces the minimum value of the collapse load w_u, differentiate Eq. 16.9 with respect to a and equate to 0:

$$\frac{\delta w_u}{\delta a} = 0 \quad -\left(\frac{M_{u_1}}{a^2} + \frac{M_{u_2}}{a^2} - \frac{M_{u_2}}{(L-a)^2}\right) = 0$$

If $M_{u_1} = M_{u_2} = M_u$, then

$$\frac{2}{a^2} = \frac{1}{(L-a)^2} \quad \text{or} \quad a = L(2-\sqrt{2}) = 0.586L$$

From Eq. 16.10, the collapse load is $w_u = 11.66\ (M_u/L^2)$, and the collapse moment is $M_u = 0.0858 w_u L^2$. The reaction at A is $0.586 w_u L$, and the reaction at B is $0.414 w_u L$.

In the equilibrium method, the equilibrium of the beam or of separate segments of the beam is studied under the forces present at collapse. To illustrate analysis by this method, the two previous examples are repeated here.

Example 16.5

For the beam shown in Fig. 16.25, calculate the collapse moments using the equilibrium method.

Solution

Two plastic hinges will develop at A and C. Referring to Fig. 16.25*e*, the reaction at A is $(P_u/2) + (M_{u_1}L)$ and the reaction at B is $(P_u/2) - (M_{u_1}/L)$.
Considering the equilibrium of beam BC and taking moments about C,

$$\left(\frac{P_u}{2} - \frac{M_{u_1}}{L}\right)\left(\frac{L}{2}\right) = M_{u_2}$$

$$M_{u_1} + 2M_{u_2} = P_u \frac{L}{2}$$

which is the same equation obtained in Example 16.3. When $M_{u_1} = M_{u_2} = M_u$, then

$$3M_u = P_u \frac{L}{2}$$

or $$M_u = P_u \frac{L}{6}$$

Example 16.6

Calculate the collapse moments for the beam shown in Fig. 16.26 by the equilibrium method.

Solution

1. Two plastic hinges will develop in this beam at A and C. Referring to Fig. 16.26d, the reaction at $A = w_u(L/2) + (M_{u_1}/L)$ and the reaction at $B = w_u(L/2) - (M_{u_1}/L)$. The load on BC is $w_u b$ acting at $b/2$ from B, and $b = (L - a)$. Considering the equilibrium of segment BC and taking moments about C,

$$\left(w_u \frac{L}{2} - \frac{M_{u_1}}{L}\right) b - (w_u b)\frac{b}{2} = M_{u_2}$$

If $M_{u_1} = M_{u_2} = M_u$, then

$$w_u \frac{b}{2}(L - b) = M_u\left(1 + \frac{b}{L}\right) = \frac{M_u}{L}(2L - a)$$

$$w_u = \frac{2M_u}{L} \times \frac{(2L - a)}{a(L - a)}$$

which is similar to the results obtained in Example 16.4.

$$M_u = \frac{w_u L}{2} \times \frac{a(L - a)}{(2L - a)}$$

2. The position of a can be determined as before, where $a = 0.586L$, $M_u = 0.0858 w_u L_2$, and $w_u = 11.66(M_u/L^2)$.

16.12 ROTATION OF PLASTIC HINGES

16.12.1 Plastic Hinge Length

The assumption that the inelastic rotation of concrete occurs at the point of maximum moment while other portions of the member act elastically is a theoretical one; in fact, the plastic rotation occurs on both sides of the maximum moment section over a finite length. This length is called the plastic hinge length, l_p. The hinge length, l_p, is a function of the effective depth d, and the distance from the section of highest moment to the point of contraflexure (zero moment).

Referring to Fig. 16.27a, the length $L_p/2$ represents the plastic hinge length on one side of the center of support. M_u and ϕ_u indicate the factored moment and ultimate curvature at the critical section, whereas M_y and ϕ_y indicate the moment and curvature at first yield. The plastic curvature at the critical section ϕ_p is equal to $\phi_u - \phi_y$) and the rotation capacity is equal to $(\phi_p l_p)$.

The estimated length of the plastic hinge was reported by many investigators. A. L. L. Baker [7] assumed that the length of the plastic hinge is approximately equal to the effective depth d. Corley [12] proposed the following expression for the equivalent length of the plastic hinge:

$$l_p = 0.5d + 0.2\sqrt{d}\left(\frac{z}{d}\right) \tag{16.11}$$

where z = distance of the critical section to the point of contraflexure and d = effective depth of the section. Mattock [13] suggested a simpler form:

$$l_p = 0.5d + 0.05z \tag{16.12}$$

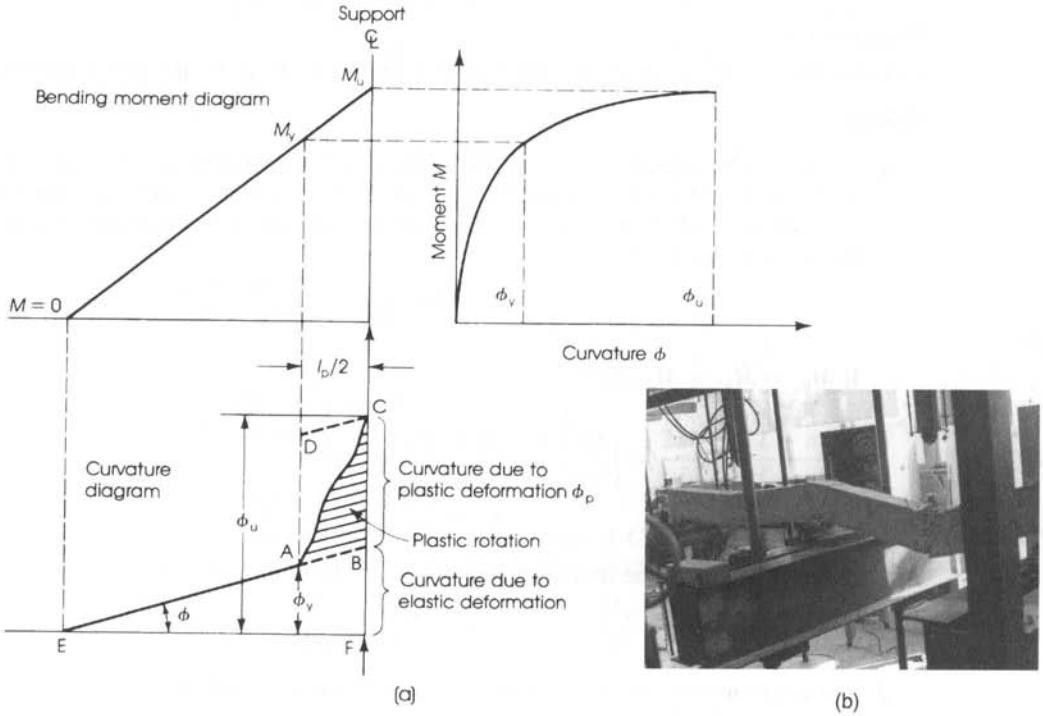


Figure 16.27 (*a*) Plastic rotation from moment-curvature and moment gradient and (*b*) development of plastic hinges in a reinforced concrete continuous beam.

Tests [14] on reinforced concrete continuous beams showed that l_p can be assumed equal to $1.06d$. They also showed that the length of the plastic hinge, in reinforced concrete continuous beams containing hooked-end steel fibers, increases with the increase in the amount of the steel fibers and the main reinforcing steel according to the following expression:

$$l_p = (1.06 + 0.13\rho\rho_s)d \tag{16.13}$$

where ρ = percentage of main steel in the section and ρ_s = percentage of steel fibers by volume, $0 \leq \rho_s \leq 1.2$. For example, if $\rho = 1.0\%$ and $\rho_s = 0.8\%$, then $l_p = 1.164d$.

16.12.2 Curvature Distribution Factor

Another important factor involving the calculation of plastic rotations is the curvature distribution factor, β. The curvature along the plastic hinge varies significantly, and in most rotation estimations this factor is ignored, which leads to an overestimation of the plastic rotations. Referring to Fig. 16.27, the shaded area, ABC, represents the inelastic rotation that can occur at the plastic hinge, whereas the unshaded area, EBF, represents the elastic contribution to the rotation over the length of the member. The shaded area ABC can be assumed to be equal to β times the total area $ABCD$ within the plastic hinge length, $l_p/2$, on one side of the critical section. The curvature distribution factor, β, represents the ratio of the actual plastic rotation, θ_{pc}, to ϕl_p, where ϕ is the curvature and l_p is the length of the plastic hinge. The value of β was reported to vary between 0.5 and 0.6. Tests [14] have showed that β can be assumed to be equal to 0.56.

When hooked-end steel fibers were used in concrete beams, the value of β decreased according to the following expression:

$$\beta = 0.56 - 0.16\rho_s \tag{16.14}$$

where ρ_s is the percentage of steel fibers, $0 \leq \rho_s \leq 1.2\%$. The reduction of the curvature distribution factor of fibrous concrete does not imply that the rotation capacity is reduced: The plastic curvature of fibrous concrete is substantially higher than that of concrete without fibers. Figure 16.28 shows the distribution of the curvature along the plastic hinge length. The area ABC_1 represents the plastic rotation for concrete that does not contain steel fibers, $\beta = 0.56$, whereas the areas ABC_2 and ABC_3 represent the plastic rotation for concretes containing 0.8% and 1.2% steel fibers, respectively.

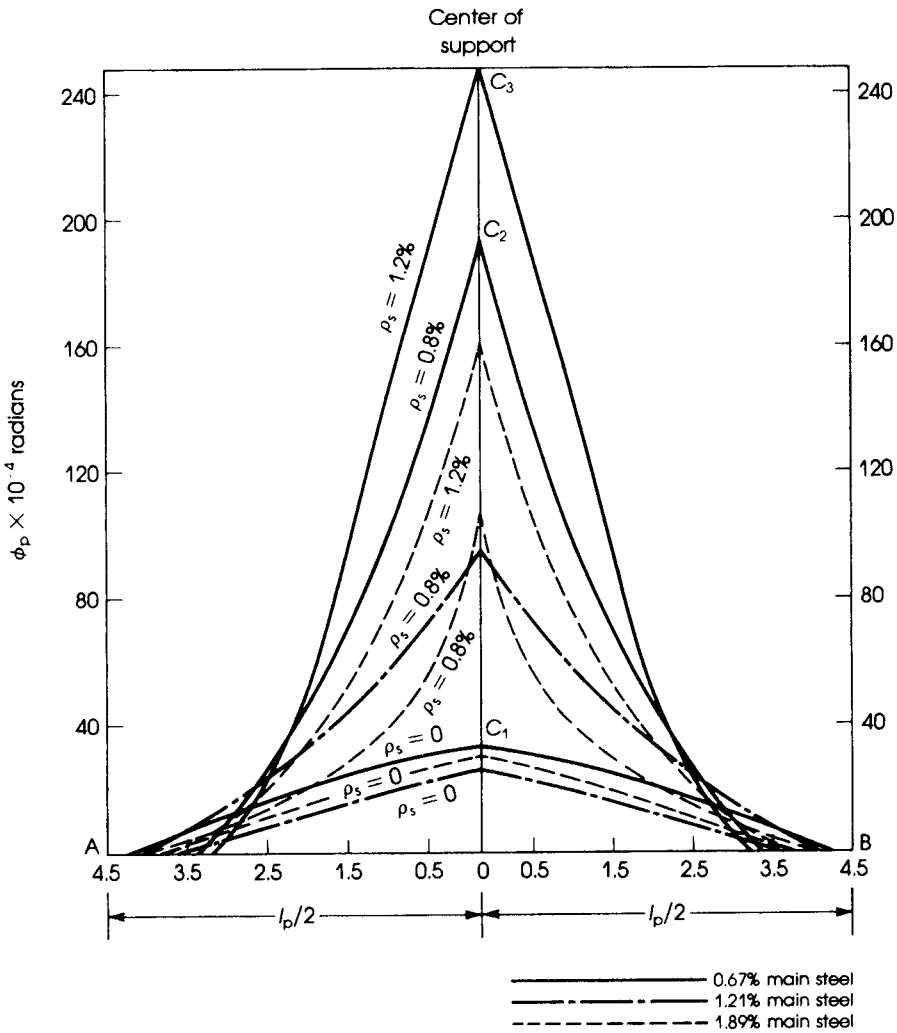


Figure 16.28 Curvature distribution along the plastic hinge.

16.12.3 Ductility Index

The ratio of ultimate to first-yield curvature is called the *ductility index*, $\mu = \phi_u/\phi_y$. The ductility index of reinforced concrete beams was reported [15] to vary between 4 and 6. If steel fibers are used in concrete beams, the ductility index increases according to the following expression [14]:

$$\mu' = (1.0 + 3.8\rho_s)\mu \tag{16.15}$$

where

μ = the ratio of ultimate to first-yield curvature
μ' = ductility index of the fibrous concrete
ρ_s = percentage of steel fibers by volume, $0 \leq \rho_s \leq 1.2\%$.

16.12.4 Required Rotation

The rotation of a plastic hinge in a reinforced concrete indeterminate structure is required to allow other plastic hinges to develop, and the structure to reach a mechanism can be determined by slope deflection from the following expression [7,20]. For a segment AB between two plastic hinges, the rotation at A is

$$\theta_A = \frac{L}{6E_cI}[2(M_A - M_{FA}) + (M_B - M_{FB})] \tag{16.16}$$

where

M_A and M_B = factored moments at A and B, respectively
M_{FA} and M_{FB} = elastic fixed-end moments at A and B
E_c = modulus of elasticity of concrete = $33w^{1.5}\sqrt{f'_c}$
I = moment of inertia of a cracked section (Chapter 5)

16.12.5 Rotation Capacity Provided

Typical tensile plastic hinges at the support and midspan sections of a frame are shown in Fig. 16.29. The rotation capacity depends mainly on the following:

1. The ultimate strain capacity of concrete, ε'_c, which may be assumed to be 0.003 or 0.0035, as used by Baker [7].
2. The length, l_p, over which yielding occurs at the plastic hinge, which can be assumed to be approximately equal to the effective depth of the section where the plastic hinge developed ($l_p = d$).
3. The depth of the compressive block c in concrete at failure at the section of the plastic hinge. Baker [7] estimated the angle of rotation, θ, of a tensile plastic hinge as follows:

$$\theta = \frac{\varepsilon_p l_p}{c} \tag{16.17}$$

where ε_p is the increase in the strain in the concrete measured from the initial yielding of steel reinforcement in the section (see Fig. 16.29*c*):

$$\varepsilon_p = \varepsilon'_c - \varepsilon_{c_1} = 0.0035 - \varepsilon_{c_1}$$

If $l_p = d$ and the ratio c/d equals $\lambda \leq 0.5$,

$$\theta = \frac{(0.0035 - \varepsilon_{c_1})d}{\lambda d} = \frac{0.0035 - \varepsilon_{c_1}}{\lambda}$$

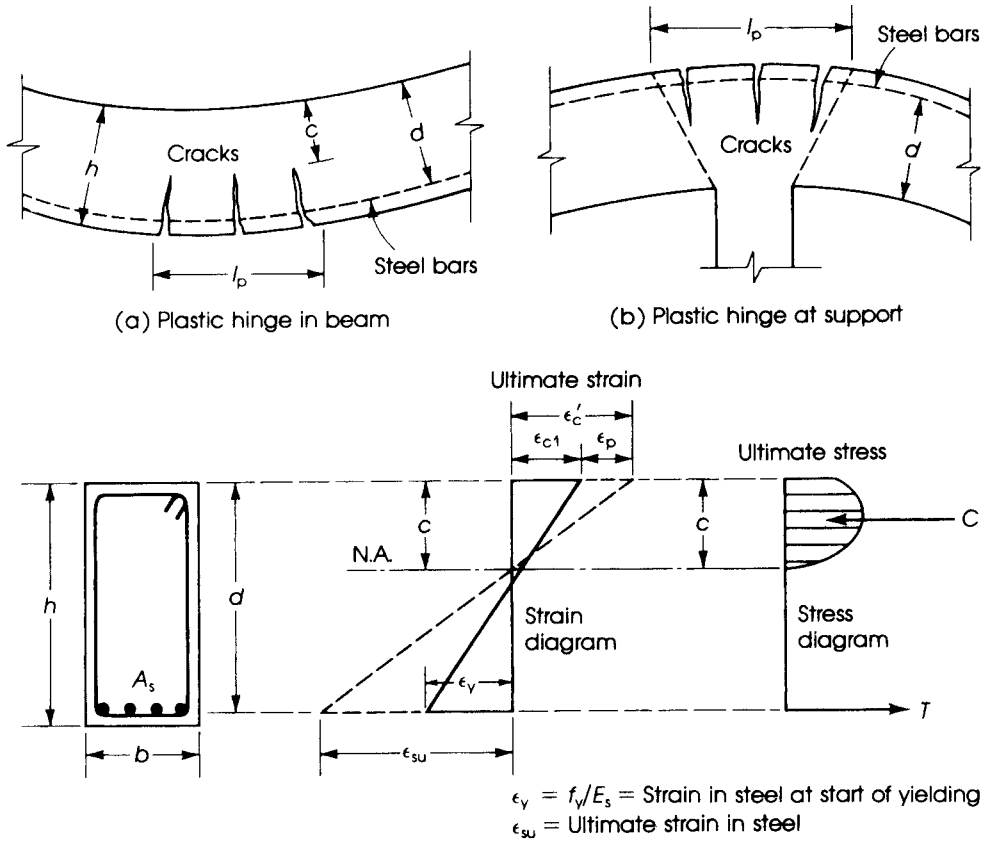


Figure 16.29 Plastic hinge and typical stress and strain distribution [2].

From strain triangles (Fig. 16.29),

$$\varepsilon_{c_1} = \varepsilon_y \left(\frac{c}{d-c}\right) = \frac{f_y}{E_s}\left(\frac{\lambda d}{d-\lambda d}\right) = \frac{f_y}{E_s}\left(\frac{\lambda}{1-\lambda}\right)$$

where f_y = yield strength of steel bars and E_s = modulus of elasticity of steel = 29×10^6 psi. Therefore,

$$\theta = \frac{0.0035}{\lambda} - \frac{\varepsilon_{c_1}}{\lambda} = \frac{0.0035}{\lambda} - \frac{f_y}{E_s(1-\lambda)} \tag{16.18}$$

For grade 40 steel, $f_y = 40$ ksi, and using a maximum value of λ of 0.50, then

$$\theta_{\min} = \frac{0.0035}{0.50} - \frac{40}{29{,}000 \times (1-0.50)} = 0.00424 \text{ rad}$$

For grade 60 steel, $f_y = 60$ ksi and $\lambda_{\max} = 0.44$;

$$\theta_{\min} = \frac{0.0035}{0.44} - \frac{60}{29{,}000(1-0.44)} = 0.00426 \text{ rad}$$

The $\theta_{\min}$ calculated here is from one side only, and the total permissible rotation at the plastic hinge equals 2θ or $2\theta_{\min}$. The actual λ can be calculated as follows, given $\alpha = \beta_1 c$ and $\beta_1 = 0.85$ for $f'_c \leq 4$ ksi:

$$c = \frac{a}{0.85} = \frac{A_s f_y}{(0.85)^2 f'_c b}$$

$$\lambda = \frac{c}{d} = \frac{A_s f_y}{0.72 f'_c bd} = \frac{\rho f_y}{0.72 f'_c} \leq 0.5 \tag{16.19}$$

where $\rho = A_s/bd$. ($\lambda_{\max}$ is obtained when $\rho_{\max}$ is used.)

If the rotation provided is not adequate, one can increase the section dimensions or reduce the percentage of steel reinforcement to obtain a smaller c, a smaller λ, and greater θ. A. L. L. Baker [3] indicated that if special binding or spirals are used, the ultimate crushing strain in bound concrete may be as high as 0.012.

For a compression plastic hinge (as in columns),

$$\theta = \frac{\varepsilon_p l_p}{h} \tag{16.20}$$

where h = overall depth of the section and l_p = length over which yielding occurs. In compression hinges, l_p varies between $0.5h$ and h.

At a concrete ultimate stress of f'_c, $\varepsilon_c = 0.002$; thus, $\varepsilon_p = \varepsilon'_c - 0.002 = 0.0035 - 0.002 = 0.0015$ is the minimum angle of rotation on one side. Therefore,

$$\theta_{\min} = \frac{0.0015 \times 0.5h}{h} = 0.00075 \text{ rad}$$

With special binding or spirals, θ may be increased to

$$\theta_{\max} = (0.012 - 0.002) \times \frac{0.5h}{h} = 0.005 \text{ rad}$$

The extreme value of $\varepsilon'_c = 0.012$ is quite high, and a smaller value may be used with proper spirals; otherwise a different section must be adopted.

In reinforced concrete continuous beams containing steel fibers, the plastic rotation may be estimated as follows [14]:

$$\theta_p = \lambda\beta \left(\frac{0.0035}{\lambda} - \frac{f_y}{E_s(1-\lambda)} \right) \tag{16.21}$$

where

$$\lambda = (4.3 + 2.24\rho_s - 0.043 f_y + 4.17\rho\rho_s) \tag{16.22}$$

$$\beta = 0.56 - 0.16\rho_s \tag{16.14}$$

f_y = yield strength of steel, ksi

E_s = modulus of elasticity of main steel

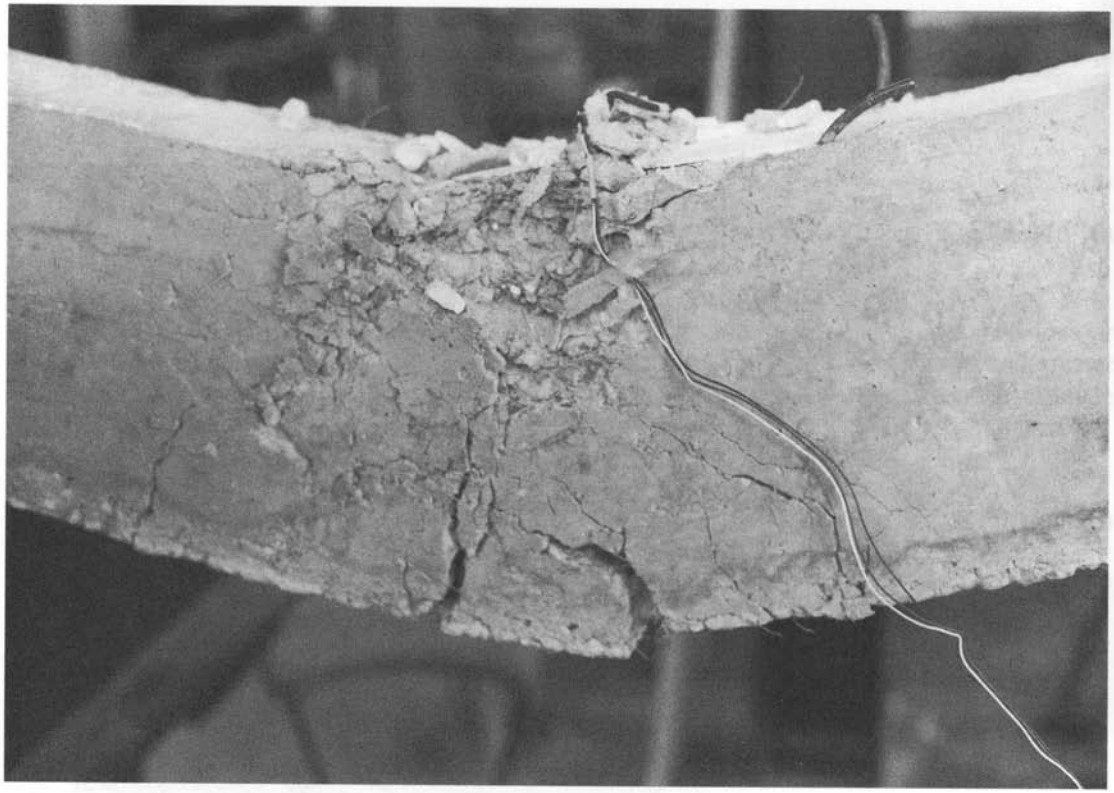
ρ = percentage of main steel

ρ_s = percentage of steel fibers

From Eq. 16.21, it is obvious that the plastic rotation of fibrous reinforced concrete is dependent upon the percentage of steel fibers and percentage of the main steel and its yield strength. Raising the yield strength of the main steel reduces the plastic rotation. Equation 16.21 also includes the effect of the plastic hinge length on rotation.



Plastic hinge in the maximum negative moment region.



Plastic hinge in the maximum positive moment region.

A simplified form can be presented [14]:

$$\theta_p = \lambda\beta\left(\frac{0.003}{\lambda}\right) \tag{16.23}$$

For example, if $\rho_s = 0$ and $f_y = 60\,\text{ksi}$, then $\theta_{p_1} = 0.00289/\lambda$, and if $\rho_s = 1.0\%$, $\rho = 1.5\%$, and $f_y = 60\,\text{ksi}$, then $\theta_{p_2} = 0.01222/\lambda$. This means that the rotation capacity of a concrete beam may be increased by about four times if 1% of steel fibers is used.

16.13 SUMMARY OF LIMIT DESIGN PROCEDURE

1. Compute the factored loads using the load factors given in Chapter 3:

$$w_u = 1.2D + 1.6L$$

2. Determine the mechanism, plastic hinges, and factored moments M_u.
3. Design the critical sections using the strength design method.
4. Determine the required rotation of plastic hinges.
5. Calculate the rotation capacity provided at the sections of plastic hinges. The rotation capacity must exceed that required.

6. Check the factor against yielding of steel and excessive cracking, that is, ϕM_u/elastic moment at service load.
7. Check deflection and cracking under service loads.
8. Check that adequate shear reinforcement is provided at all sections.

For more details, see Ref. 21.

Example 16.7

The beam shown in Fig. 16.30 is fixed at both ends and carries a uniform factored load of 5.5 K/ft, and a concentrated factored load of 48 K. Design the beam using the limit design procedure. Use $b = 14$ in., $f'_c = 3$ ksi, and $f_y = 40$ ksi.

Solution

1. Factored uniform load $w_u = 5.5$ K/ft. Factored concentrated load $P_u = 48$ K.
2. The plastic hinges will develop at A, B, and C, causing the mechanism shown in Fig. 16.30. Using the virtual work method of analysis and assuming a unit deflection at C, then the external work is equal to

$$W_e = 48 \times 1 + 5.5\left(24 \times \frac{1}{2}\right) = 114 \text{ K·ft}$$

The internal work absorbed by the plastic hinges is

$$W_i = M_u\theta(\text{at } A) + M_u\theta(\text{at } B) + M_u(2\theta) \text{ at } C$$

$$= 4M_u\theta = 4M_u\left(\frac{1}{12}\right) = \frac{M_u}{3}$$

Equating W_e and W_i gives $M_u = 342$ K·ft. The general analysis gives directly

$$M_u = \frac{w_u L^2}{16} + P_u\frac{L}{8} = \frac{5.5}{16}(24)^2 + 48\frac{24}{8} = 342 \text{ K·ft}$$

3. Design the critical sections at A, B, and C for $M_u = 342$ K·ft. From tables in Appendix A and for $f'_c = 3$ ksi, $f_y = 40$ ksi, and a steel percentage $\rho = 0.013$, $R_u = 420$ psi ($\rho_{\max} = 0.0203$).

$$M_u = R_u b d^2$$

$$342 \times 12 = 0.42 \times 14(d)^2$$

$d = 26.4$ in. and the total depth is $h = 26.4 + 2.5 = 28.9$ in., say, 29 in.

$$A_s = \rho bd = 0.013 \times 14 \times 26.4 = 4.8 \text{ in.}^2$$

Use five no. 9 bars in one row; A_s provided $= 5.0$ in.^{2}, $b_{\min} = 13.875$ in. < 14 in.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{5.0 \times 40}{0.85 \times 3 \times 14} = 5.6 \text{ in.}$$

$$c = \frac{a}{0.85} = 6.6 \text{ in.} \quad \lambda = \frac{c}{d} = \frac{6.6}{26.4} = 0.25$$

4. The required rotation of plastic hinges is as follows:

a.

$$\theta_A = \frac{L}{6E_c I}[2(M_A - M_{FZ}) + (M_B - M_{FB})]$$

$$E_c = 57{,}400\sqrt{f'_c} = 3.144 \times 10^6 \text{ psi}$$

$$E_s = 29 \times 10^6 \text{ psi} \quad \text{and} \quad n = \frac{E_s}{E_c} = 9.2$$

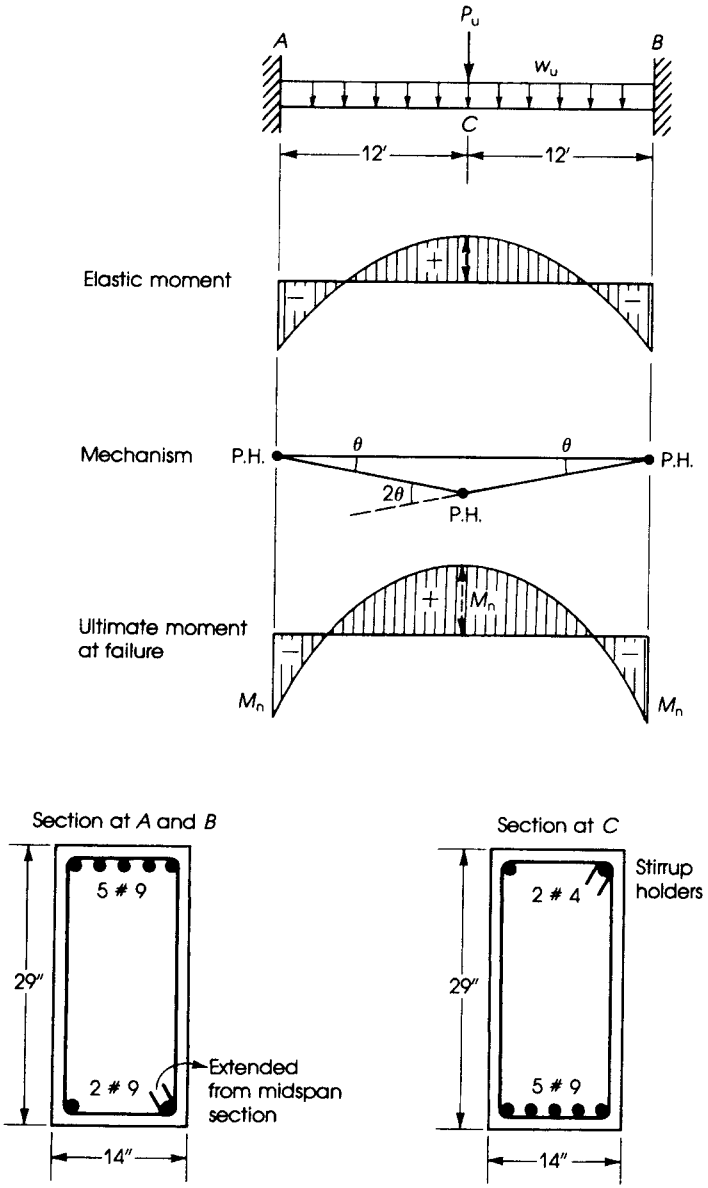


Figure 16.30 Example 16.7.

b. Determine the fixed end moments at A and B using factored loads:

$$M_{FA} = M_{FB} = \frac{w_u L^2}{12} \text{ (uniform load)} + \frac{P_u L}{8} \text{ (concentrated load)}$$

$$= 5.5\frac{(24)^2}{12} + 48 \times \frac{24}{8} = 408 \text{ K·ft}$$

Plastic M_A = plastic M_B = 342 K·ft

c. The cracked moment of inertia can be calculated from

$$I_{cr} = b\frac{x^3}{3} + nA_s(d-x)^2$$

where x is the distance from compression fibers to the neutral axis (kd). To determine x (see Chapter 6), $x = 10.3$ in. and $I_{cr} = 17{,}172\,\text{in.}^4$

d. Required minimum rotation: Considering all moments at supports A and B are negative, then

$$\theta_A = \frac{24 \times 12}{6 \times 3.144 \times 10^6 \times 17{,}172}[2(-342+408)+(-342+408)](12{,}000) = 0.00211 \text{ rad}$$

5. The rotation capacity provided is

$$\theta_A = \frac{0.0035}{\lambda} - \frac{f_y}{E_s(1-\lambda)} = \frac{0.0035}{0.25} - \frac{40}{29{,}000(1-0.25)}$$
$$= 0.0122 \text{ rad} > 0.00211 \quad \text{required}$$

The rotation capacity provided is about 5.5 times that required, indicating that the section is adequate.

6. Check the ratio of factored to elastic moment at service load:

$$M_A = M_B = \frac{wL^2}{12} + \frac{PL}{8}$$
$$= 3.5\frac{(24)^2}{12} + \frac{30 \times 24}{8} = 258 \text{ K·ft}$$

Actual $\phi M_n = \phi A_s f_y[d - (a/2)] = 0.9 \times 5 \times 40[26.5 - (5.6/2)]/12 = 356$ K·ft. The ratio is $356/258 = 1.38$, which represents the factor of safety against the yielding of steel bars at the support.

7. Check maximum deflection due to service load (at midspan): Let the uniform service load (w) = 3.5 K/ft, and $P = 30$ K then:

$$\Delta_1 = \frac{wL^4}{384EI}$$

For a concentrated load at midspan,

$$\Delta_2 = \frac{PL^3}{192EI}$$

and total deflection is

$$\Delta = \frac{(3500/12)(24 \times 12)^4}{384(17{,}172)(3.144 \times 10^6)} + \frac{30{,}000(24 \times 12)^3}{192(17{,}172)(3.144 \times 10^6)} = 0.166 \text{ in.}$$

$$\frac{\Delta}{L} = \frac{0.166}{24 \times 12} = \frac{1}{1735}$$

which is a very small ratio.

8. Adequate shear reinforcement must be provided to avoid any possible shear failure.

16.14 MOMENT REDISTRIBUTION OF MAXIMUM NEGATIVE OR POSITIVE MOMENTS IN CONTINUOUS BEAMS

Moment redistribution of maximum positive or negative moments in continuous flexural members is based on the net tensile strain (NTS), ε_t, for both reinforced and prestressed concrete members. Figure 16.31 shows the permissible limits on moment redistribution. It indicates that the percentage decrease in the negative moments at supports and positive moments between supports of continuous beam, q', calculated by the elastic theory, must not exceed $1000\varepsilon_t\%$, with maximum of 20%. Moment redistribution is allowed only when $\varepsilon_t \geq 0.0075$, indicating adequate ductility is available at the section at which moment is reduced. When $\varepsilon_t < 0.0075$, no moment redistribution is allowed. The modified negative moments must be used to calculate the modified positive moments within the span, ACI Code, Section 8.4. Moment redistribution does not apply to members designed by the direct design method for slab systems. (Refer to Chapter 17.)

In summary, the percentage of decrease in maximum negative or positive moments in continuous beams is as follows:

1. When $\varepsilon_t \geq 0.0075$, moment redistribution is allowed. $(\rho/\rho_b > 0.476)$
2. When $\varepsilon_t = 0.0075$, the percentage of moment redistribution is 75% $(\rho/\rho_b = 0.476)$.
3. When $\varepsilon_t \geq 0.020$, the percentage of moment redistribution is 20% $(\rho/\rho_b = 0.217)$.
4. When $0.0075 < \varepsilon_t < 0.020$, the percentage of moment redistribution is:

$$q' = 1000\varepsilon_t \tag{16.24}$$

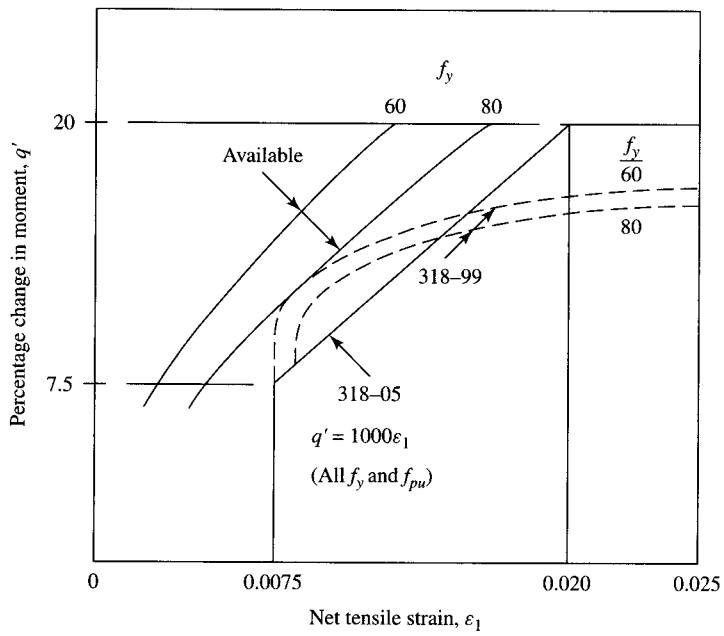


Figure 16.31 Permissible moment redistribution for minimum rotation capacity [22]. Courtesy of ACI-PCA.

Table 16.1 Percentage Change in Moment Redistribution (q'), $f_y = 60$ ksi

ε_t	0.0075	0.0100	0.0125	0.0150	0.0175	0.0200	0.0225
ρ/ρ_b	0.476	0.385	0.323	0.278	0.244	0.217	0.196
q' %	7.5	10.0	12.5	15.0	17.5	20.0	20.0

For example, if $\varepsilon_t = 0.010$, then the percentage of moment redistribution is 10%. The relationship between the steel percentage, ρ, in the section and the net tensile strain, ε_t, is as follows (refer to Section 3.10):

$$\varepsilon_t = \left[\frac{\left(0.003 + \frac{f_y}{E_s}\right)}{\left(\frac{\rho}{\rho_b}\right)}\right] - 0.003 \tag{3.24}$$

For grade 60 steel, $f_y = 60$ ksi and $E_s = 29{,}000$ ksi. Assuming $f_y/E_s = 0.002$, then

$$\varepsilon_t = \left[\frac{0.005}{\left(\frac{\rho}{\rho_b}\right)}\right] - 0.003 \tag{3.25}$$

For $\varepsilon_t = 0.0075$, the ductility limit $\varepsilon_t/\varepsilon_y = 0.0075/0.002 = 3.75$. The percentage change in moment redistribution according to these limitations and for $f_y = 60$ ksi given in Tables 16.1 and 16.2.

Whatever percentage of moment redistribution is used, it is essential to ensure that no sections is likely to suffer local damage or excessive cracking at service loads and that adequate rotation capacity is maintained at every critical section in the structure. The redistribution of moments in a statically indeterminate structure will result in a reduction in the negative moments at the supports and in the positive moments within the spans. This reduction will not imply that the safety of the structure has been reduced or jeopardized as compared with determinate structures. In fact, continuity in structures provides additional strength, stability and economy in the design.

Moment redistribution factor, q, based on the ACI Code 318-02 is calculated as follows:

$$q = 20\left[1 - \frac{(\rho - \rho')}{\rho_b}\right] \tag{16.25}$$

In Eq. 16.25, the steel ratio ρ or $(\rho - \rho')$, at the section where the moment is limited to a maximum ratio of $0.5\rho_b$. The minimum steel ratio in the section, for flexural design is limited

Table 16.2 Percentage Change in Moment Redistribution (q') for a Given ρ/ρ_b Ratio

ρ/ρ_b	0.48	0.45	0.40	0.35	0.30	0.25	0.20
ε_t	0.0074	0.0081	0.0095	0.0113	0.0137	0.017	0.022
q' %	0.0	8.1	9.5	11.3	13.7	17.0	20.0

Table 16.3 Maximum and Minimum Moment Redistribution q (Eq. 16.25)

f'_c (ksi)	f_y (ksi)	ρ_b	ρ_{min}	q_{max} % (for ρ_{min})	q_{min} % (for 0.5 ρ_b)
3	60	0.0215	0.0033	16.9	10
4	60	0.0285	0.0033	17.7	10
5	60	0.339	0.0035	17.9	10

to $3\sqrt{f'_c}/f_y \geq 200/f_y$. Using these extreme limitations, the maximum and minimum moment redistribution percentages are shown in Table 16.3.

Example 16.8

Determine the maximum elastic moments at the supports and midspans of the continuous beam of four equal spans shown in Fig. 16.32*a*. The beam has a uniform section and carries a uniform dead load of 8 K/ft and a live load of 6 K/ft. Assume 10% maximum redistribution of moments and consider the following two cases: (1) When the live load is placed on alternate spans, calculate the maximum positive moments within the spans, and (2) when the live load is placed on adjacent spans, calculate the maximum negative moments at the supports.

Solution

1. The beam has a uniform moment of inertia I and has the same E; thus, EI is constant. The three-moment equation to analyze the beam and for a constant EI is

$$M_A L_1 + 2M_b(L_1 + L_2) + M_c L_2 = -\frac{w_1 L_1^3}{4} - \frac{w_2 L_2^3}{4}$$

Because the spans are equal,

$$M_A + 4M_B + M_C = -\frac{L^2}{4}(w_1 + w_2) \tag{16.26}$$

In this example $M_A = M_E = 0$. Six different cases of loading will be considered, as shown in Fig. 16.31:
Case 1. Dead load is placed on the whole beam *ABCDE* (Fig. 16.32*b*).
Case 2. Live load is placed on *AB* and *CD* for maximum positive moments within *AB* and *CD* (Fig. 16.32*c*).
Case 3. Similar to Case 2 for beams *BC* and *DE* (Fig. 16.32*d*).
Case 4. Live load is placed on *AB*, *BC*, and *DE* for a maximum negative moment at *B* (Fig. 16.32*e*).
Case 5. Live load is placed on spans *CD* and *DE* (Fig. 16.32*f*).
Case 6. Live load is placed on *BC* and *CD* for a maximum negative moment at *C* (Fig. 16.32*g*).

2. Case 1. Apply Eq. 16.26 to the beam segments *ABC*, *BCD*, and *CDE*, respectively:

$$4M_B + M_C = -\frac{(20)^2}{4}(8 + 8) = -1600 \text{ K·ft}$$
$$M_B + 4M_C + M_D = -1600 \text{ K·ft}$$
$$M_C + 4M_D = -1600 \text{ K·ft}$$

Solve the three equations to get

$$M_B = M_D = -342.8 \text{ K·ft} \quad \text{and} \quad M_C = -228.6 \text{ K·ft}$$

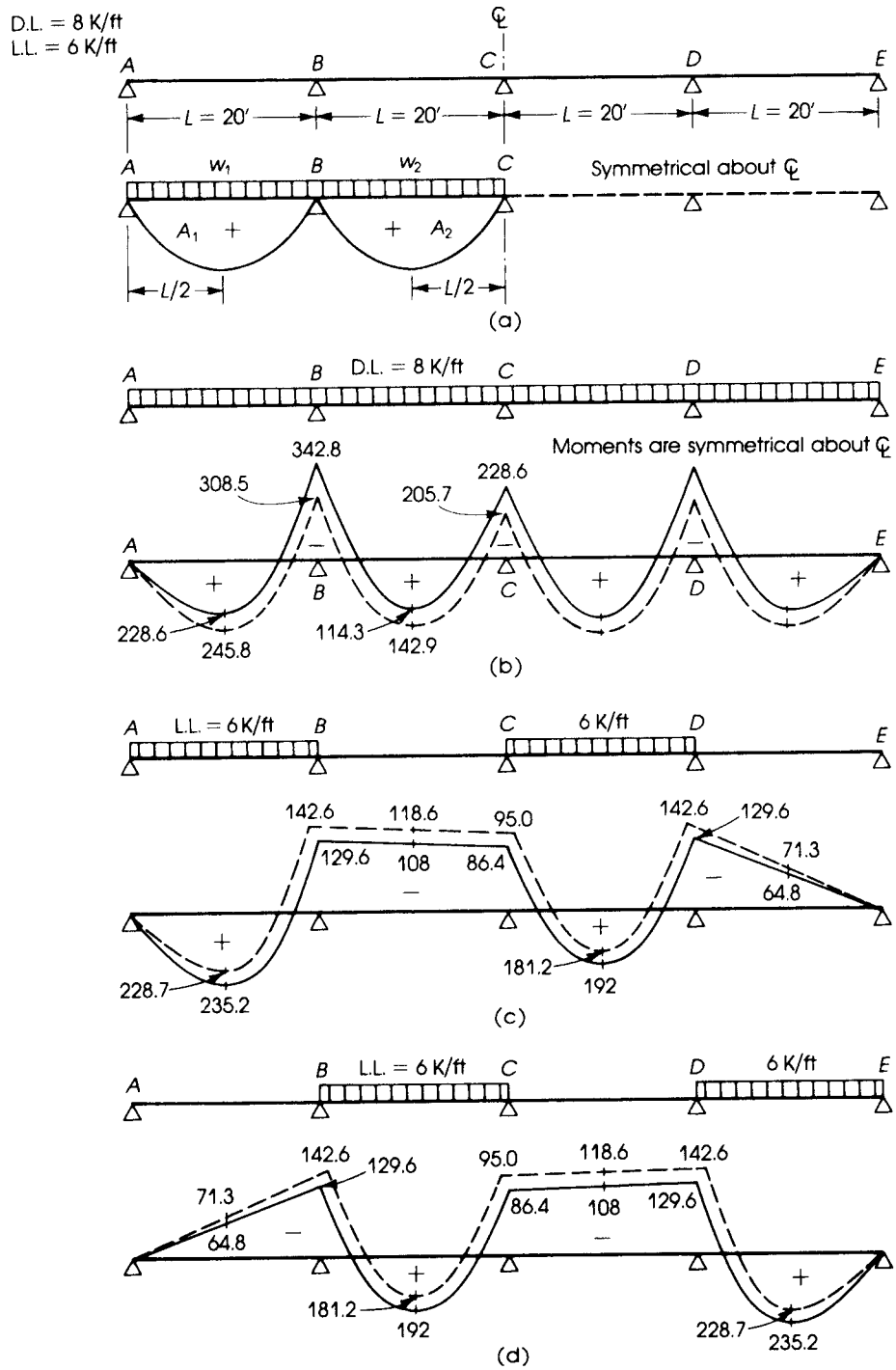
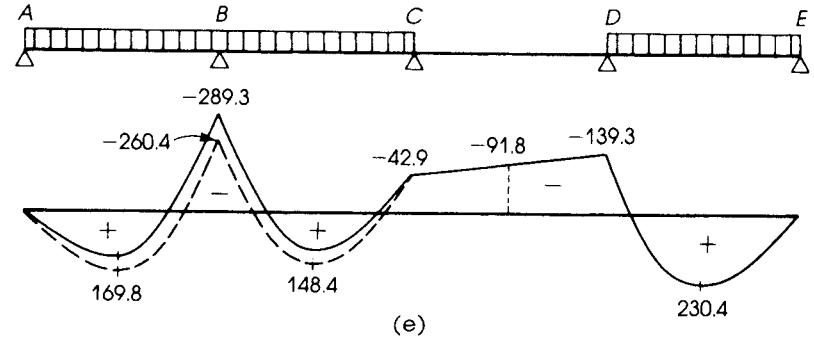
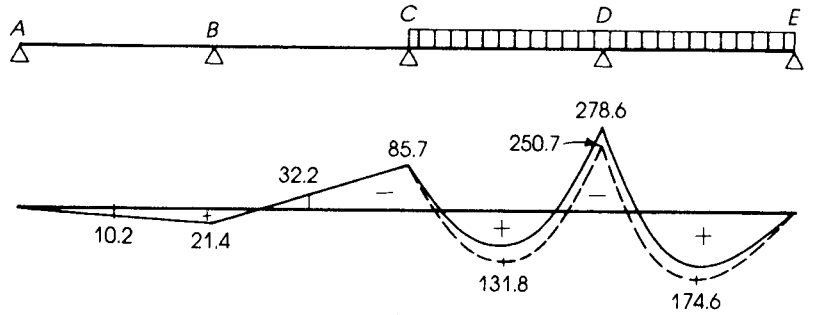


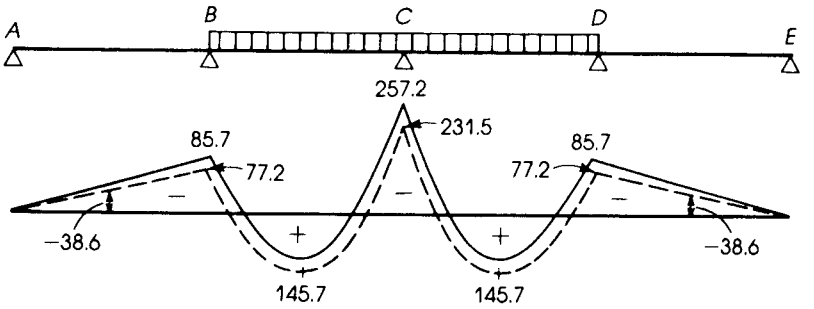
Figure 16.32 Example 16.8: Bending moments are drawn on the tension side.



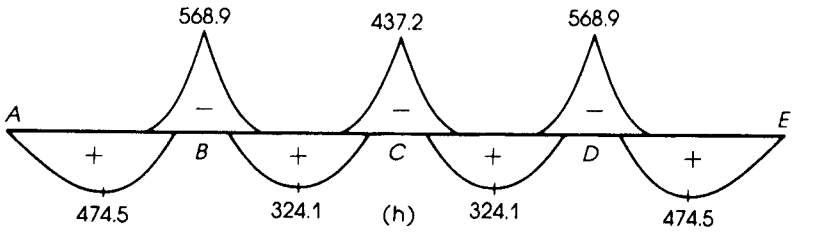
(e)



(f)



(g)



(h)

Figure 16.32 *(continued)*

For a 10% reduction in moments,

$$M'_B = M'_D = 0.9(-342.8) = -308.5 \text{ K·ft}$$
$$M'_C = 0.9(-228.6) = -205.7 \text{ K·ft}$$

The corresponding midspan moments are

$$\text{Span } AB = DE = \frac{w_D L^2}{8} + \frac{1}{2}M_B = \frac{8(20)^2}{8} - \frac{1}{2} \times 308.5 = 245.8 \text{ K·ft}$$

$$\text{Span } BC = CD = \frac{w_D L^2}{8} - \frac{1}{2}(308.5 + 205.7) = \frac{8(20)^2}{8} - 257.1 = 142.9 \text{ K·ft}$$

3. Case 2. Apply Eq. 16.26 to *ABC*, *BCD*, and *CDE*, respectively:

$$4M_B + M_C = -\frac{(20)^2}{4}(6) = -600 \text{ K·ft}$$
$$M_B + 4M_C + M_D = -600 \text{ K·ft}$$
$$M_C + 4M_D = -600 \text{ K·ft}$$

Solve the three equations to get

$$M_B = M_D = -129.6 \text{ K·ft} \qquad M_C = -86.4 \text{ K·ft}$$

The corresponding elastic midspan moments are

$$\text{Beam } AB = \frac{w_L L^2}{8} + \frac{M_B}{2} = \frac{6(20)^2}{8} - \frac{129.6}{2} = +235.2 \text{ K·ft}$$
$$BC = 0 - \frac{1}{2}(129.6 + 86.4) = -108 \text{ K·ft}$$
$$CD = \frac{w_L L^2}{8} - \frac{1}{2}(129.6 + 86.4) = \frac{6(20)^2}{8} - 108 = +192 \text{ K·ft}$$
$$DE = 0 - \frac{1}{2} \times 129.6 = -64.8 \text{ K·ft}$$

To reduce the positive span moment, increase the support moments by 10% and calculate the corresponding positive span moments. The resulting positive moment must be at least 90% of the first calculated moments given previously.

$$M'_B = M'_D = 1.1(-129.6) = -142.6 \text{ K·ft}$$
$$M'_C = 1.1(-86.4) = -95.0 \text{ K·ft}$$

The corresponding midspan moments are

$$\text{Beam } AB = \frac{w_L L^2}{8} + \frac{M'_B}{2} = \frac{6(20)^2}{8} - \frac{142.6}{2} = +228.7 \text{ K·ft}$$
$$BC = -\frac{1}{2}(142.6 + 95) = -118.8 \text{ K·ft}$$
$$CD = \frac{w_L L^2}{8} + \frac{1}{2}(M'_C + M'_D) = \frac{6(20)^2}{8} - \frac{1}{2}(95 + 142.6) = 181.2 \text{ K·ft}$$
$$DE = -\frac{1}{2} \times 142.6 = -71.3 \text{ K·ft}$$

4. Case 3. This case is similar to Case 2, and the moments are shown in Fig. 16.32*d*.

5. Case 4. Consider the spans AB, BC, and DE loaded with live load to determine the maximum negative moment at support B:

$$4M_B + M_C = -\frac{w_L L^2}{2} = -\frac{6(20)^2}{2} = -1200 \text{ K·ft}$$

$$M_B + 4M_C + M_D = -\frac{w_L L^2}{4} = -\frac{6(20)^2}{4} = -600 \text{ K·ft}$$

$$M_C + 4M_D = -\frac{6(20)^2}{4} = -600 \text{ K·ft}$$

Solve the three equations to get

$$M_C = -42.9 \text{ K·ft}$$

$$M_B = -289.3 \text{ K·ft}$$

$$M_D = -139.3 \text{ K·ft}$$

For 10% reduction in moment at support B,

$$M'_B = 0.9 \times (-289.3) = -260.4 \text{ K·ft}$$

The corresponding midspan moments are

$$\text{Beam } AB = \frac{w_L L^2}{8} + \frac{M_B}{2} = \frac{6(20)^2}{8} - \frac{260.4}{2} = 169.8 \text{ K·ft}$$

$$BC = \frac{w_L L^2}{8} - \frac{1}{2}(260.4 + 42.9) = 148.4 \text{ K·ft}$$

$$CD = -\frac{1}{2}(42.9 + 139.3) = -91.1 \text{ K·ft}$$

$$DE = 300 - \frac{1}{2} \times 139.3 = +230.4 \text{ K·ft}$$

6. Case 5. This is similar to Case 4, except that one end span is not loaded to produce maximum positive moment at support B (or support D for similar loading). The bending moment diagrams are shown in Fig. 16.32f.

7. Case 6. Consider the spans BC and CD loaded with live load to determine the maximum negative moment at support C:

$$4M_B + M_C = \frac{w_L L^2}{4} = -600 \text{ K·ft}$$

$$M_B + 4M_C + M_D = -\frac{w_L L^2}{2} = -1200 \text{ K·ft}$$

$$M_C + 4M_D = -\frac{w_L L^2}{4} = -600 \text{ K·ft}$$

Solve the three equations to get

$$M_C = -257.2 \text{ K·ft}$$

$$M_B = M_D = -85.7 \text{ K·ft}$$

For 10% reduction in support moments,

$$M'_C = 0.9 \times (-257.2) = -231.5 \text{ K·ft}$$

$$M'_B = M'_C = 0.9 \times (-85.7) = -77.2 \text{ K·ft}$$

Table 16.4 Final Moments of Example 16.8 after Moment Redistribution

Case	1	2	3	4	5
Section Location	D.L. Moments	L.L. Maximum Negative	L.L. Maximum Positive	D.L. + L.L. (1) + (2) Maximum Negative	D.L. + L.L. (1) + (3) Maximum Positive
			Support		
A	0	0	0	0	0
B	−308.5	−260.4	+21.4	−568.9*	−287.1
C	−205.7	−231.5	—	−437.2*	−205.7
D	−308.5	−260.4	+21.4	−568.9*	−287.1
E	0	0	0	0	0
			Midspan		
AB	245.8	−71.3	228.7	±174.5	±474.5*
BC	142.9	−118.6	181.2	±24.3	±324.1*
CD	142.9	−118.6	181.2	±24.3	±324.1*
DE	245.8	−71.3	228.7	±174.5	±474.5*

*Final maximum and minimum design moments.

The corresponding midspan moments are

$$\text{Beam } AB = DE = -\frac{77.2}{2} = -38.6 \text{ K·ft}$$

$$BC = CD = \frac{w_L L^2}{8} - \frac{1}{2}(231.5 + 77.2) = \frac{6(20)^2}{8} - 154.3 = 145.7 \text{ K·ft}$$

8. The final maximum and minimum moments after moment redistribution are shown in Table 16.4. The moment envelope is shown in Fig. 16.32*h*.
9. In this example, the midspan sections are used for simplicity: The midspan moments are not necessarily the maximum positive moments. In the case of the end spans AB and DE, the maximum moment after 10% moment redistribution is equal to $(w_D L_2)/12.2$ and occurs at $0.4L$ from A and D.

Example 16.9

Determine the permissible redistribution of negative moments at supports B, C, D, and E of the continuous beam $ABCDEF$ shown in Fig. 16.33. The beam has a rectangular section, $b = 12$ in., $h = 22$ in., and $d = 19.5$ in., and it is reinforced as shown in the following table ($f'_c = 4$ ksi and $f_y = 60$ ksi).

1. Use Appendix B.
2. Use the ACI Code limitations.

Solution

1. For $f'_c = 4$ ksi and $f_y = 60$ ksi, $\rho_b = 0.0285$. The ACI Code redistribution factor was given as follows:

$$q = 20\left[1 - \frac{\rho - \rho'}{\rho_b}\right] \tag{16.25}$$

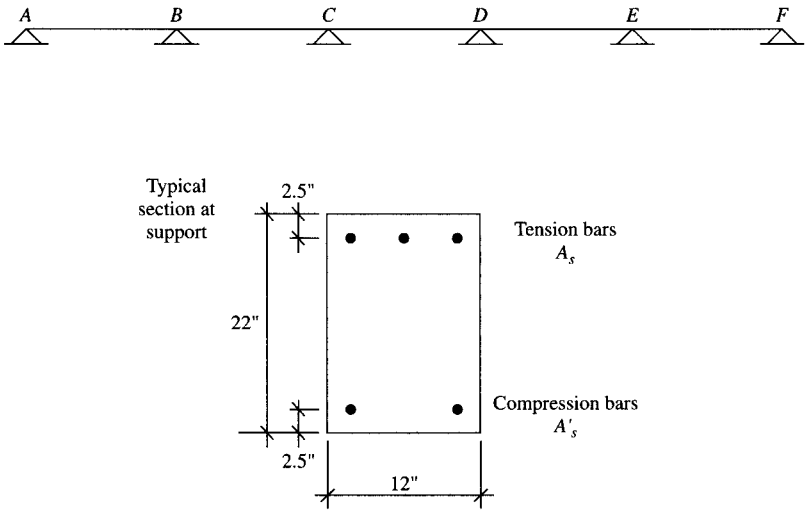


Figure 16.33 Example16.9.

2. The ACI Code redistribution factor is a function of the net tensile strain, ε_t, and varies between 7.5% and 20%, as shown in Fig. 16.31.

$$q' = 1000\varepsilon_t \tag{16.24}$$

$$\varepsilon_t = \frac{0.003 + \dfrac{f_y}{E_s}}{\dfrac{\rho}{\rho_b}} - 0.003$$

and

$$\varepsilon_t = \frac{0.005}{\dfrac{\rho}{\rho_b}} - 0.003 \quad (\text{for } f_y = 60 \text{ ksi})$$

The table shows the values of q and q', which are not compatible.

Support	Tension Bars (A_s)	ρ	Compression Bars (A'_s)	ρ'	$\frac{\rho - \rho'}{\rho_b}$	q%	ε_t	q'
B	3 no. 9	0.01282	0	0	0.45	11.0	0.0113	11.3
C	3 no. 10	0.0160	0	0	0.56	8.8	0.006	0
D	3 no. 6	0.00564	0	0	0.198	16.0	0.0226	20
E	4 no. 8	0.01342	3 no. 6	0.0056	0.273	14.5	0.0153	15.3

SUMMARY

Sections 16.1–16.3

In continuous beams, the maximum and minimum moments are obtained by considering the dead load acting on all spans, whereas pattern loading is considered for live or moving loads,

as shown in Figs. 16.2 and 16.3. The ACI moment coefficients given in Chapter 9 may be used to compute approximate values for the maximum and minimum moments and shears.

Sections 16.4–16.5

A frame subjected to a system of loads may be analyzed by the equivalent frame method. Frames may be statically determinate or indeterminate.

Section 16.6

There are several types of frame hinges: Mesnager, Considére, lead, and concrete hinges. The steel for a Mesnager hinge is calculated as follows:

$$A_{s_1} = \frac{R_1}{0.55 f_y} \quad \text{and} \quad A_{s_2} = \frac{R_2}{0.55 f_y} \tag{16.2}$$

$$\text{Burst force:} \quad F = \frac{P_u}{2} \tan\theta + \frac{Ha}{0.85d} \tag{16.4}$$

$$\text{Stress in ties} \quad f_s = \frac{F}{0.005ab + A_{\text{st}}\ (\text{ties})} \leq 0.85\ f_y \tag{16.6}$$

Sections 16.7–16.8

Limit design in reinforced concrete refers to redistribution of moments, which occurs throughout the structure as steel reinforcement reaches its yield strength. Ultimate strength is reached when the structure is on the verge of collapse. This case occurs when a number of plastic hinges, n, develop in a structure with redundants, r, such that $n = 1 + r$.

Sections 16.9–16.11

For limit design to be valid, four conditions must be satisfied: mechanism, equilibrium, yield, and rotation. Two methods of analysis may be used: the virtual work method and the equilibrium method, which are both explained in Examples 16.3 through 16.6.

Sections 16.12–16.13

The plastic hinge length, l_p, can be considered equal to the effective depth, d. In fibrous concrete,

$$l_p = (1.06 + 0.13\rho\rho_s)d \tag{16.13}$$

$$\text{Ductility index } \mu = \frac{\phi_u}{\phi_y}$$

For fibrous concrete,

$$\mu' = (1.0 + 3.8\rho_s)\mu \tag{16.15}$$

$$\text{Angle of rotation } \theta = \frac{0.0035}{\lambda} - \frac{f_y}{E_s(1-\lambda)} \tag{16.18}$$

$$\lambda = \frac{\rho f_y}{0.72 f_c'} \leq 0.5 \tag{16.19}$$

A summary of the limit design procedure is given in Section 16.14.

Section 16.14

Moment redistribution may be taken into account in the analysis of statically indeterminate structures. In this case, the maximum negative moments calculated by the elastic theory may be increased or decreased by not more than the ratio q', where

$$q' = 1000\varepsilon_t \tag{16.24}$$

Table 16.1 gives the different values of q. Moment redistribution is explained in detail in Example 16.8.

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PROBLEMS

16.1 The slab-beam floor system shown in Fig. 16.34 carries a uniformly distributed dead load (excluding weight of slab and beam) of 40 psf and a live load of 100 psf. Using the ACI Code coefficients, design the interior continuous beam *ABCD* and draw detailed sections. Given: $f'_c = 4$ ksi, $f_y = 60$ ksi, width of beam web = 12 in., slab thickness = 4.0 in., and column dimensions = 14 by 14 in.

16.2 Repeat Problem 16.1 using span lengths of the beams shown in Fig. 16.32 as follows:

$$L_1 = 20 \text{ ft} \quad L_2 = 24 \text{ ft}$$

$$L_3 = 20 \text{ ft} \quad L_4 = 10 \text{ ft}$$

16.3 For the beam shown in Fig. 16.35, compute the reactions at *A*, *B*, and *C* using constant *EI*. Draw the shear and bending moment diagrams and design all critical sections, using $b = 14$ in, $h = 25$ in., $f'_c = 4$ ksi, $f_y = 60$ ksi, and a load factor = 1.6.

16.4 Repeat Problem 16.3 using span lengths of beams as follows: span *AB* = 20 ft and span *BC* = 16 ft.

16.5 The two-hinged portal frame *ABCD* shown in Fig. 16.36 carries a uniform dead load (excluding self-weight) = 2.6 K/ft and a uniform live load of 1.8 K/ft. Design the frame *ABCD*, the hinges, and footings using $f'_c = 4$ ksi, $f_y = 60$ ksi, and a beam width of $b = 16$ in. The footing is placed 5 ft below ground level and the allowable bearing soil pressure is 5 ksf. Use a slab thickness of 6 in.

16.6 Design the portal frame *ABCD* of Problem 16.5 if the frame ends at *A* and *D* are fixed.

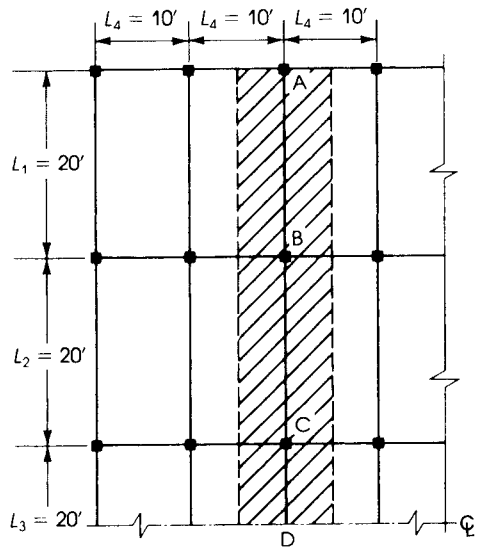


Figure 16.34 Problem 16.1.

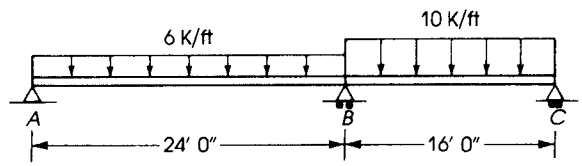


Figure 16.35 Problem 16.3.

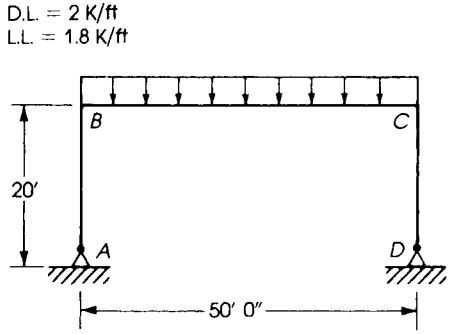


Figure 16.36 Problem 16.5.

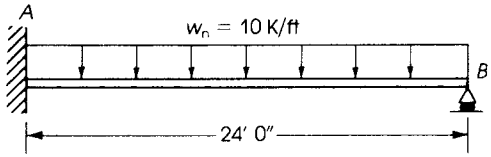


Figure 16.37 Problem 16.7.

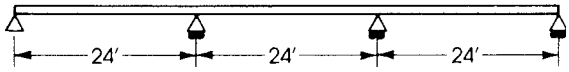


Figure 16.38 Problem 16.8.

16.7 Calculate the collapse moments at the critical sections of the beams shown in Fig. 16.37.

16.8 Repeat Problem 16.7 for Fig. 16.38.

16.9 If the beam shown in Fig. 16.36 carries a uniform dead load of 2.5 K/ft and a live load of 2.4 K/ft, design the beam using the limit design procedure. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, and a beam width of $b = 14$ in.

16.10 Determine the maximum and minimum elastic moments at the supports and midspans of the three-span continuous beam shown in Fig. 16.37. The beam has a uniform rectangular section and carries a uniform dead load of 6 K/ft and a live load of 5 K/ft. Assuming 10% maximum redistribution of moments, recalculate the maximum and minimum moments at the supports and midspans of the beam *ABC*. *Note:* Place the live load on alternate spans to calculate maximum positive moments and on adjacent spans to calculate the maximum negative (minimum) moments (Example 16.8).

16.11 Repeat Problem 16.10 if the beam consists of four equal spans, each 24 ft in length (Fig. 16.39).

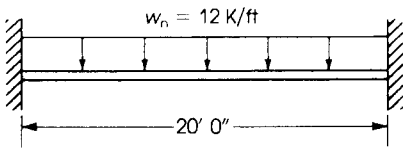


Figure 16.39 Problem 16.11.

CHAPTER 17

DESIGN OF TWO-WAY SLABS



The Bonaventure Complex and the Bonaventure Hilton Hotel, Montreal, Canada.

17.1 INTRODUCTION

Slabs can be considered as structural members whose depth, h, is small as compared to their length, L, and width, S. The simplest form of a slab is one supported on two opposite sides, which primarily deflects in one direction and is referred to as a *one-way slab*. The design of one-way slabs was discussed in Chapter 9.

When the slab is supported on all four sides and the length, L, is less than twice the width, S, the slab will deflect in two directions, and the loads on the slab are transferred to all four supports. This slab is referred to as a *two-way slab*. The bending moments and deflections in such slabs are less than those in one-way slabs; thus, the same slab can carry more load when supported on four sides. The load in this case is carried in two directions, and the bending moment in each direction is much less than the bending moment in the slab if the load were carried in one direction only. Typical slab-beam-girder arrangements of one-way and two-way slabs are shown in Fig. 17.1.

17.2 TYPES OF TWO-WAY SLABS

Structural two-way concrete slabs may be classified as follows:

1. *Two-Way Slabs on Beams:* This case occurs when the two-way slab is supported by beams on all four sides (Fig. 17.1). The loads from the slab are transferred to all four supporting beams, which, in turn, transfer the loads to the columns.
2. *Flat Slabs:* A flat slab is a two-way slab reinforced in two directions that usually does not have beams or girders, and the loads are transferred directly to the supporting columns. The column tends to punch through the slab, which can be treated by three methods (refer to Figs. 17.2 and 17.3):
   a. Using a drop panel and a column capital.
   b. Using a drop panel without a column capital. The concrete panel around the column capital should be thick enough to withstand the diagonal tensile stresses arising from the punching shear.

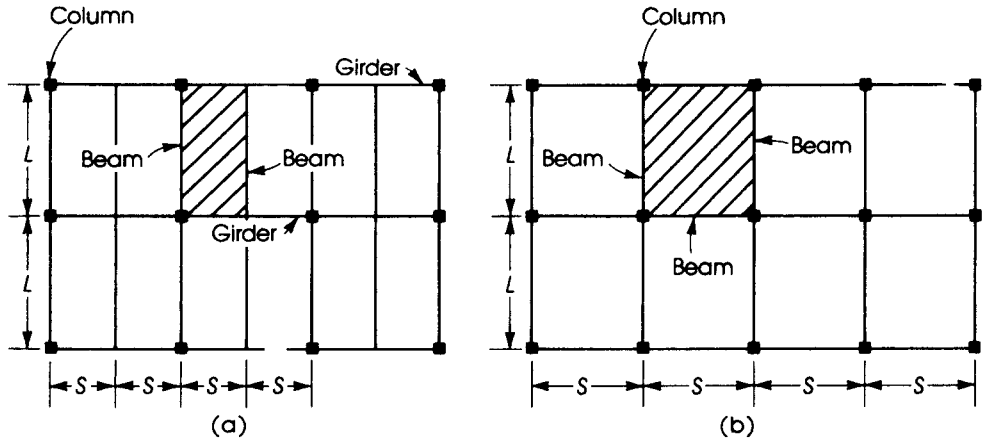


Figure 17.1 (*a*) One-way slab, $L/S > 2$, and (*b*) two-way slab, $L/S \leq 2$.

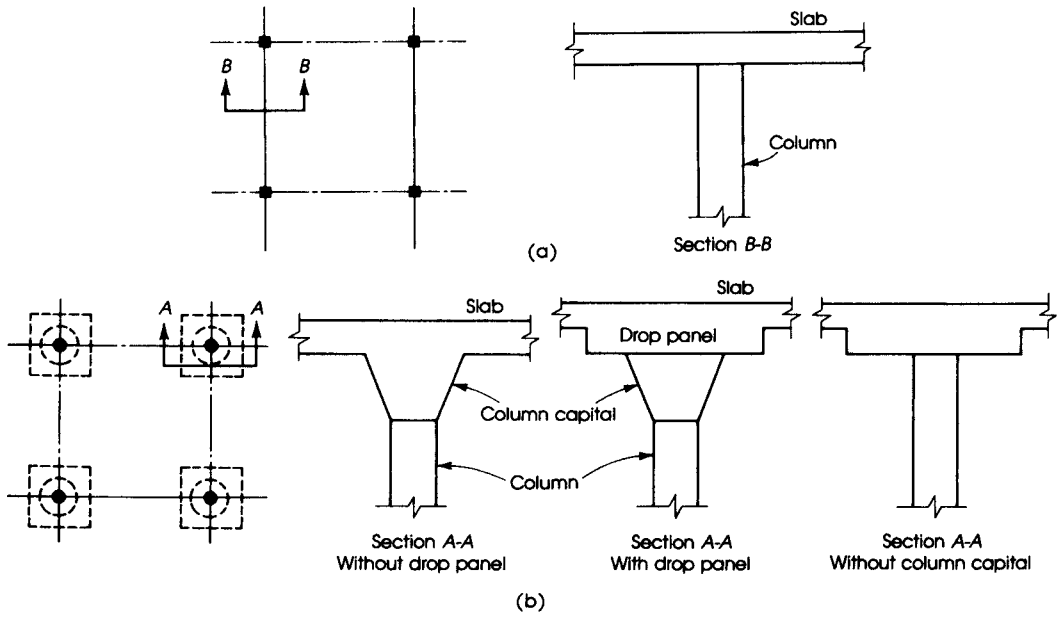


Figure 17.2 Two-way slabs without beams: (*a*) flat plate floor and section; (*b*) flat slab floor and sections; (*c*) ribbed slab and sections.

c. Using a column capital without drop panel, which is not common.

3. *Flat-Plate Floors:* A flat-plate floor is a two-way slab system consisting of a uniform slab that rests directly on columns and does not have beams or column capitals (Fig. 17.2*a*). In this case the column tends to punch through the slab, producing diagonal tensile stresses. Therefore, a general increase in the slab thickness is required or special reinforcement is used.

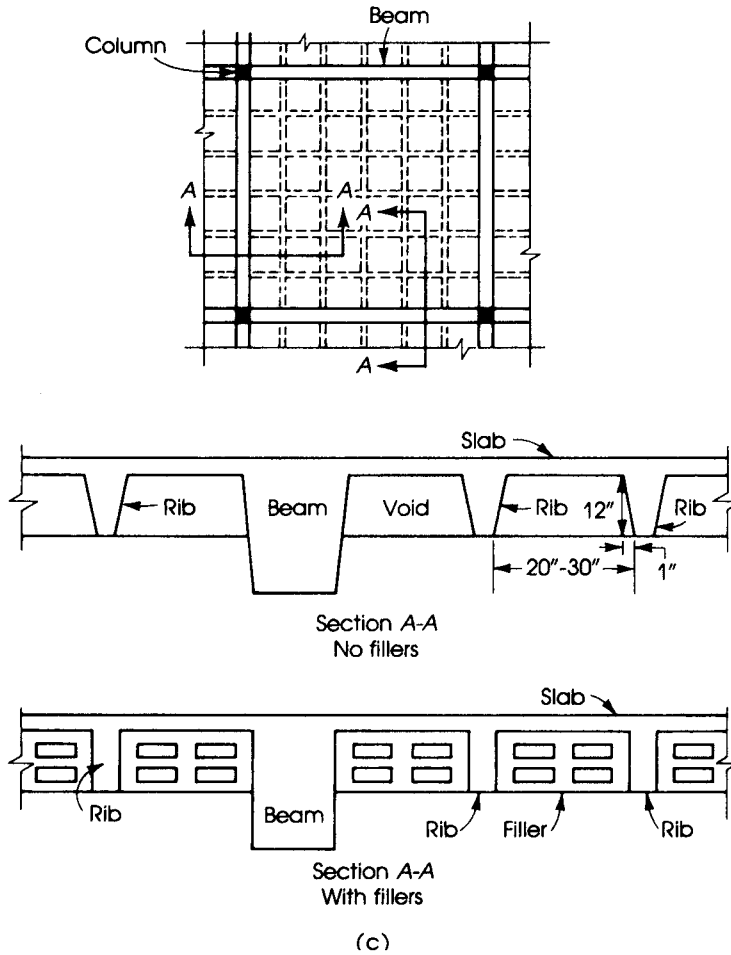
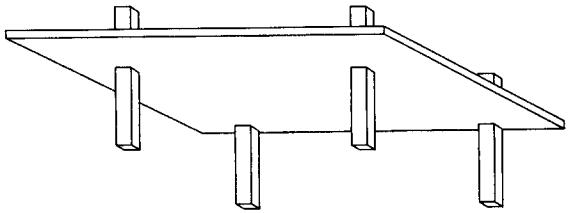
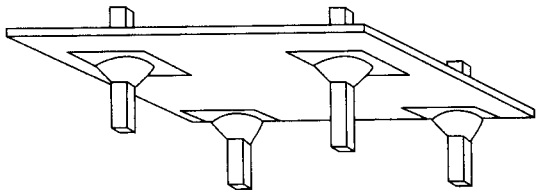


Figure 17.2 (*continued*)

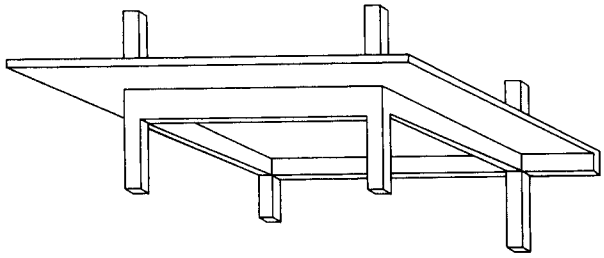
4. *Two-Way Ribbed Slabs and the Waffle Slab System:* This type of slab consists of a floor slab with a length-to-width ratio less than 2. The thickness of the slab is usually 2 to 4 in. and is supported by ribs (or joists) in two directions. The ribs are arranged in each direction at spacings of about 20 to 30 in., producing square or rectangular shapes (Fig. 17.2*c*). The ribs can also be arranged at 45° or 60° from the centerline of slabs, producing architectural shapes at the soffit of the slab. In two-way ribbed slabs, different systems can be adopted:
   a. A two-way rib system with voids between the ribs, obtained by using special removable and usable forms (pans) that are normally square in shape. The ribs are supported on four sides by girders that rest on columns. This type is called a *two-way ribbed (joist) slab system*.
   b. A two-way rib system with permanent fillers between ribs that produce horizontal slab soffits. The fillers may be of hollow, lightweight or normal-weight concrete or any other lightweight material. The ribs are supported by girders on four sides, which in turn are supported by columns. This type is also called a *two-way ribbed (joist) slab system* or a *hollow-block two-way ribbed system*.



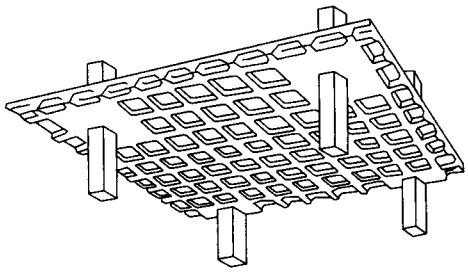
(a)



(b)



(c)



(d)

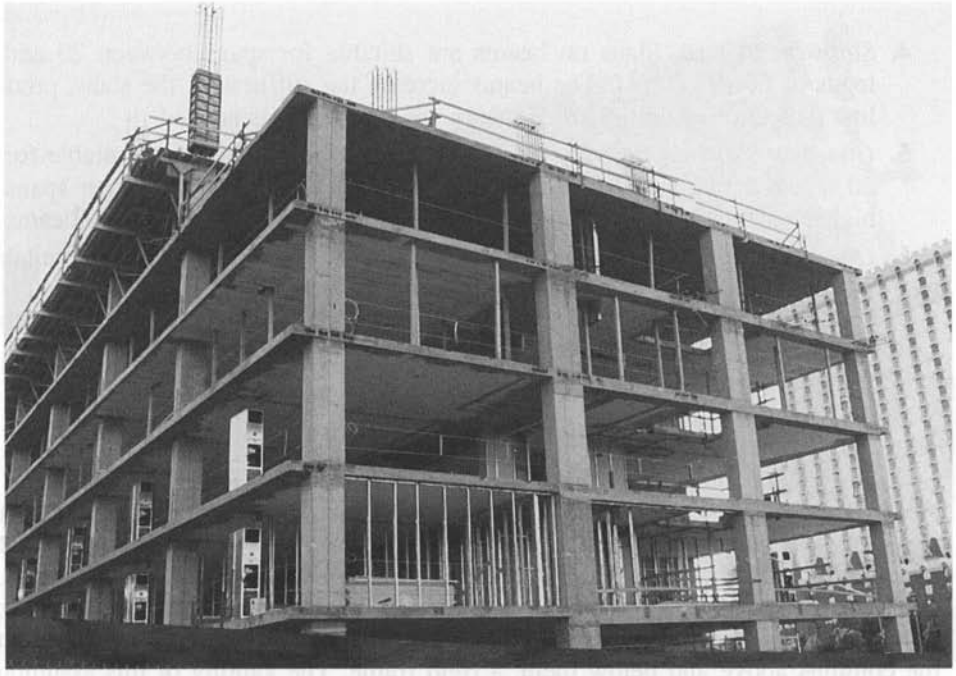
Figure 17.3 Types of two-way slab systems: (*a*) flat plate, (*b*) flat slab, (*c*) slab on beams, and (*d*) waffle slab.

c. A two-way rib system with voids between the ribs with the ribs continuing in both directions without supporting beams and resting directly on columns through solid panels above the columns. This type is called a *waffle slab system*.

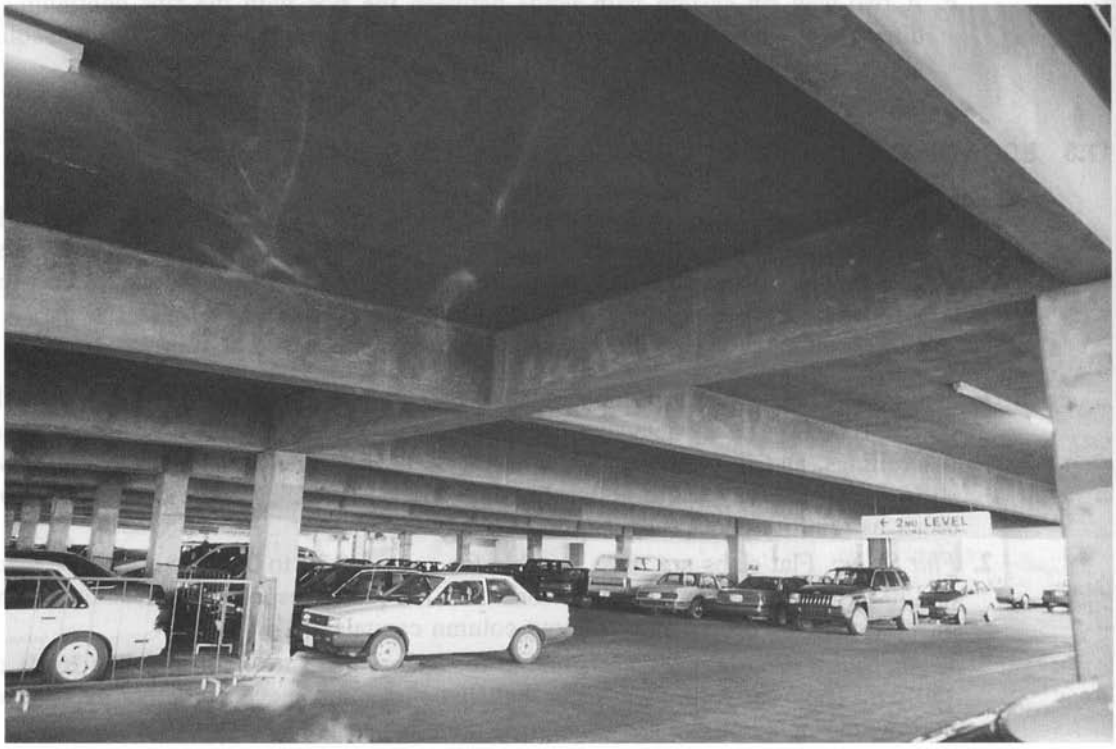
17.3 ECONOMICAL CHOICE OF CONCRETE FLOOR SYSTEMS

Various types of floor systems can be used for general buildings, such as residential, office, and institutional buildings. The choice of an adequate and economic floor system depends on the type of building, architectural layout, aesthetic features, and the span length between columns. In general, the superimposed live load on buildings varies between 80 and 150 psf. A general guide for the economical use of floor systems can be summarized as follows:

1. *Flat Plates:* Flat plates are most suitable for spans of 20 to 25 ft and live loads between 60 and 100 psf. The advantages of adopting flat plates include low-cost formwork, exposed flat ceilings, and fast construction. Flat plates have low shear capacity and relatively low stiffness, which may cause noticeable deflection. Flat plates are widely used in buildings either as reinforced or prestressed concrete slabs.
2. *Flat Slabs:* Flat slabs are most suitable for spans of 20 to 30 ft and for live loads of 80 to 150 psf. They need more formwork than flat plates, especially for column capitals. In most cases, only drop panels without column capitals are used.
3. *Waffle Slabs:* Waffle slabs are suitable for spans of 30 to 48 ft and live loads of 80 to 150 psf. They carry heavier loads than flat plates and have attractive exposed ceilings. Formwork, including the use of pans, is quite expensive.



Flat-plate floor system.



Slab on beams.

4. *Slabs on Beams:* Slabs on beams are suitable for spans between 20 and 30 ft and live loads of 60 to 120 psf. The beams increase the stiffness of the slabs, producing relatively low deflection. Additional formwork for the beams is needed.
5. *One-Way Slabs on Beams:* One-way slabs on beams are most suitable for spans of 10 to 20 ft and a live load of 60 to 100 psf. They can be used for larger spans with relatively higher cost and higher slab deflection. Additional formwork for the beams is needed.
6. *One-Way Joist Floor System:* A one-way joist floor system is most suitable for spans of 20 to 30 ft and live loads of 80 to 120 psf. Because of the deep ribs, the concrete and steel quantities are relatively low, but expensive formwork is expected. The exposed ceiling of the slabs may look attractive.

17.4 DESIGN CONCEPTS

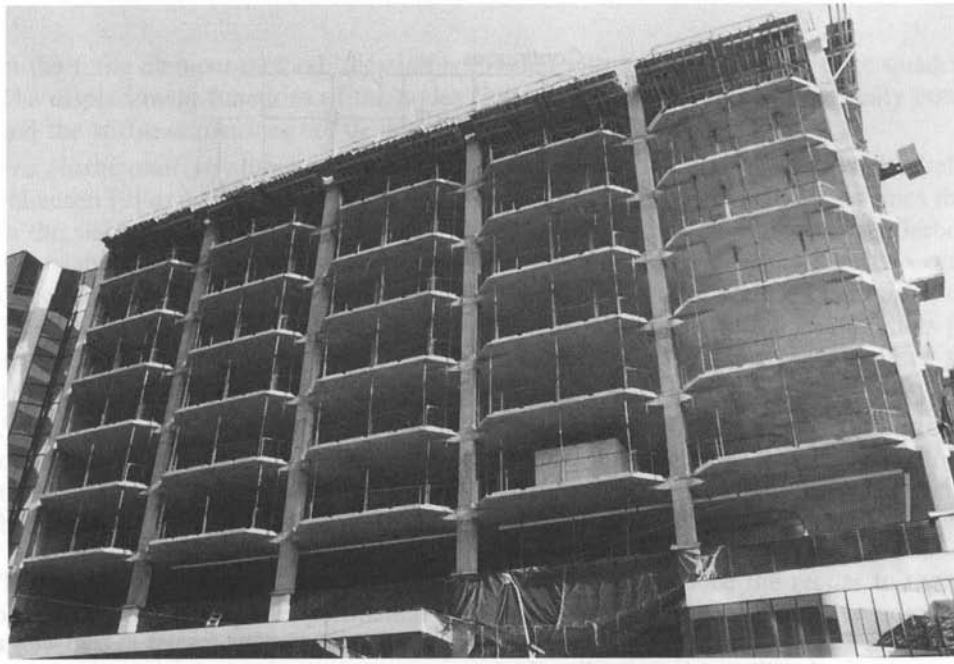
An exact analysis of forces and displacements in a two-way slab is complex, due to its highly indeterminate nature; this is true even when the effects of creep and nonlinear behavior of the concrete are neglected. Numerical methods such as finite elements can be used, but simplified methods such as those presented by the ACI Code are more suitable for practical design. The ACI Code, Chapter 13, assumes that the slabs behave as wide, shallow beams that form, with the columns above and below them, a rigid frame. The validity of this assumption of dividing the structure into equivalent frames has been verified by analytical [1,2] and experimental [3,4]

research. It is also established [3,5] that factored load capacity of two-way slabs with restrained boundaries is about twice that calculated by theoretical analysis, because a great deal of moment redistribution occurs in the slab before failure. At high loads, large deformations and deflections are expected; thus, a minimum slab thickness is required to maintain adequate deflection and cracking conditions under service loads.

The ACI Code specifies two methods for the design of two-way slabs:

1. The direct design method, DDM (ACI Code, Section 13.6), is an approximate procedure for the analysis and design of two-way slabs. It is limited to slab systems subjected to uniformly distributed loads and supported on equally or nearly equally spaced columns. The method uses a set of coefficients to determine the design moments at critical sections. Two-way slab systems that do not meet the limitations of the ACI Code, Section 13.6.1, must be analyzed by more accurate procedures.
2. The equivalent frame method, EFM (ACI Code, Section 13.7), is one in which a three-dimensional building is divided into a series of two-dimensional equivalent frames by cutting the building along lines midway between columns. The resulting frames are considered separately in the longitudinal and transverse directions of the building and treated floor by floor, as shown in Fig. 17.4.

Two ACI Code procedures are based on the results of elastic analysis of the structure as a whole using factored loads. A modified approach to the direct design method was presented in the commentary of the 1989 Code as the modified stiffness method, or MSM. It is based on specific distribution factors introduced as a function of the stiffness ratio, α_{ec}, for proportioning the total static moment in an end span. This method is explained later.



Flat slab system with drop panels (no column capitals).

In addition to the ACI Code procedures, a number of other alternatives are available for the analysis and design of slabs. The resulting slabs may have a greater or lesser amount of reinforcement. The analytical methods may be classified in terms of the basic relationship between load and deformation as elastic, plastic, and nonlinear.

1. In *elastic analysis*, a concrete slab may be treated as an elastic plate. The flexure, shear, and deflection may be calculated by the fourth differential equation relating load to deflection for thin plates with small displacements, as presented by Timoshenko [6]. Finite difference as well as finite element solutions have been proposed to analyze slabs and plates [7,8].

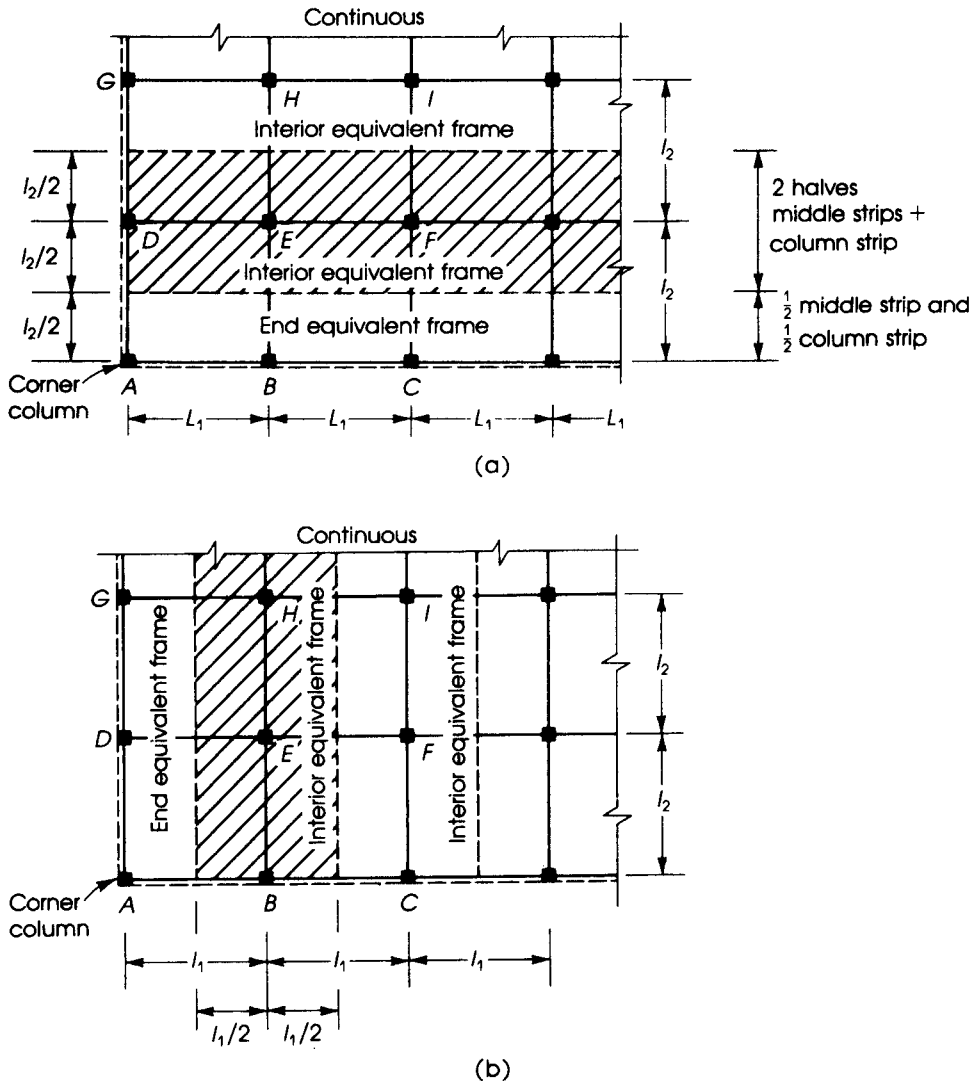


Figure 17.4 (*a*) Longitudinal and (*b*) transverse equivalent frames in plan view and (*c*) in elevation and perspective views.

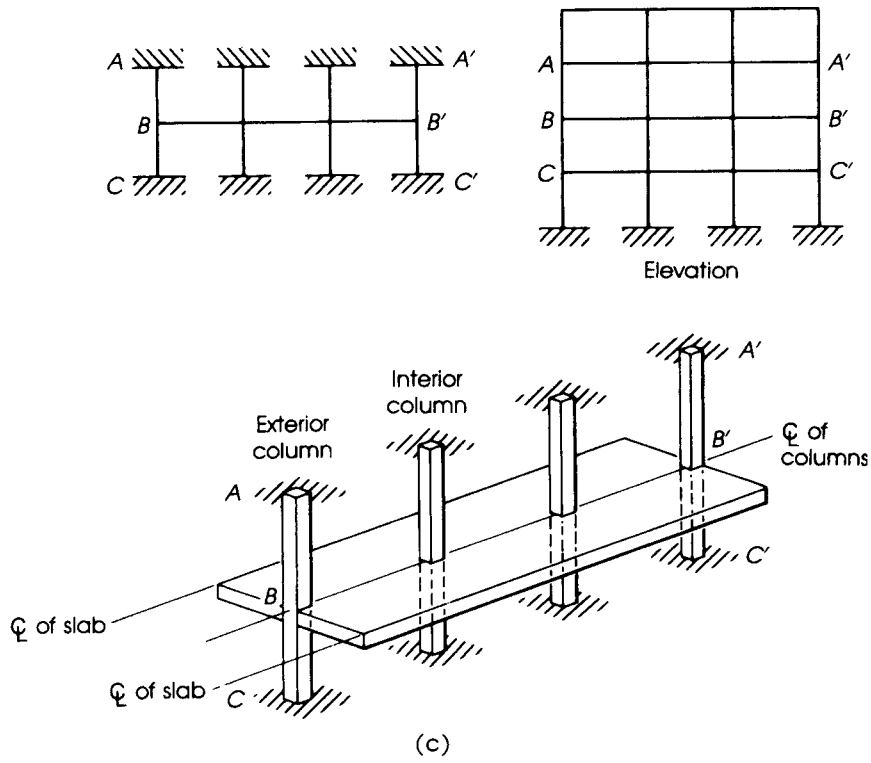


Figure 17.4 (*continued*)

In the finite element method, the slab is divided into a mesh of triangles or quadrilaterals. The displacement functions of the nodes (intersecting mesh points) are usually established, and the stiffness matrices are developed for computer analysis.

2. For *plastic analysis*, three methods are available. The *yield line* method was developed by Johansen [9] to determine the limit state of the slab by considering the yield lines that occur in the slab as a collapse mechanism. The *strip* method was developed by Hillerborg [10]. The slab is divided into strips, and the load on the slab is distributed in two orthogonal directions. The strips are analyzed as simple beams. The third method is *optimal analysis*. There has been considerable research into optimal solutions. Rozvany and others [11] presented methods for minimizing reinforcement based on plastic analysis. Optimal solutions are complex in analysis and produce complex patterns of reinforcement.
3. *Nonlinear analysis* simulates the true load deformation characteristics of a reinforced concrete slab when the finite element method takes into consideration the nonlinearity of the stress strain–relationship of the individual elements [11,12]. In this case, the solution becomes complex unless simplified empirical relationships are assumed.

The preceding methods are presented very briefly to introduce the reader to the different methods of analysis of slabs. Experimental work on slabs has not been extensive in recent years, but more research is probably needed to simplify current design procedures with adequate safety, serviceability, and economy [11].



Waffle slab with light fixtures at the centers of the squares.

17.5 COLUMN AND MIDDLE STRIPS

Figure 17.5 shows an interior panel of a two-way slab supported on columns A, B, C, and D. If the panel is loaded uniformly, the slab will deflect in both directions, with maximum deflection at the center, O. The highest points will be at the columns A, B, C, and D; thus, the part of the slab around the columns will have a convex shape. A gradual change in the shape of the slab occurs, from convexity at the columns to concavity at the center of the panel O, each radial line crossing a point of inflection. Sections at O, E, F, G, and H will have positive bending moments, whereas the periphery of the columns will have maximum negative bending moments. Considering a strip along AFB, the strip bends like a continuous beam (Fig. 17.5*b*), having negative moments at A and B and positive bending moment at F. This strip extends between the two columns A and B and continues on both sides of the panel, forming a column strip.

Similarly, a strip along EOG will have negative bending moments at E and G and a positive moment at O, forming a middle strip. A third strip along DHC will behave similarly to strip AFB. Therefore, the panel can be divided into three strips, one in the middle along EOG, referred to as the *middle strip*, and one on each side, along AFB and DHC, referred to as *column strips* (Fig. 17.5*a*). Each of the three strips behaves as a continuous beam. In a similar way, the panel is divided into three strips in the other direction, one middle strip along FOH and two column strips along AED and BGC, respectively (Fig. 17.5*e*).

Referring to Fig. 17.5*a*, it can be seen that the middle strips are supported on the column strips, which in turn transfer the loads onto the columns, A, B, C, and D in this panel. Therefore, the column strips carry more load than the middle strips. Consequently, the positive bending moment in each column strip (at E, F, G, and H) is greater than the positive bending moment at O in the middle strip. Also, the negative moments at the columns A, B, C, and D in the column strips are greater than the negative moments at E, F, G, and H in the middle strips. The portions of the design moments assigned to each critical section of the column and middle strips are discussed in Section 17.8.

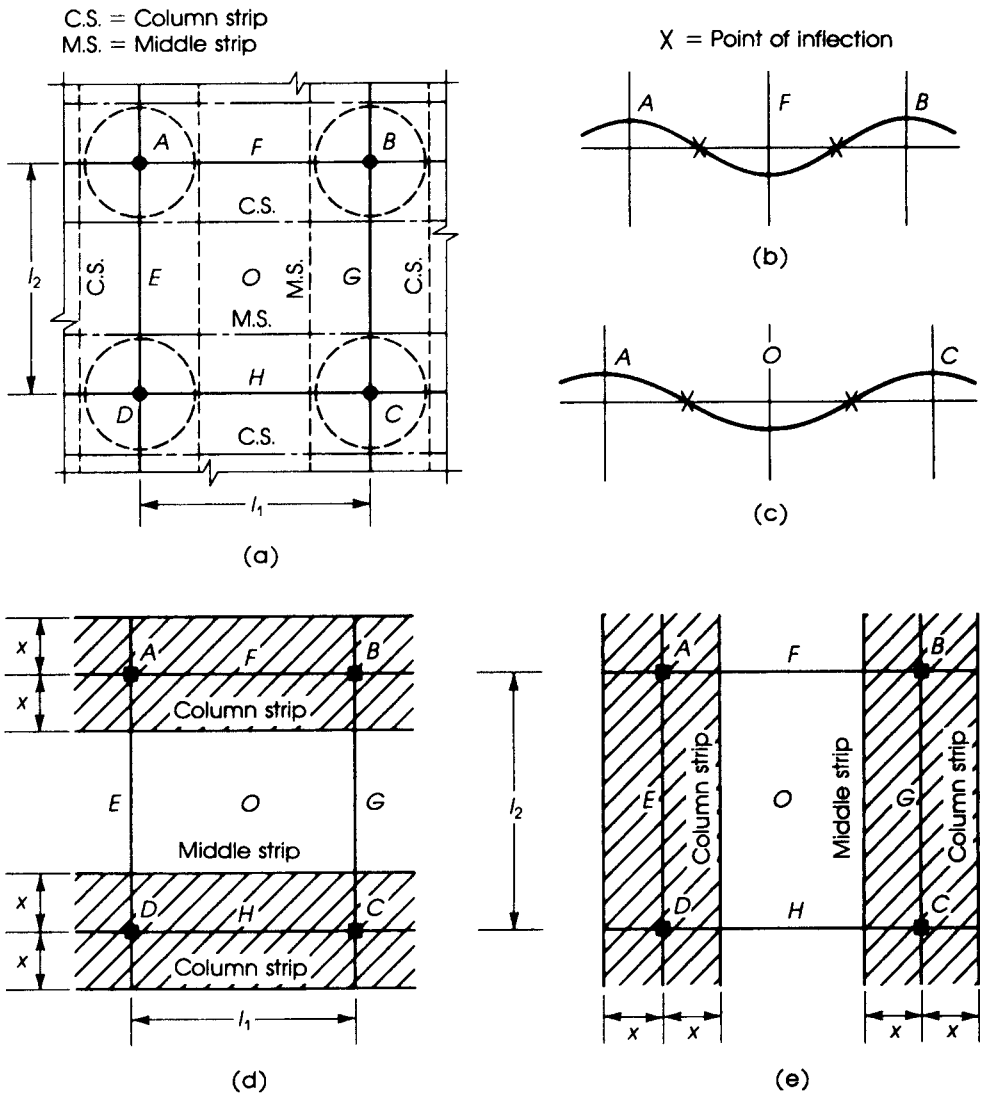


Figure 17.5 Column and middle strips; $x = 0.25/_1$ or $0.25/_2$, whichever is smaller.

The extent of each of the column and middle strips in a panel is defined by the ACI Code, Section 13.2. The column strip is defined by a slab width on each side of the column centerline, x in Fig. 17.5, equal to one-fourth the smaller of the panel dimensions l_1 and l_2, including beams if they are present, where

l_1 = span length, center to center of supports, in the direction moments are being determined

l_2 = span length, center to center of supports, in the direction perpendicular to l_1

The portion of the panel between two column strips defines the middle strip.

17.6 MINIMUM SLAB THICKNESS TO CONTROL DEFLECTION

The ACI Code, Section 9.5.3, specifies a minimum slab thickness in two-way slabs to control deflection. The magnitude of a slab's deflection depends on many variables, including the flexural stiffness of the slab, which in turn is a function of the slab thickness, h. By increasing the slab thickness, the flexural stiffness of the slab is increased, and consequently the slab deflection is reduced [13]. Because the calculation of deflections in two-way slabs is complicated and to avoid excessive deflections, the ACI Code limits the thickness of these slabs by adopting the following three empirical limitations, which are based on experimental research. If these limitations are not met, it will be necessary to compute deflections.

1. For $0.2 \leq \alpha_{fm} \leq 2$,

$$h = \frac{l_n\left(0.8 + \dfrac{f_y}{200{,}000}\right)}{36 + 5\beta(\alpha_{\text{fm}} - 0.2)} \quad (f_y \text{ in psi}) \qquad h = \frac{l_n\left(0.8 + \dfrac{f_y}{1400}\right)}{36 + 5\beta(\alpha_{\text{fm}} - 0.2)} \quad (f_y \text{ in MPa}) \tag{17.1}$$

   but not less than 5 in.

2. For $\alpha_{fm} > 2.0$,

$$h = \frac{l_n\left(0.8 + \dfrac{f_y}{200{,}000}\right)}{36 + 9\beta} \quad (f_y \text{ in psi}) \qquad h = \frac{l_n\left(0.8 + \dfrac{f_y}{1400}\right)}{36 + 9\beta} \quad (f_y \text{ in MPa}) \tag{17.2}$$

   but not less than 3.5 in.

3. For $\alpha_{fm} < 0.2$,

$$h = \text{minimum slab thickness without interior beams (Table 17.1)} \tag{17.3}$$

where

l_n = clear span in the long direction measured face to face of columns (or face to face of beams for slabs with beams)

β = the ratio of the long to the short clear spans

Table 17.1 Minimum Thickness of Slabs Without Interior Beams

	Without Drop Panels[b]			With Drop Panels[b]		
	Exterior Panels			Exterior Panels		
Yield Stress f_y psi (1)[a]	Without Edge Beams	With Edge Beams	Interior Panels	Without Edge Beams	With Edge Beams[c]	Interior Panels
40,000	$\frac{l_n}{33}$	$\frac{l_n}{36}$	$\frac{l_n}{36}$	$\frac{l_n}{36}$	$\frac{l_n}{40}$	$\frac{l_n}{40}$
60,000	$\frac{l_n}{30}$	$\frac{l_n}{33}$	$\frac{l_n}{33}$	$\frac{l_n}{30}$	$\frac{l_n}{36}$	$\frac{l_n}{36}$

[a] For values of reinforcement, yield stress between 40,000 and 60,000 psi minimum thickness shall be obtained by linear interpolation.

[b] Drop panel is defined in ACI Sections 13.3.7.1 and 13.3.7.2.

[c] Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall be not less than 0.8.

α_{fm} = the average value of α for all beams on the sides of a panel

α_f = the ratio of flexural stiffness of a beam section $E_{cb}I_b$ to the flexural stiffness of the slab $E_{cs}I_s$, bounded laterally by the centerlines of the panels on each side of the beam

$$\alpha_f = \frac{E_{cb}I_b}{E_{cs}I_s} \tag{17.4}$$

where E_{cb} and E_{cs} are the moduli of elasticity of concrete in the beam and the slab, respectively, and

I_b = the gross moment of inertia of the beam section about the centroidal axis (the beam section includes a slab length on each side of the beam equal to the projection of the beam above or below the slab, whichever is greater, but not more than four times the slab thickness)

I_s = the moment of inertia of the gross section of the slab

However, the thickness of any slab shall not be less than the following:

1. For slabs with $\alpha_{fm} \leq 2.0$ then thickness $\geq$ 5.0 in. (125 mm)
2. For slabs with $\alpha_{fm} > 2.0$ then thickness $\geq$ 3.5 in. (90 mm)

If no beams are used, as in the case of flat plates, then $\alpha_f = 0$ and $\alpha_{fm} = 0$. The ACI Code equations for calculating slab thickness, h, take into account the effect of the span length, the panel shape, the steel reinforcement yield stress, f_y, and the flexural stiffness of beams. When very stiff beams are used, Eq. 17.1 may give a small slab thickness, and Eq. 17.2 may control. For flat plates and flat slabs, when no interior beams are used, the minimum slab thickness may be determined directly from Table 9.5*c* of the ACI Code, which is shown here as Table 17.1.

Other ACI Code limitations are summarized as follows:

1. For panels with discontinuous edges, end beams with a minimum α equal to 0.8 must be used; otherwise, the minimum slab thickness calculated by Eqs. 17.1 and 17.2 must be increased by at least 10% (ACI Code, Section 9.5.3).
2. When drop panels are used without beams, the minimum slab thickness may be reduced by 10%. The drop panels should extend in each direction from the centerline of support a distance not less than one-sixth of the span length in that direction between center to center of supports and also project below the slab at least $h/4$. This reduction is included in Table 17.1.
3. Regardless of the values obtained by Eqs. 17.1 and 17.2, the thickness of two-way slabs shall not be less than the following: (1) for slabs without beams or drop panels, 5 in. (125 mm); (2) for slabs without beams but with drop panels, 4 in. (100 mm); (3) for slabs with beams on all four sides with $\alpha_{fm} \geq 2.0$, $3\frac{1}{2}$in. (90 mm), and for $\alpha_{fm} < 2.0$, 5 in. (125 mm) (ACI Code, Section 9.5.3.).

The following steps summarize these calculations:

1. For slabs without interior beams (flat plates and flat slabs),
   a. Calculate the minimum slab thickness directly from Table 17.1. However, Eqs. 17.1 and 17.2 may be used, and Eq. 17.1 normally controls. Minimum slab thickness shall be greater than or equal to 5 in. (125 mm) for slabs without drop panels and greater than or equal to 4 in. (100 mm) for slabs with drop panels.

b. At discontinuous edges, an edge beam with $\alpha_f \geq 0.8$ should be used. Otherwise, the minimum slab thickness calculated by Eqs. 17.1 and 17.2 should be increased by 10%. This increase of 10% has already been included in the second columns of Table 17.1.

c. If drop panels are used in flat slabs, the minimum slab thickness may be reduced by 10% on the condition that the drop panel extends in each direction from the centerline of the support a distance not less than one-sixth of the span and projects below the slab at least $h/4$. This reduction is included in the factors of Table 17.1.

2. For slabs with beams on all sides ($\alpha_{fm} > 0$),

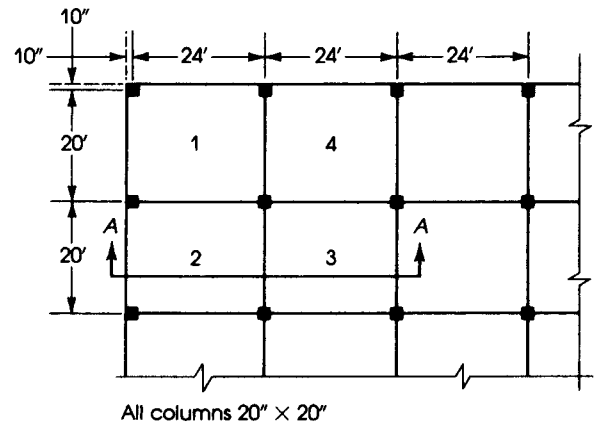
a. Calculate α_{fm} and then calculate the minimum slab thickness from Eqs. 17.1 and 17.2. In most cases, Eq. 17.2 controls.

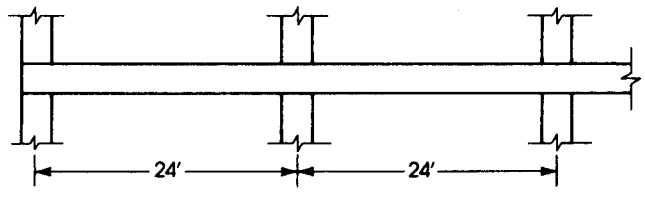
b. The slab thickness should be greater than or equal to 5 in. for slabs with $\alpha_{fm} < 2.0$ and should be greater than or equal to 3.5 in. for slabs with $\alpha_{fm} \geq 2.0$.

3. For all slabs: A slab thickness less than the minimum thickness given in steps 1 and 2 may be used if shown by computation that deflection will not exceed the ACI Code, Table 9.5*b* limitations explained earlier in Chapter 6.

Example 17.1

A flat-plate floor system with panels 24 by 20 ft is supported on 20-in. square columns. Using the ACI Code equations, determine the minimum slab thickness required for the interior and corner panels shown in Fig. 17.6. Edge beams are not used. Use $f'_c = 4$ ksi and $f'_c = 60$ ksi.





Section *A-A*

Figure 17.6 Example 17.1.

Solution

1. For corner panel no. 1, the minimum thickness is $l_n/30$ ($f_y = 60$ ksi, and no edge beams are used; see Table 17.1).

$$l_{n_1} = 24 - \frac{20}{12} = 22.33 \text{ ft} \quad \text{(long direction)}$$

$$h_{\min} = \frac{22.33 \times 12}{30} = 8.93 \text{ in.}, \quad \text{say, } 9.0 \text{ in.}$$

   Alternatively, Eqs. 17.1 and 17.2 can be used to calculate the minimum thickness with $\alpha_f = \alpha_{fm} = 0$.

2. For the interior panel no. 3 and $f_y = 60$ ksi, the minimum slab thickness is $l_n/33 = (22.33 \times 12)/33 = 8.12$ in., say, 8.5 in. Alternatively, Eqs. 17.1 and 17.2 can be used. If a uniform slab thickness is used for all panels, then $h = 9.0$ in. will be adopted.

Example 17.2

The floor system shown in Fig. 17.7 consists of solid slabs and beams in two directions supported on 20-in. square columns. Using the ACI Code equations, determine the minimum slab thickness required for an interior panel. Use $f'_c = 3$ ksi and $f_y = 60$ ksi.

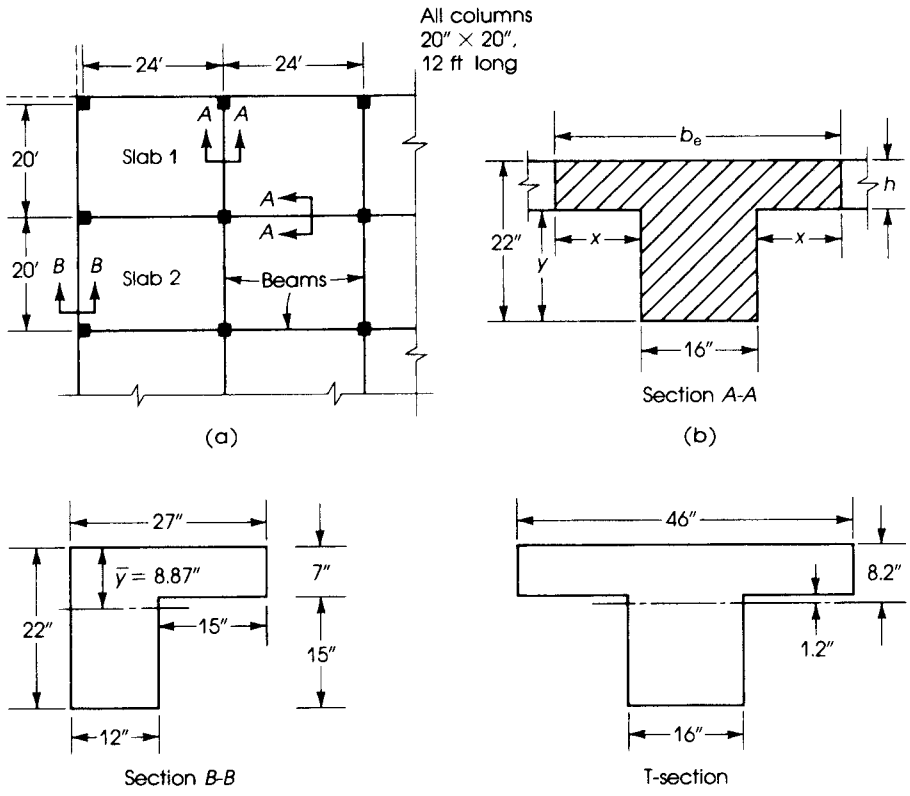


Figure 17.7 Example 17.2.

Solution

1. To use Eq. 17.1, α_m should be calculated first. Therefore, it is required to determine I_b, I_s, and α_f for the beams and slabs in the long and short directions.
2. The gross moment of inertia of the beam, I_b, is calculated for the section shown in Fig. 17.7*b*, which is made up of the beam and the extension of the slab on each side of the beam $x = y$ but not more than four times the slab thickness. Assume $h = 7$ in., to be checked later; then $x\ x = y = 22 - 7 = 15$ in. $< 4 \times 7 = 28$ in. Therefore, $b_e = 16 + 2 \times 15 = 46$ in., and the T-section is shown in Fig. 17.7*c*. Determine the centroid of the section by taking moments about the top of the flange:

$$\text{Area of flange} = 7 \times 46 = 322 \text{ in.}^2$$

$$\text{Area of web} = 16 \times 15 = 240 \text{ in.}^2$$

$$\text{Total area} = 562 \text{ in.}^2$$

$$(322 \times 3.5) + 240 \times (7 + 7.5) = 562y$$

$$y = 8.20 \text{ in.}$$

$$I_b = \left[\frac{46}{12}(7)^3 + 322 \times (4.7)^2\right] + \left[\frac{16(15)^3}{12} + 240(7.5 - 1.2)^2\right] = 22{,}453 \text{ in.}^4$$

3. The moment of inertia of the slab in the long direction is $I_s = (bh^3)/12$, where $b = 20$ ft and $h = 7$ in.

$$I_l = \frac{(20 \times 12)(7)^3}{12} = 6860 \text{ in.}^4$$

$$\alpha_{f1}(\text{in the long direction}) = \frac{EI_b}{EI_s} = \frac{22{,}453}{6860} = 3.27$$

4. The moment of inertia of the slab in the short direction is $I_s = (bh^3)/12$ where $b = 24$ ft and $h = 7$ in.

$$I_s = \frac{(24 \times 12)(7)^3}{12} = 8232 \text{ in.}^4$$

$$\alpha_s = \frac{EI_b}{EI_s} = \frac{22{,}453}{8232} = 2.27$$

5. α_{fm} is the average of α_{f1} and α_s:

$$\alpha_{\text{fm}} = \frac{3.27 + 2.72}{2} = 3.0$$

6.

$$\beta = \frac{(24 - \frac{20}{12})}{(20 - \frac{20}{12})} = \frac{22.33}{18.33} = 1.22$$

7. Determine $h_{\min}$ using Eq. 17.1 ($l_n = 22.33$ ft):

$$h_{\min} = \frac{(22.33 \times 12)(0.8 + 0.005 \times 60)}{36 + (5 \times 1.22)[3.0 - 0.2]} = 5.57 \text{ in.}$$

However, this value must not be less than h given by Eq. 17.2 ($\alpha_{fm} > 2.0$):

$$h = \frac{294.8}{36 + 9(1.22)} = 6.27 \text{ in.}$$

Also, $h_{\min} = 3.5$ in. Therefore, $h = 6.27$ in. controls. A slab thickness of 6.5 in. or 7.0 in. may be adopted. Note that in most practical cases, Eq. 17.2 controls.

17.7 SHEAR STRENGTH OF SLABS

In a two-way floor system, the slab must have adequate thickness to resist both bending moments and shear forces at the critical sections. To investigate the shear capacity of two-way slabs, the following cases should be considered.

17.7.1 Two-Way Slabs Supported on Beams

In two-way slabs supported on beams, the critical sections are at a distance d from the face of the supporting beams, and the shear capacity of each section is $\phi V_c = \phi(2\lambda\sqrt{f'_c}bd)$. When the supporting beams are stiff and are capable of transmitting floor loads to the columns, they are assumed to carry loads acting on floor areas bounded by 45° lines drawn from the corners, as shown in Fig. 17.8. The loads on the trapezoidal areas will be carried by the long beams AB and CD, whereas the loads on the triangular areas will be carried by the short beams AC and BD. The shear per unit width of slab is highest between E and F in both directions, and $V_u = w_u(l_2/2)$, where w_u is the uniform factored load per unit area.

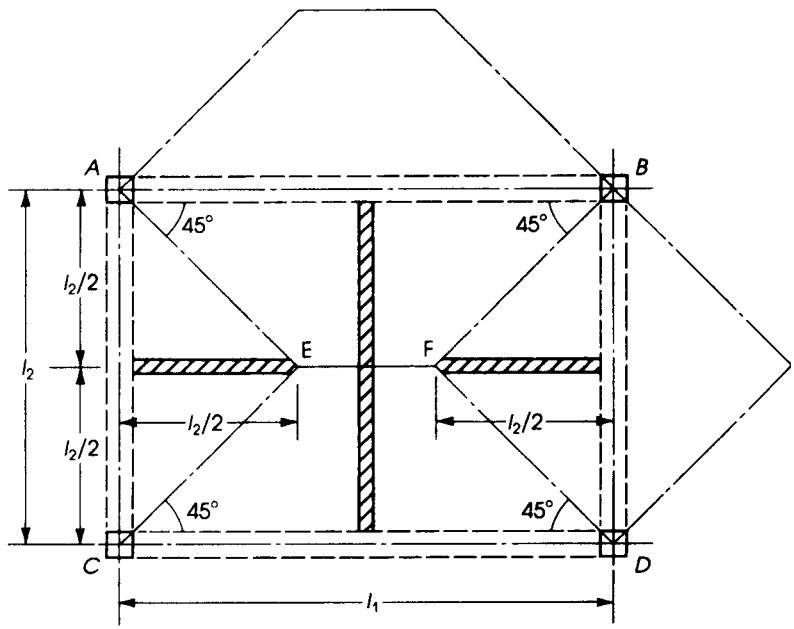


Figure 17.8 Areas supported by beams in two-way slab floor system.

If no shear reinforcement is provided, the shearing force at a distance d from the face of the beam, V_{ud}, must be equal to

$$V_{ud} \leq \phi V_c \leq \phi(2\lambda\sqrt{f'_c}bd),$$

where

$$V_{ud} = w_u\left(\frac{l_2}{2} - d\right).$$

17.7.2 Two-Way Slabs Without Beams

In flat plates and flat slabs, beams are not provided, and the slabs are directly supported by columns. In such slabs, two types of shear stresses must be investigated; the first is one-way shear, or beam shear. The critical sections are taken at a distance d from the face of the column, and the slab is considered as a wide beam spanning between supports, as in the case of one-way beams. The shear capacity of the concrete section is $\phi V_c = \phi(2\lambda\sqrt{f'_c}bd)$. The second type of shear to be studied is two-way, or punching, shear, as was previously discussed in the design of footings. Shear failure occurs along a truncated cone or pyramid around the column. The critical section is located at a distance $d/2$ from the face of the column, column capital, or drop panel (Fig. 17.9*a*). If shear reinforcement is not provided, the shear strength of concrete is the smaller of Eq. 17.5 and 17.6:

$$\phi V_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c}b_o d \leq 4\phi\sqrt{f'_c}b_o d \tag{17.5}$$

where

b_o = perimeter of the critical section
β = ratio of the long side of column (or loaded area) to the short side

$$\phi V_c = \phi\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}b_o d \tag{17.6}$$

where α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns. When shear reinforcement is provided, the shear strength should not exceed

$$\phi V_n \leq \phi(6\sqrt{f'_c}b_o d) \tag{17.7}$$

17.7.3 Shear Reinforcement in Two-Way Slabs Without Beams

In flat-slab and flat-plate floor systems, the thickness of the slab selected may not be adequate to resist the applied shear stresses. In this case, either the slab thickness must be increased or shear reinforcement must be provided. The ACI Code allows the use of shear reinforcement by shearheads and anchored bars or wires.

Shearheads consist of steel I-shapes or channel shapes welded into four cross-arms and placed in the slabs above the column (Fig. 17.9*c*, *d*). Shearhead designs do not apply to exterior columns, where large torsional and bending moments must be transferred between slab and column. The ACI Code, Section 11.11.4.8, indicates that on the critical section the nominal shear strength, V_n, should not exceed $4\sqrt{f'_c}b_o d$, but if shearhead reinforcement is provided, V_n

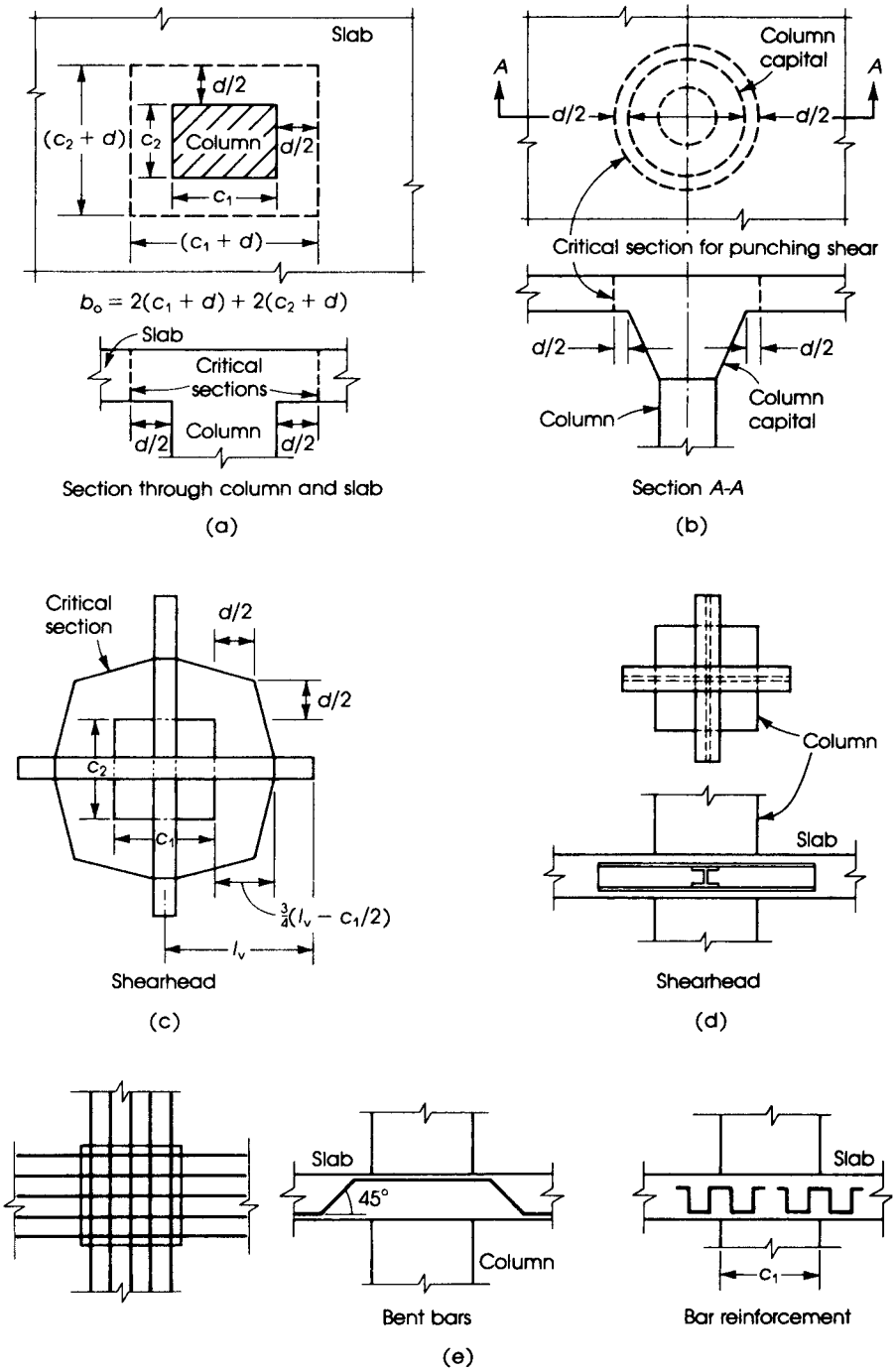
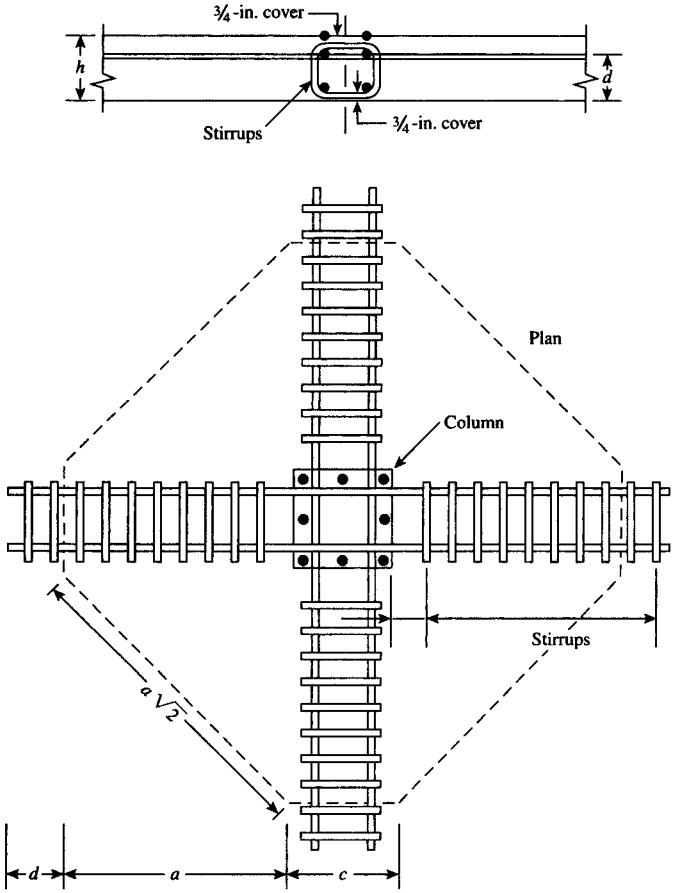


Figure 17.9 Critical section for punching shear in (*a*) flat plates and (*b*) flat slabs, reinforcement by (*c*, *d*) shearheads and (*e*) anchored bars, (*f*) conventional stirrup cages, and (*g*) studded steel strips used as shear reinforcement.



(f)

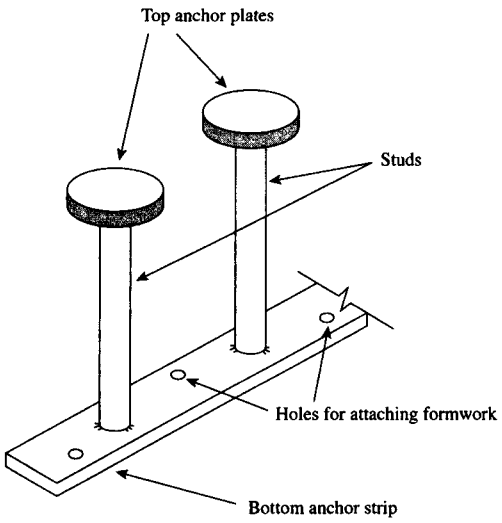


Figure 17.9 (*continued*)

should not exceed $7\sqrt{f'_c}b_o d$. To determine the size of the shearhead, the ACI Code, Section 11.11.4.8, gives the following limitations:

1. The ratio α_v between the stiffness of shearhead arm, $E_s I$, and that of the surrounding composite cracked section of width, $c_2 + d$, must not be less than 0.15.
2. The compression flange of the steel shape must be located within $0.13d$ of the compression surface of the slab.
3. The depth of the steel shape must not exceed 70 times the web thickness.
4. The plastic moment capacity, M_p, of each arm of the shearhead is computed by

$$\phi M_p = \frac{V_u}{2n}\left[h_v + \alpha_v\left(l_v - \frac{c_1}{2}\right)\right] \quad \text{(ACI Code, Eq. 11.37)} \tag{17.8}$$

where

$\phi = 0.9$
V_u = factored shear force around the periphery of the column face
n = number of arms
h_v = depth of the shearhead
l_v = length of the shearhead measured from the centerline of the column.

5. The critical slab section for shear must cross each shearhead arm at a distance equal to (3/4) $(l_v - c_1/2)$ from the column face to the end of the shearhead arm, as shown in Fig. 17.9*c*. The critical section must have a minimum perimeter, b_o, but it should not be closer than $d/2$ from the face of the column.
6. The shearhead is considered to contribute a moment resistance, M_v, to each slab column strip as follows:

$$M_v = \frac{\phi}{2n}\alpha_v V_u\left(l_v - \frac{c_1}{2}\right) \quad \text{(ACI Code, Eq. 11.38)} \tag{17.9}$$

but it should not be more than the smallest of 30% of the factored moment required in the column strip or the change in the column strip moment over the length l_v or M_p given in Eq. 17.8.

The use of anchored bent bars or wires is permitted by the ACI Code, Section 11.11.3. The bars are placed on top of the column, and the possible arrangements are shown in Fig. 17.9*e*. When bars or wires are used as shear reinforcement, the nominal shear strength is

$$V_n = V_c + V_s = (2\lambda\sqrt{f'_c})b_o d + \frac{A_v f_y d}{s} \tag{17.10}$$

where A_v is the total stirrup bar area and b_o is the length of the critical section of two-way shear at a distance $d/2$ from the face of the column. The nominal shear strength, V_n, should not exceed $6\sqrt{f'_c}b_o d$.

The use of shear reinforcement in flat plates reduces the slab thickness and still maintains the flat ceiling to reduce the cost of formwork. Typical stirrup cages for shear reinforcement are shown in Fig. 17.9*f*. Another type of shear reinforcement consists of studded steel strips (Fig. 17.9*g*). The steel strip is positioned with bar chairs and fastened to the formwork, replacing the stirrup gages. The yield strength of the stud material is specified between 40 and 60 ksi to achieve complete anchorage at ultimate load.

17.8 ANALYSIS OF TWO-WAY SLABS BY THE DIRECT DESIGN METHOD

The direct design method is an approximate method established by the ACI Code to determine the design moments in uniformly loaded two-way slabs. To use this method, some limitations must be met, as indicated by the ACI Code, Section 13.6.1.

17.8.1 Limitations

1. There must be a minimum of three continuous spans in each direction.
2. The panels must be square or rectangular; the ratio of the longer to the shorter span within a panel must not exceed 2.0.
3. Adjacent spans in each direction must not differ by more than one-third of the longer span.
4. Columns must not be offset by a maximum of 10% of the span length, in the direction of offset, from either axis between centerlines of successive columns.
5. All loads must be uniform, and the ratio of the unfactored live to unfactored dead load must not exceed 2.0.
6. If beams are present along all sides, the ratio of the relative stiffness of beams in two perpendicular directions, $\alpha_{f_1} l_l^2 / \alpha_{f_2} l_1^2$ must not be less than 0.2 nor greater than 5.0.

17.8.2 Total Factored Static Moment

If a simply supported beam carries a uniformly distributed load w K/ft, then the maximum positive bending moment occurs at midspan and equals $M_o = wl_1^2/8$, where l_1 is the span length. If the beam is fixed at both ends or continuous with equal negative moments at both ends, then the total moment $M_o = M_p$(positive moment at midspan) + M_n(negative moment at support) = $wl_1^2/8$ (Fig. 17.10). Now if the beam AB carries the load W from a slab that has a width l_2 perpendicular to l_1, then $W = w_u l_2$, and the total moment is $M_o = \frac{(w_u l_2) l_1^2}{8}$, where w_u = load intensity in K/ft^2. In this expression, the actual moment occurs when l_1 equals the clear span between supports A and B. If the clear span is denoted by l_n, then

$$M_o = (w_u l_2)\frac{l_n^2}{8} \quad \text{(ACI Code, Eq. 13.4)} \tag{17.11}$$

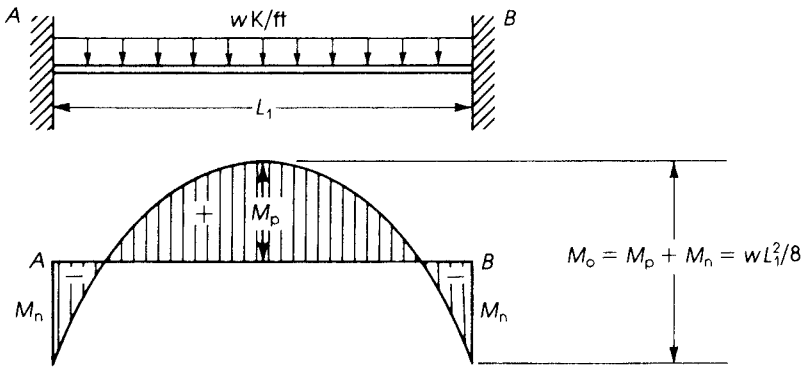


Figure 17.10 Bending moment in a fixed-end beam.

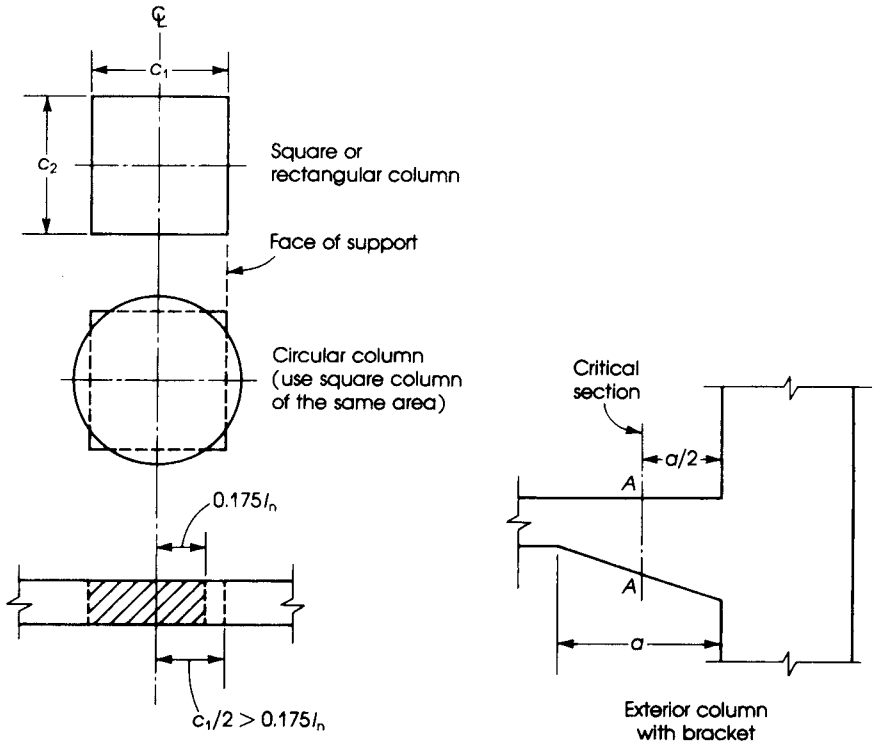


Figure 17.11 Critical sections for negative design moments. *A-A*, section for negative moment at exterior support with bracket.

The clear span, l_n, is measured face to face of supports in the direction in which moments are considered, but not less than 0.65 times the span length from center to center of supports. The face of the support where the negative moments should be calculated is illustrated in Fig. 17.11. The length l_2 is measured in a direction perpendicular to l_n and equals the direction between center to center of supports (width of slab). The total moment M_o calculated in the long direction will be referred to here as M_{ol} and that in the short direction, as M_{os}.

Once the total moment, M_o, is calculated in one direction, it is divided into a positive moment, M_p, and a negative moment, M_n, such that $M_o = M_p + M_n$ (Fig. 17.10). Then each moment, M_p and M_n, is distributed across the width of the slab between the column and middle strips, as is explained shortly.

17.8.3 Longitudinal Distribution of Moments in Slabs

In a typical *interior panel*, the total static moment, M_o, is divided into two moments, the positive moment, M_p, at midspan, equal to $0.35M_o$, and the negative moment, M_n, at each support, equal to $0.65M_o$, as shown in Fig. 17.12. These values of moment are based on the assumption that the interior panel is continuous in both directions, with approximately equal spans and loads, so that the interior joints have no significant rotation. Moreover, the moment values are approximately the same as those in a fixed-end beam subjected to uniform loading, where the negative moment at the support is twice the positive moment at midspan. In Fig. 17.12, if $l_1 > l_2$, then the distribution of moments in the long and short directions is as follows:

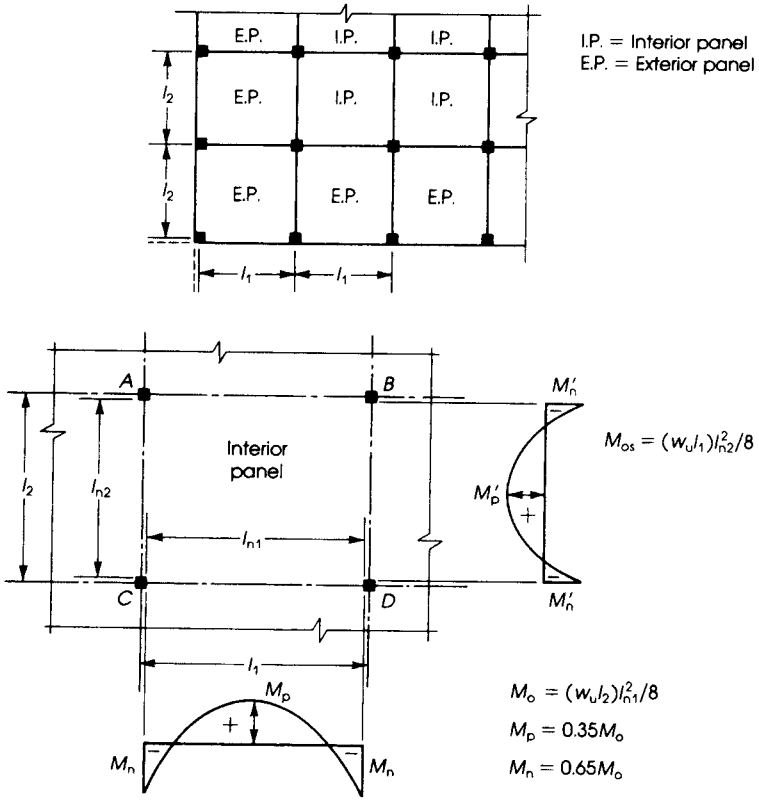


Figure 17.12 Distribution of moments in an interior panel.

$$M_{ol} = (w_u l_2)\frac{l_{n1}^2}{8} \quad M_{pl} = 0.35M_{ol} \quad M_{nl} = 0.65M_{ol}$$

$$M_{os} = (w_u l_1)\frac{l_{n2}^2}{8} \quad M_{ps} = 0.35M_{os} \quad M_{ns} = 0.65M_{os}$$

If the magnitudes of the negative moments on opposite sides of an interior support are different because of unequal span lengths, the ACI Code specifies that the larger moment should be considered to calculate the required reinforcement.

In an *exterior panel*, the slab load is applied to the exterior column from one side only, causing an unbalanced moment and a rotation at the exterior joint. Consequently, there will be an increase in the positive moment at midspan and in the negative moment at the first interior support. The magnitude of the rotation of the exterior joint determines the increase in the moments at midspan and at the interior support. For example, if the exterior edge is a simple support, as in the case of a slab resting on a wall (Fig. 17.13), the slab moment at the face of the wall there is 0, the positive moment at midspan can be taken as $M_p = 0.63M_o$, and the negative moment at the interior support is $M_n = 0.75\ M_o$. These values satisfy the static equilibrium equation

$$M_o = M_p + \frac{1}{2}M_n = 0.63M_o + \frac{1}{2}(0.75M_o)$$

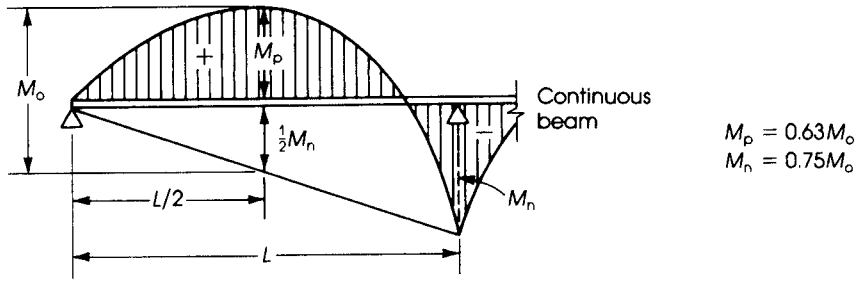


Figure 17.13 Exterior panel.

In a slab-column floor system, there is some restraint at the exterior joint provided by the flexural stiffness of the slab and by the flexural stiffness of the exterior columns.

According to Section 13.6.3 of the ACI Code, the total static moment M_o in an end span is distributed in different ratios according to Table 17.2 and Fig. 17.14. The moment coefficients in column 1 for an unrestrained edge are based on the assumption that the ratio of the flexural stiffness of columns to the combined flexural stiffness of slabs and beams at a joint, α_{ec} is equal to 0. The coefficients of column 2 are based on the assumption that the ratio α_{ec} is equal to infinity. The moment coefficients in columns 3, 4, and 5 have been established by analyzing the slab systems with different geometries and support conditions.

17.8.4 Transverse Distribution of Moments

The longitudinal moment values mentioned in the previous section are for the entire width of the equivalent building frame. This frame width is the sum of the widths of two half-column strips and two half-middle strips of two adjacent panels, as shown in Fig. 17.15. The transverse distribution of the longitudinal moments to the middle and column strips is a function of the ratios l_2/l_1,

$$\alpha_f = \frac{E_{cb}I_b}{E_{cs}I_s} = \frac{\text{beam stiffness}}{\text{slab stiffness}} \tag{17.12}$$

$$\beta_t = \frac{E_{cb}C}{2E_{cs}I_s} = \frac{\text{torsional rigidity of edge beam section}}{\text{flexural rigidity of a slab of width equal to beam span length}} \tag{17.13}$$

Table 17.2 Distribution of Moments in an End Panel

	Exterior Edge		Slab with Beams Between All Supports	Slab Without Beams Between Interior Supports	
	Unrestrained (1)	Fully Restrained (2)	(3)	With Edge Beam (4)	Without Edge Beam (5)
Exterior negative factored moment	0	0.65	0.16	0.30	0.26
Positive factored moment	0.63	0.35	0.57	0.50	0.52
Interior negative factored moment	0.75	0.65	0.70	0.70	0.70

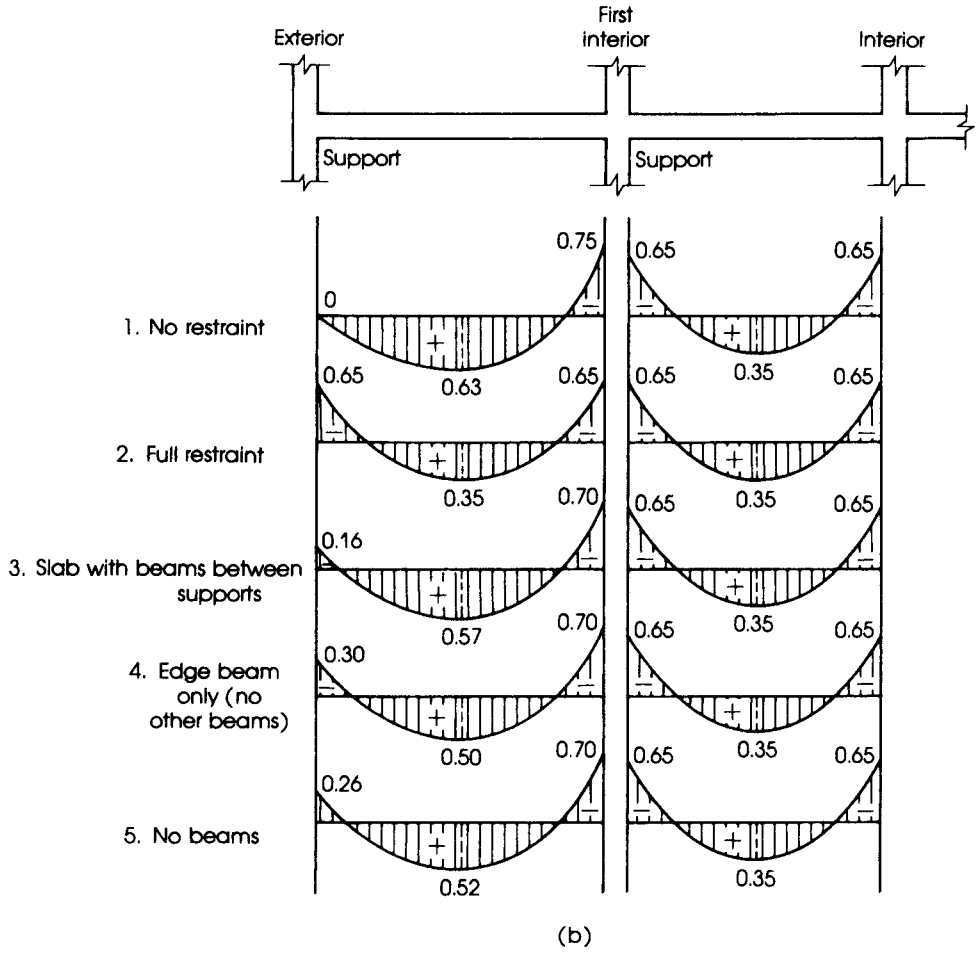


Figure 17.14 Distribution of total static moment into negative and positive span moments.

where

$$C = \text{torsional constant} = \sum \left(1 - \frac{0.63x}{y}\right)\left(\frac{x^3 y}{3}\right) \tag{17.14}$$

where x and y are the shorter and longer dimension of each rectangular component of the section. The percentages of each design moment to be distributed to column and middle strips for interior and exterior panels are given in Tables 17.3 through Table 17.6. In a typical *interior* panel, the portion of the design moment that is not assigned to the column strip (Table 17.3) must be resisted by the corresponding half-middle strips. Linear interpolation of values of l_2/l_1 between 0.5 and 2.0 and of $\alpha_{f_1} l_2/l_1$ between 0 and 1 is permitted by the ACI Code. From Table 17.3 it can be seen that when no beams are used, as in the case of flat plates or flat slabs, $\alpha_{f_1} = 0$. The final percentage of moments in the column and middle strips as a function of M_o are given in Table 17.4.

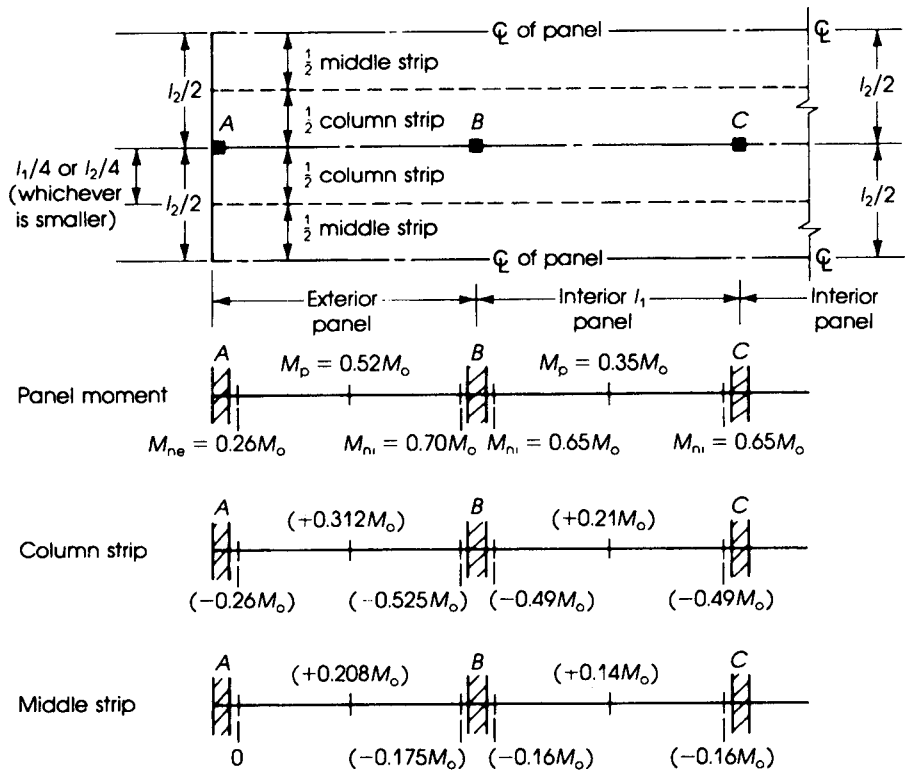


Figure 17.15 Width of the equivalent rigid frame (equal spans in this figure) and distribution of moments in flat plates, flat slabs, and waffle slabs with no beams.

Table 17.3 Percentage of Longitudinal Moment in Column Strips, Interior Panels (ACI Code, Section 13.6.4)

		Aspect Ratio, l_2/l_1		
	$\alpha_{f_1} l_2/l_1$	0.5	1.0	2.0
Negative moment at interior support	0	75	75	75
	≥1.0	90	75	45
Positive moment near midspan	0	60	60	60
	≥1.0	90	75	45

For *exterior* panels, the portion of the design moment that is not assigned to the column strip (Table 17.5) must be resisted by the corresponding half-middle strips. Again, linear interpolation between values shown in Table 17.5 is permitted by the ACI Code, Section 13.6.4.2. When no beams are used in an exterior panel, as in the case of flat slabs or flat plates with no edge (spandrel) beam, $\alpha_{f_1} = 0$, $C = 0$, and $\beta_t = 0$. This means that the end column provides the restraint to the exterior end of the slab. The applicable values of Table 17.5 for this special case are shown in Table 17.6 and Fig. 17.15.

From Table 17.6 it can be seen that when no edge beam is used at the exterior end of the slab, $\beta_t = 0$ and 100% of the design moment is resisted by the column strip. The middle strip

Table 17.4 Percentage of Moments in Two-Way Interior Slabs Without Beams ($\alpha_1 = 0$)

	Total Design Moment = $M_o = (w_u l_2)\left(\frac{l_{n1}^2}{8}\right)\frac{n!}{r!(n-r)!}$	
	Negative Moment	**Positive Moment**
Longitudinal moments in one panel	$-0.65M_o$	$\pm 0.35M_o$
Column strip	$0.75(-0.65M_o) = -0.49M_o$	$0.60(0.35M_o) = 0.21M_o$
Middle strip	$0.25(-0.65M_o) = 0.16M_o$	$0.40(0.35M_o) = 0.14M_o$

Table 17.5 Percentage of Longitudinal Moment in Column Strips, Exterior Panels (ACI Code, Section 13.6.4)

			Aspect Ratio l_2/l_1		
	$\alpha_{f_1} l_2/l_1$	β_t	**0.5**	**1.0**	**2.0**
Negative moment at exterior support	0	0	100	100	100
		≥ 2.5	75	75	75
	≥ 1.0	0	100	100	100
		≥ 2.5	90	75	45
Positive moment near midspan	0		60	60	60
	≥ 1.0		90	75	45
Negative moment at interior support	0		75	75	75
	≥ 1.0		90	75	45

Table 17.6 Percentage of Longitudinal Moment in Column and Middle Strips, Exterior Panels (For All Ratios of l_2/l_1), Given $\alpha_{f_1} = \beta_t = 0$

	%	Column Strip	Middle Strip	Final Moment as a Function of M_o and α_{ec} (Column Strip)
Negative moment at exterior support	100	$0.26M_o$	0	$\left[\frac{0.65}{(1+1/\alpha_{ec})}\right](M_o)$
Positive moment ($0.6 \times 0.52M_o$)	60	$0.312M_o$	$0.208M_o$	$\left[0.63 - \frac{0.28}{(1+1/\alpha_{ec})}\right](M_o)$
Negative moment at interior support ($0.75 \times 0.70M_o$)	75	$0.525M_o$	$0.175M_o$	$\left[0.75 - \frac{0.10}{(1+1/\alpha_{ec})}\right](M_o)$

will not resist any moment; therefore, minimum steel reinforcement must be provided. The ACI Code, Section 13.6.4.3, specifies that when the exterior support is a column or wall extending for a distance equal to or greater than three-fourths the transverse span length, l_2, used to compute M_o, the exterior negative moment is to be uniformly distributed across l_2. When beams are provided along the centerlines of columns, the ACI Code, Section 13.6.5, requires that beams must be proportioned to resist 85% of the moment in the column strip if $\alpha_{f_1}(l_2/l_1) \geq 1.0$. For values of $\alpha_{f_1}(l_2/l_1)$ between 1.0 and 0, the moment assigned to the beam is determined by linear interpolation. Beams must also be proportioned to resist additional moments caused by all loads

applied directly to the beams, including the weight of the projecting beam stem. The portion of the moment that is not assigned to the beam must be resisted by the slab in the column strip.

17.8.5 ACI Provisions for Effects of Pattern Loadings

In continuous structures, the maximum and minimum bending moments at the critical sections are obtained by placing the live load in specific patterns to produce the extreme values. Placing the live load on all spans will not produce either the maximum positive or negative bending moments. The maximum and minimum moments depend mainly on the following:

1. The ratio of live to dead load. A high ratio will increase the effect of pattern loadings.
2. The ratio of column to beam stiffnesses. A low ratio will increase the effect of pattern loadings.
3. Pattern loadings. Maximum positive moments within the spans are less affected by pattern loadings.

To determine the design factored moments in continuous structures, the ACI Code, Section 13.7.6, specifies the following:

1. When the loading pattern is known, the equivalent frame shall be analyzed for that load.
2. When the live load is variable but does not exceed $\frac{3}{4}$ of the dead load, $w_L \leq 0.75w_D$, or when all the panels is almost loaded simultaneously with the live load, it is permitted to analyze the frame with full factored live load on the entire slab system.
3. For other loading conditions, it is permitted to assume that the maximum positive factored moment near a midspan occurs with 0.75 of the full factored live load on the panel and alternate panels. For the maximum negative factored moment in the slab at a support, it is permitted to assume that 0.75 of the full factored live load is applied on adjacent panels only.
4. Factored moments shall not be taken less than the moments occurring with full factored live load on all continuous panels.

17.8.6 Reinforcement Details

After all the percentages of the static moments in the column and middle strips are determined, the steel reinforcement can be calculated for the negative and positive moments in each strip, as was done for beam sections in Chapter 4:

$$M_u = \phi A_s f_y \left(d - \frac{a}{2}\right) = R_u b d^2 \tag{17.15}$$

Calculate R_u and determine the steel ratio ρ using the tables in Appendix A or use the following equation:

$$R_u = \phi \rho f_y \left(1 - \frac{\rho f_y}{1.7 f'_c}\right) \tag{17.16}$$

where $\phi = 0.9$. The steel area is $A_s = \rho b d$. When the slab thickness limitations, as discussed in Section 17.4, are met, no compression reinforcement will be required. Fig. 13.3.8 of the ACI Code indicates the minimum length of reinforcing bars and reinforcement details for slabs without beams; it is reproduced here as Fig. 17.16. The spacing of bars in the slabs must not exceed the ACI limits of maximum spacing: 18 in. (450 mm) or twice the slab thickness, whichever is smaller.

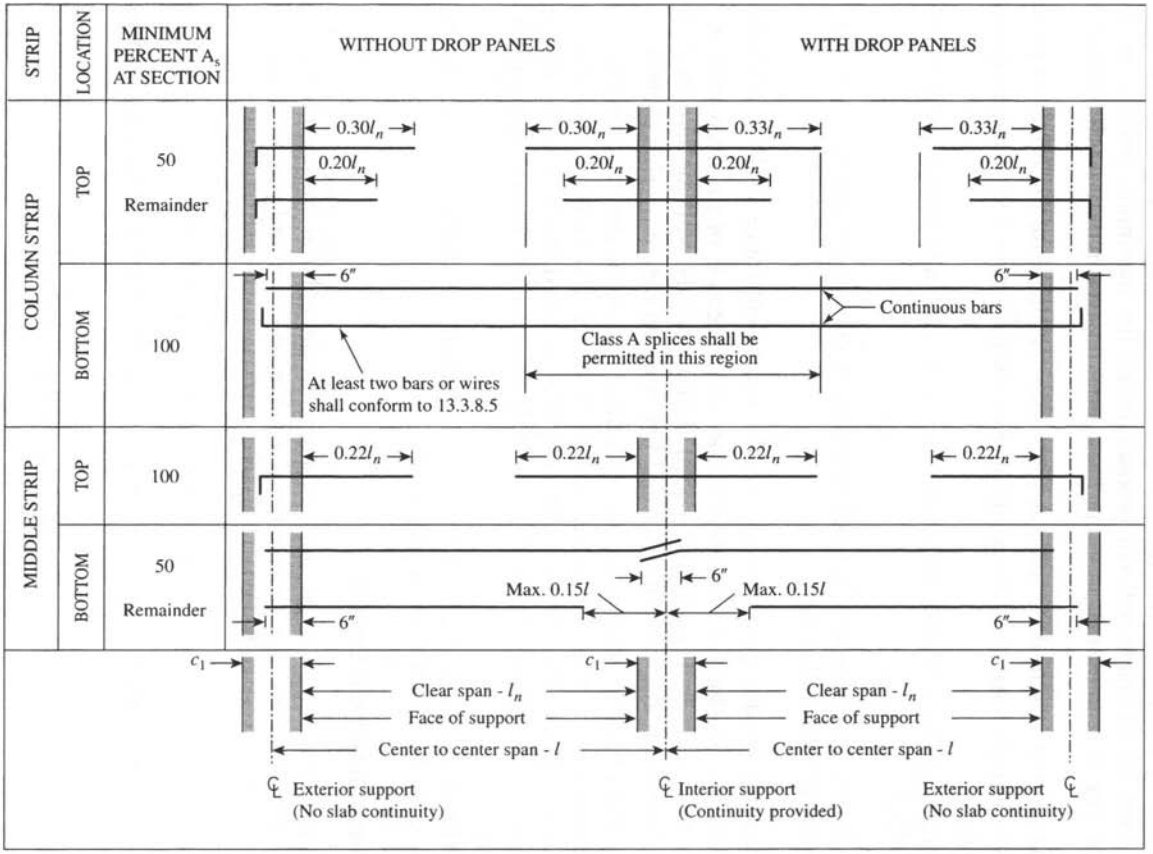


Figure 17.16 Minimum extensions for reinforcement in slabs without beams (ACI Code, Fig. 13.3.8). Courtesy of American Concrete Institute [14].

17.8.7 Modified Stiffness Method for End Spans

In this method, the stiffnesses of the slab end beam and of the exterior column are replaced by the stiffness of an equivalent column, K_{ec}. The flexural stiffness of the equivalent column, K_{ec}, can be calculated from the following expression:

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{K_t} \quad \text{or} \quad K_{ec} = \frac{\sum K_c}{1 + \sum K_c / K_t} \tag{17.17}$$

where

K_{ec} = flexural stiffness of the equivalent column
K_c = flexural stiffness of the actual column
K_t = torsional stiffness of the edge beam

The sum of the flexural stiffness of the columns above and below the floor slab can be taken as follows:

$$\sum K_c = 4E\left(\frac{I_{c1}}{L_{c1}} + \frac{I_{c2}}{L_{c2}}\right) \tag{17.18}$$

where I_{c1} and L_{c1} the moment of inertia and length of column above slab level and I_{c2} and L_{c2} = the moment of inertia and length of column below slab level. The torsional stiffness of the end beam, K_t, may be calculated as follows:

$$K_t = \sum \frac{9E_{cs}C}{l_2\left(1 - \frac{c_2}{l_2}\right)^3} \tag{17.19}$$

where

c_2 = size of the rectangular or equivalent rectangular column, capital, or bracket measured on transverse spans on each side of the column
E_{cs} = modulus of elasticity of the slab concrete
C = torsion constant determined from the following expression:

$$C = \sum\left(1 - 0.63\frac{x}{y}\right)\left(\frac{x^3 y}{3}\right) \tag{17.20}$$

where x is the shorter dimension of each component rectangle and y is the longer dimension of each component rectangle. In calculating C, the component rectangles of the cross-section must be taken in such a way as to produce the largest value of C.

The preceding expressions are introduced here and will also be used in Section 17.2,"Equivalent Frame Method."

If a panel contains a beam parallel to the direction in which moments are being determined, the torsional stiffness, K_t, given in Eq. 17.19 must be replaced by a greater value, K_{ta}, computed as follows:

$$K_{ta} = K_t \times \frac{I_{sb}}{I_s}$$

where

$I_s = \dfrac{I_2 h^3}{12}$ = moment of inertia of a slab that has a width equal to the full width between panel centerlines (excluding that portion of the beam stem extending above or below the slab)

$I_{sb} = I_s$, including the portion of the beam stem extending above or below the slab.

Cross-sections of some attached torsional members are shown in Fig. 17.17. Once K_{ec} is calculated, the stiffness ratio, α_{ec}, is obtained as follows:

$$\alpha_{ec} = \frac{K_{ec}}{\sum(K_s + K_b)} \tag{17.21}$$

where

$K_s = \dfrac{4E_{cs}I_s}{l_1}$ = flexural stiffness of the slab

$K_b = \dfrac{4E_{cb}I_b}{l_1}$ = flexural stiffness of the beam

I_b = gross moment of inertia of the longitudinal beam section

The distribution of the total static moment, M_o, in an exterior panel is given as a function of α_{ec} as follows:

$$\text{Interior negative factored moment} = \left[0.75 - \frac{0.1}{(1 + 1/\alpha_{ec})}\right] M_o$$

$$\text{Positive factored moment} = \left[0.63 - \frac{0.28}{(1 + 1/\alpha_{ec})}\right] M_o$$

$$\text{Exterior negative factored moment} = \left[\frac{0.65}{(1 + 1/\alpha_{ec})}\right] M_o$$

These values are shown for a typical exterior panel in Fig. 17.18. These factors take into consideration the effect of the stiffness of the exterior column as well as the slab end beam giving adequate distribution of moments.

17.8.8 Summary of the Direct Design Method (DDM)

Case 1. Slabs without beams (flat slabs and flat plates).

1. Check the limitation requirements explained in Section 17.8.1. If limitations are not met, DDM cannot be used.
2. Determine the minimum slab thickness (h_{min}) to control deflection using values in Table 17.1. Exterior panels without edge beams give the highest $h_{min}(l_n/30$ for $f_y = 60$ ksi). It is a common practice to use the same slab depth for all exterior and interior panels.
3. Calculate the factored loads, $W_u = 1.2W_D + 1.6W_L$.
4. Check the slab thickness, h, as required by one-way and two-way shear. If the slab thickness, h, is not adequate, either increase h or provide shear reinforcement.
5. Calculate the total static moment, M_o, in both directions (Eq. 17.11).

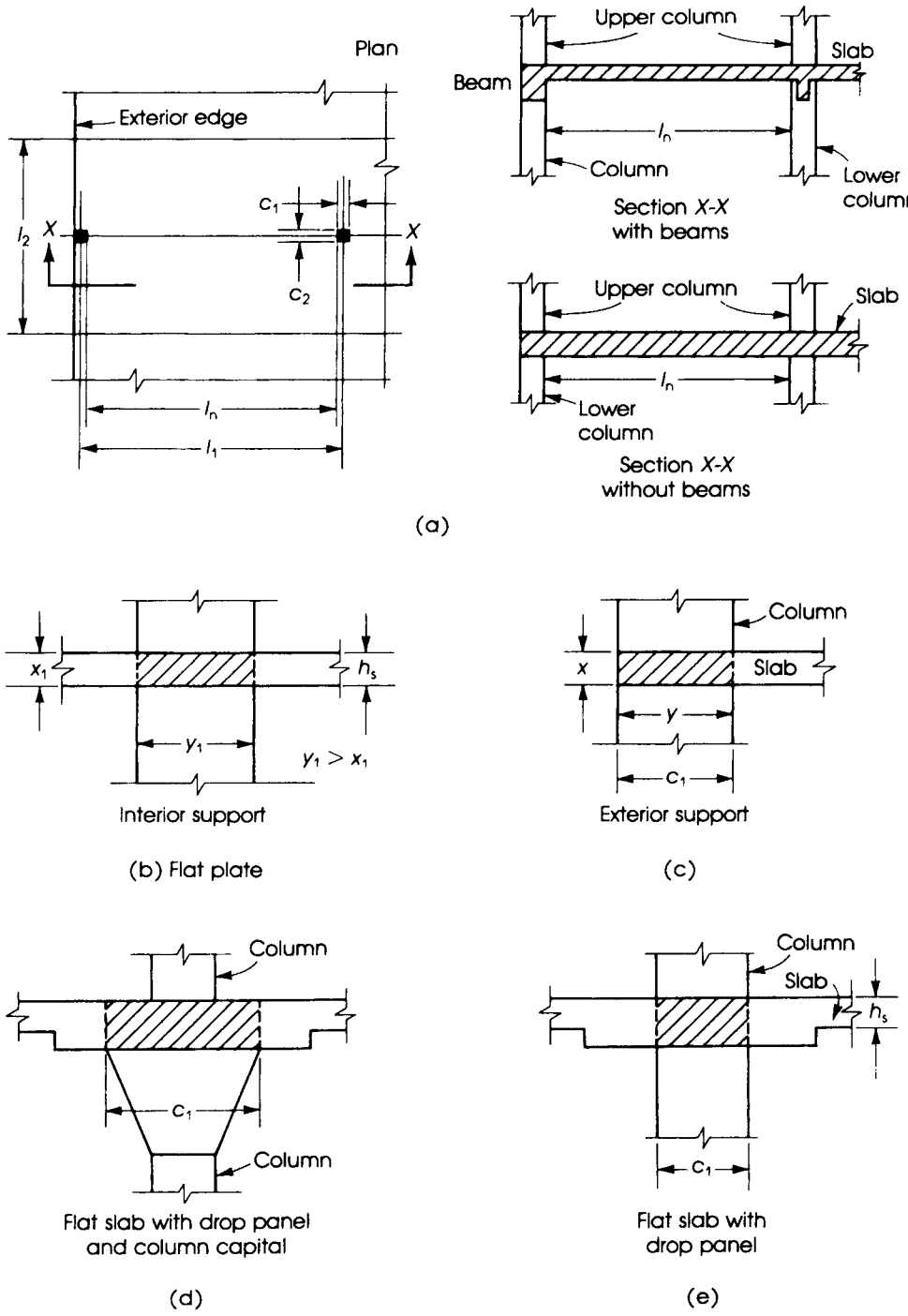
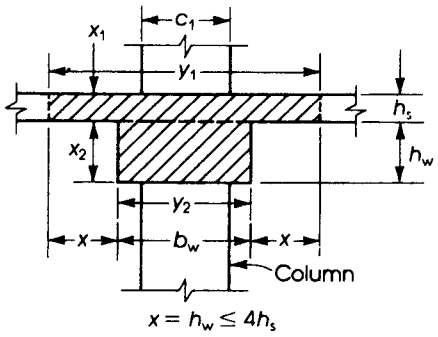
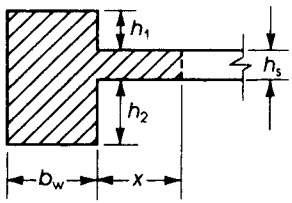


Figure 17.17 Cross-sections of some attached torsional members.



Slab beam floor system (interior)

(f)

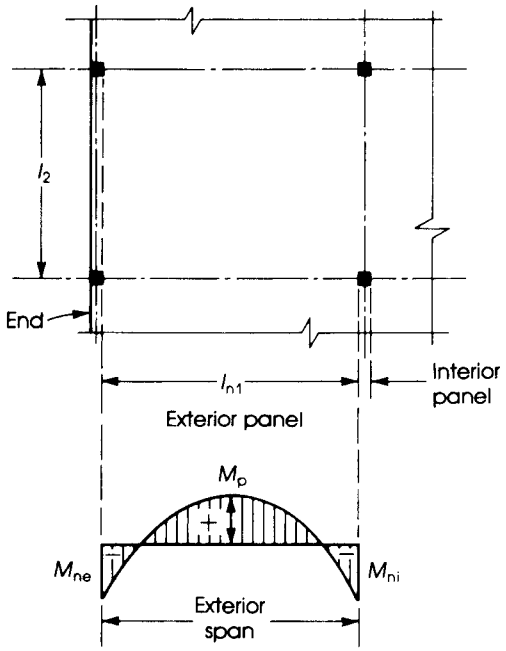


x = larger of h_1 or h_2
$x \le 4h_s$
$x = h_2 \le 4h_s$ when $h_1 = 0$

Slab beam floor system (exterior)

(g)

Figure 17.17 *(continued)*



$$M_o = (w_u l_2)\, l_n^2/8$$

$$M_p = \left(0.63 - \frac{0.28}{(1 + 1/\alpha_{ec})}\right) M_o$$

$$M_{ne} = \frac{0.65\, M_o}{(1 + 1/\alpha_{ec})}$$

$$M_{ni} = \left(0.75 - \frac{0.1}{(1 + 1/\alpha_{ec})}\right) M_o$$

Build up

Figure 17.18 Distribution of moments in an exterior panel.

6. Determine the distribution factors for the positive and negative moments in the longitudinal and transverse directions for each column and middle strip in both interior and exterior panels as follows:
   - **a.** For interior panels, use the moment factors given in Table 17.4 or Fig. 17.15.
   - **b.** For exterior panels without edge beams, the panel moment factors are given in Table 17.2 or Fig. 17.14 (Case 5). For the distribution of moments in the transverse direction, use

Table 17.6 or Fig. 17.15 for column-strip ratios. The middle strip will resist the portion of the moment that is not assigned to the column strip.

c. For exterior panels with edge beams, the panel moment factors are given in Table 17.2 or Fig. 17.14 (Case 4). For the distribution of moments in the transverse direction, use Table 17.5 for the column strip. The middle strip will resist the balance of the panel moment.

7. Determine the steel reinforcement for all critical sections of the column and middle strips and extend the bars throughout the slab according to Fig. 17.16.
8. Compute the unbalanced moment and check if transfer of unbalanced moment by flexure is adequate. If not, determine the additional reinforcement required in the critical width. (Refer to Section 17.10.)
9. Check if transfer of the unbalanced moment by shear is adequate. If not, increase h or provide shear reinforcement. (Refer to Section 17.10.)

Case 2. Slabs with interior and exterior beams.

1. Check the limitation requirements as explained in Section 17.8.1.
2. Determine the minimum slab thickness ($h_{\min}$) to control deflection using Eqs. 17.1 through 17.3. In most cases, Eq. 17.2 controls. Equation 17.1 should be calculated first, as shown in Example 17.1.
3. Calculate the factored load, W_u.
4. Check the slab thickness, h, according to one-way and two-way shear requirements. In general, shear is not critical for slabs supported on beams.
5. Calculate the total static moment, M_o in both directions (Eq. 17.17).
6. Determine the distribution factors for the positive and negative moments in the longitudinal and transverse directions for each column and middle strips in both interior and exterior panels as follows:
   a. For interior panels, use moment factors in Fig. 17.14 (Case 3) or Fig. 17.12. For the distribution of moments in the transverse direction, use Table 17.3 for column strips. The middle strips will resist the portion of the moments not assigned to the column strips. Calculate α_1 from Eq. 17.12.
   b. For exterior panels, use moment factors in Table 17.2 or Fig. 17.14 (Case 3). For the distribution of moments in the transverse direction, use Table 17.5 for the column strip. The middle strip will resist the balance of the panel moment.
   c. In both cases (a) and (b), the beams must resist 85% of the moment in the column strip when $\alpha_{f_1}(l_2/l_1) \geq 1.0$, whereas the ratio varies between 85% and 0% when $\alpha_{f_1}(l_2/l_1)$ varies between 1.0 and 0.
7. Determine the steel reinforcement for all critical sections in the column strip, beam, and middle strip; then extend the bars throughout the slab according to Fig. 17.16.
8. Compute the unbalanced moment and then check the transfer of moment by flexure and shear. (Refer to Section 17.10.)

Example 17.3

Using the direct design method, design the typical *interior flat-plate* panel shown in Figs. 17.6 and 17.19. The floor system consists of four panels in each direction with a panel size of 24 by 20 ft. All panels are supported by 20- by 20-in. columns, 12 ft long. The slab carries a uniform service live

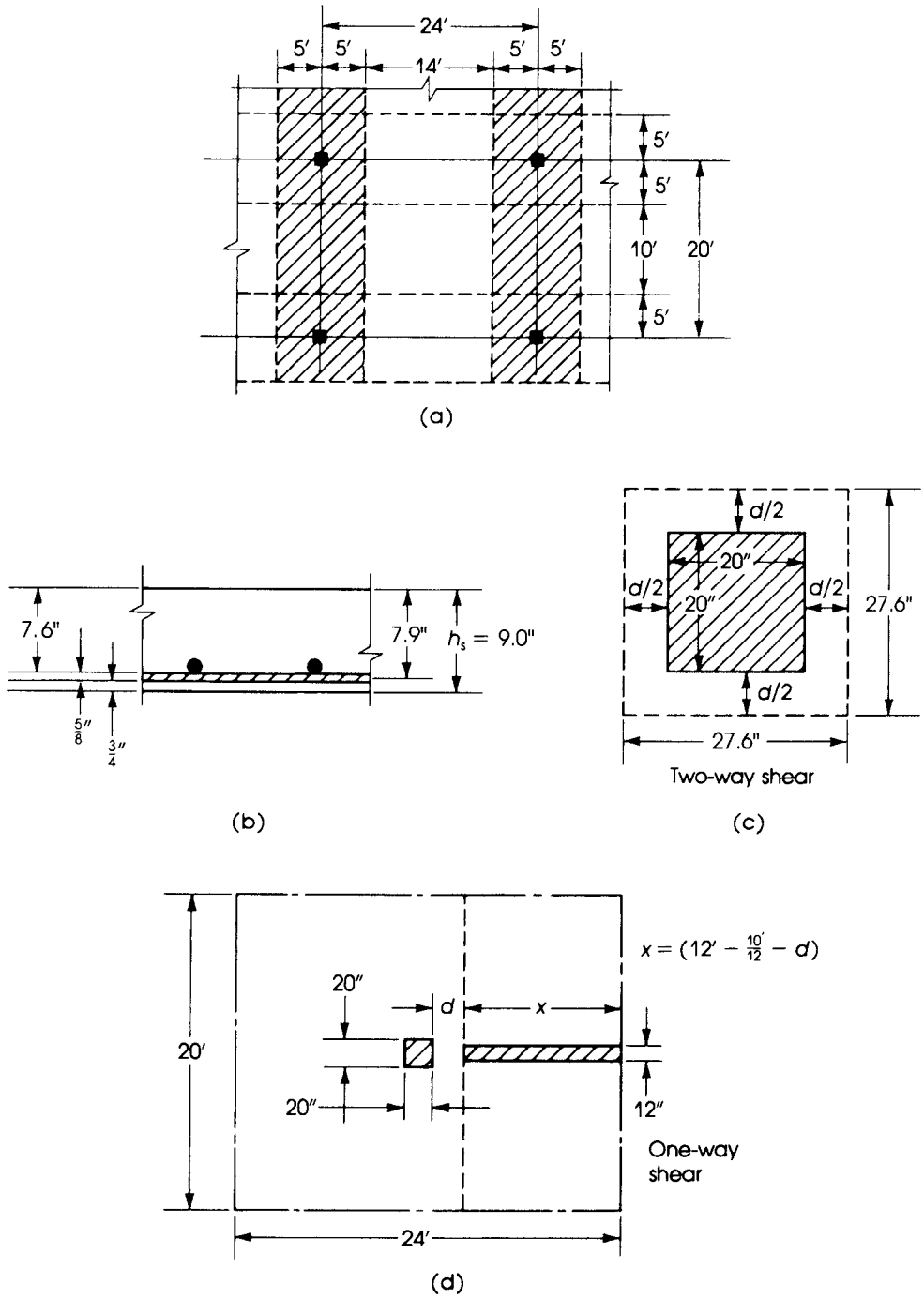


Figure 17.19 Example 17.3: interior flat plate.

load of 100 psf and a service dead load that consists of 24 psf of floor finish in addition to the slab self-weight. Use normal-weight concrete with $f'_c = 4$ ksi and $f_y = 60$ ksi.

Solution

1. Determine the minimum slab thickness using Table 17.1 for flat plates. From Example 17.1, a 9-in. slab thickness is adopted.
2. Calculate the factored loads:

$$w_D = 24 + \text{weight of slab} = 24 + \frac{9.0}{12} \times 150 = 136.5 \text{ psf}$$

$$w_u = 1.2 \times (136.5) + 1.6 \times (100) = 323 \quad \text{say, } 330 \text{ psf}$$

3. Check one- and two-way shears:
   a. Check punching shear at a distance $d/2$ from the face of the column (two-way action): Assuming $\frac{3}{4}$ in. concrete cover and no. 5 bars, then the average d is $9.0 - 0.75 - \frac{5}{8} = 7.6$ in. and $b_o = 4(20 + 7.16) = 110$ in. (See Fig. 17.19*c*).

$$V_u = \left[l_1 l_2 - \left(\frac{27.6}{12} \times \frac{27.6}{12}\right)\right] \times w_u = (24 \times 20 - 5.3) \times 0.330 = 156.7 \text{ K}$$

$$\phi V_c = \phi(4\sqrt{f'_c}) b_o d = \frac{0.75 \times 4}{1000} \times \sqrt{4000} \times 110 \times 7.6 = 158.6 \text{ K}$$

   which is greater than V_u.

   b. Check beam shear at a distance d from the face of the column; average d is 7.6 in. Consider a 1-ft strip (Fig. 17.19*d*), with the length of the strip being

$$x = 12 - \frac{10}{12} - \frac{7.6}{12} = 10.5 \text{ ft}$$

$$V_u = w_u(1 \times 10.5) = 0.330 \times 1 \times 10.5 = 3.47 \text{ K}$$

$$\phi V_c = \phi(2\lambda\sqrt{f'_c}) bd = \frac{0.75 \times 2 \times 1}{1000} \times \sqrt{4000} \times (12 \times 7.6) = 8.7 \text{ K}$$

   which is greater than $V_u = 3.47$ K. In normal loadings, one-way shear does not control.
4. Calculate the total static moments in the long and short directions. In the long direction,

$$M_{ol} = \frac{w_u l_2 l_{n_1}^2}{8} = \frac{0.33}{8} \times 20(22.33)^2 = 411.4 \text{ K·ft}$$

In the short direction,

$$M_{os} = \frac{w_u l_1 l_{n_2}^2}{8} = \frac{0.33}{8} \times 24 \times (18.33)^2 = 333 \text{ K·ft}$$

Because $l_2 < l_1$, the width of half a column strip in the long direction is $0.25 \times 20 = 5$ ft, and the width of the middle strip is $20 - 2 \times 5 = 10$ ft. The width of half the column strip in the short direction is 5 ft, and the width of the middle strip is $24 - 2 \times 5 = 14$ ft. To calculate the effective depth, d, in each direction, assume that steel bars in the short direction are placed on top of the bars in the long direction. Therefore, d(long direction) $= 9.0 - 0.75 - \frac{5}{16} = 7.9$ in. and d (short direction) $= 9.0 - 0.75 - \frac{5}{8} - \frac{5}{16} - = 7.3$ in. For practical applications, an average $d = 9 - 1.5 = 7.15$ in. can be used for both directions.

The design procedure can be conveniently arranged in a table form, as in Tables 17.7 and 17.8.

Table 17.7 Design of Interior Flat-Plate Panel (Long Direction)

	$M_o = 411.4$ K·ft $M_n = 0.65M_o = -267.4$ K·ft $M_p = +0.35M_o = +144$ K·ft			
	Column Strip		Middle Strip	
Long Direction	**Negative**	**Positive**	**Negative**	**Positive**
Moment distribution (%)	75	60	25	40
M_u (K·ft)	$0.75M_n = -201.6$	$0.6M_p = \pm 86.4$	$0.25M_n = -66.8$	$0.4M_p = \pm 57.16$
Width of strip b (in.)	120	120	120	120
Effective depth d (in.)	7.9	7.9	7.9	7.9
$R_u = \frac{M_u}{bd^2}$ (psi)	323	128	107	93
Steel ratio ρ (%)	0.633	0.262	0.2	0.175
$A_s = \rho bd$ (in.^{2})	6.00	2.48	1.92	1.66
Min. $A_s = 0.0018bh_s$ (in.^{2})	1.94	1.94	1.94	1.94
Bars selected (Straight)	20 no. 5	10 no. 5	10 no. 4	10 no. 4
Spacing $\leq 2h_s = 18$ in.	6 in.	12	12	12

Table 17.8 Design of Interior Flat-Plate Panel (Short Direction)

	$M_o = 333$ K·ft $M_n = 0.65M_o = -216.5$ K·ft $M_p = +0.35M_o = +116.5$ K·ft			
	Column Strip		Middle Strip	
Short Direction	**Negative**	**Positive**	**Negative**	**Positive**
Moment distribution (%)	75	60	25	40
M_u (K·ft)	$0.75M_n = -162.4$	$0.6M_p = \pm 69.9$	$0.25M_n = -54.1$	$0.4M_p = \pm 46.16$
Width of strip b (in.)	120	120	168	168
Effective depth d (in.)	7.3	7.3	7.3	7.3
$R_u = \frac{M_u}{bd^2}$ (psi)	305	131	73	62
Steel ratio ρ (%)	0.60	0.25	0.14	0.12
$A_s = \rho bd$ (in.^{2})	5.23	2.18	1.72	1.46
Min. $A_s = 0.0018bh_s$ (in.^{2})	1.94	1.94	2.72	2.72
Bars selected (Straight)	18 no. 5	10 no. 5	14 no. 4	14 no. 4
Spacing $\leq 2h_s = 18$ in.	6.7	12	12	12

The details for the bars selected for this interior slab are shown in Fig. 17.20 using the staight bar system. Minimum lengths of the bars must meet those shown in Fig. 17.16.

Straight bars and $f_y = 60$ ksi steel bars are more often preferred by contractors.

$$\text{Maximum spacing} = \frac{\text{width of panel}}{\text{no. of bars}} = \frac{168}{14} = 12 \text{ in.}$$

occurs at the middle strip in the short direction; this spacing of 12 in. is adequate, because it is less than $2h_s = 18$ in. and less than 18 in. specified by the ACI Code. Note that all steel ratios are less than $\rho_{max} = 0.018$. Thus $\phi = 0.9$.

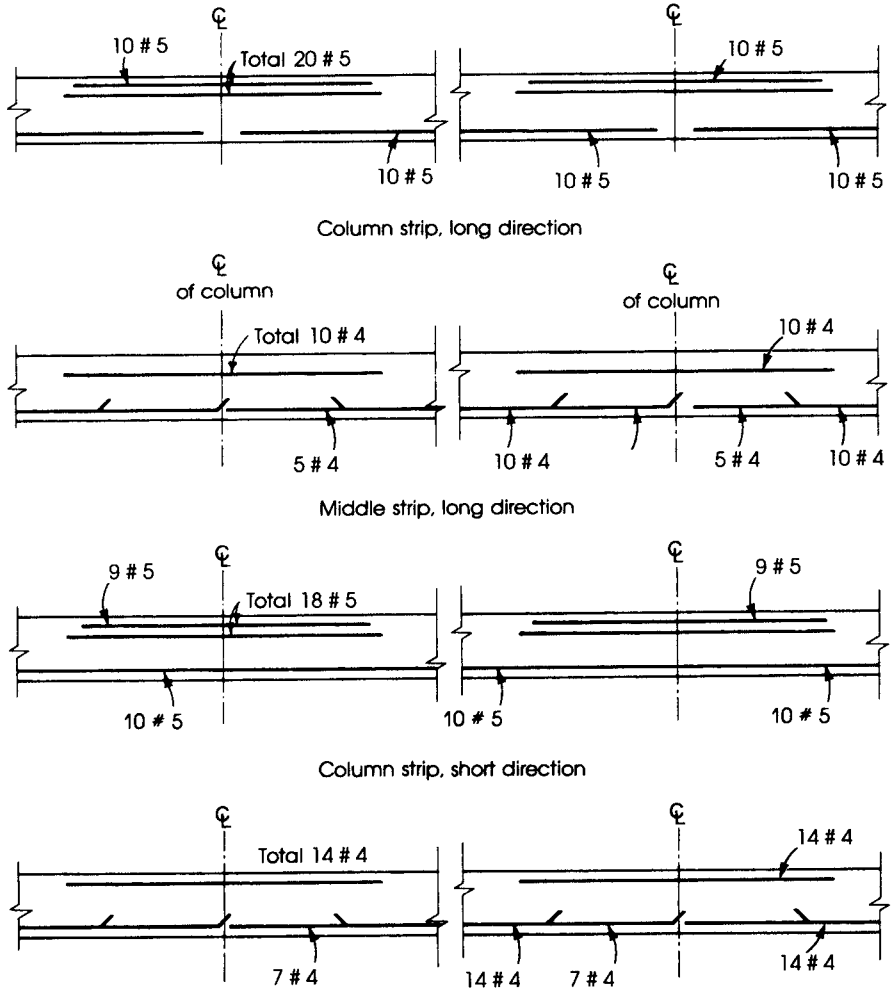


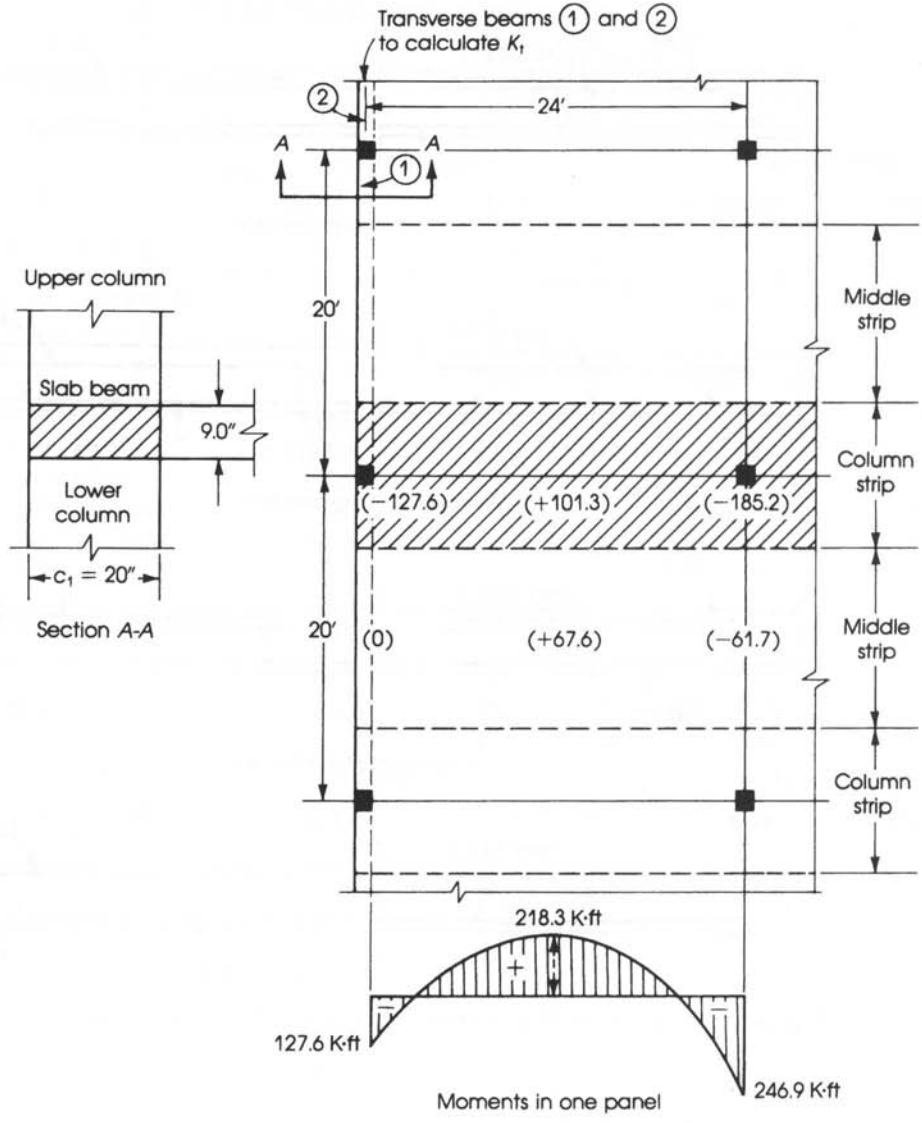
Figure 17.20 Example 17.3: reinforcement details. For bar length, refer to Fig. 17.16.

Example 17.4

Using the direct design method, design an *exterior flat-plate* panel that has the same dimensions, loads, and concrete and steel strengths given in Example 17.3. No beams are used along the edges (Fig. 17.21).

Solution

1. Determine the minimum slab thickness using Table 17.1 for flat plates. From Example17.1, a 9.0-in. slab thickness is adopted.
2. Calculate factored loads: $W_u = 330$ psf. (See Example 17.3.)
3. Check one- and two-way shear (refer to Example 17.3 and Fig. 17.19).
   a. Check punching shear at an interior column, $V_u = 156.7 < \phi V_c = 158.6$ K.
   b. Check one-way shear: $V_u = 3.47$ K $< \phi\ V_c = 8.7$ K.



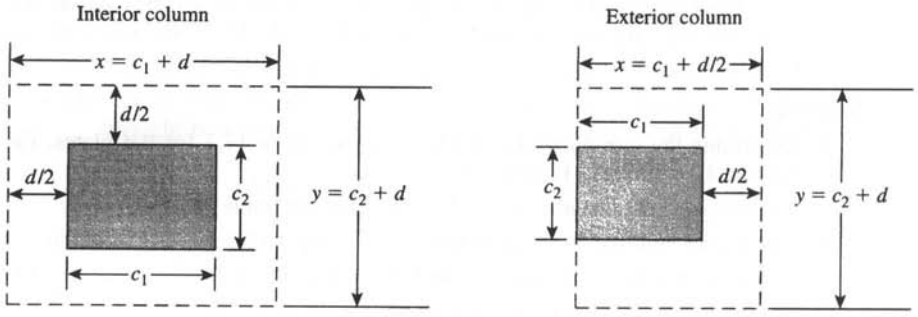


Figure 17.21 Example 17.4: distribution of bending moments.

c. Check punching shear at the exterior column: $d = 7.6$ in.

$$x = 20 + \frac{d}{2} = 20 + \frac{7.6}{2} = 23.8 \text{ in.} = 1.98 \text{ ft}$$

$$y = 20 + d = 20 + 7.6 = 27.6 \text{ in.} = 2.30 \text{ ft}$$

$$b_o = 2x + y = 75.2 \text{ in.}$$

$$V_u = \left[20\left(12 + \frac{10}{12}\right) - 1.98(2.30)\right]0.33 = 83.2 \text{ K}$$

$$\phi V_c = \phi 4\sqrt{f'_c} b_o d = 108.4 \text{ K} > V_u$$

d. Check punching shear at a corner column: $d = 7.6$ in.

$$x = y = 20 + \frac{d}{2} = 23.8 \text{ in.} = 1.98 \text{ ft}$$

$$b_o = x + y = 47.6 \text{ in.}$$

$$V_u = \left[\left(10 + \frac{10}{12}\right)\left(12 + \frac{10}{12}\right) - (1.98)(1.98)\right]0.33 = 44.6 \text{ K}$$

$$\phi V_c = \phi 4\sqrt{f'_c} b_o d = 68.6 \text{ K} > V_u$$

4. Calculate the total static moments. From Example 17.3,

$$M_{ol} \text{ (long direction)} = 411.4 \text{ K·ft} \qquad d = 7.9 \text{ in.}$$

$$M_{os} \text{ (short direction)} = 333 \text{ K·ft} \qquad d = 7.3 \text{ in.}$$

The width of the column strip is 120 in., and the width of the middle strip is 168 in.

5. Calculate the design moments in the long direction: $l_1 = 24$ ft. (Refer to Table 17.5 or Fig. 17.15). The distribution of the total moment, M_{ol}, in the column and middle strips is computed as follows:

a. Column strip:

$$\text{Interior negative moment} = -0.525M_o = -0.525(411.4) = -216 \text{ K·ft}$$

$$\text{Positive moment within span} = 0.312M_o = 0.312(411.4) = +128.4 \text{ K·ft}$$

$$\text{Exterior negative moment} = -0.26M_o = -0.26(411.14) = -107 \text{ K·ft}$$

b. Middle strip:

$$\text{Interior negative moment} = -0.175M_o = -0.175 \times 411.4 = -72 \text{ K·ft}$$

$$\text{Positive moment within span} = 0.208M_o = 0.208 \times 411.4 = +85.6 \text{ K·ft}$$

$$\text{Exterior negative moment} = 0$$

6. Calculate the design moments in the short direction: $l_s = 20$ ft. It will be treated as an interior panel because it is continuous on both sides. Referring to Table 17.4 or Fig. 17.15, the distribution of the total moment, M_{os}, in the column and middle strips is computed as follows:

a. Column strip:

$$\text{Negative moment} = 0.49M_o = -0.49(333) = -163.2 \text{ K·ft}$$

$$\text{Positive moment} = +0.21M_o = +0.21(333) = +70.0 \text{ K·ft}$$

Table 17.9 Design of Exterior Flat-Plate Panel for Example 17.4 ($d = 7.9$ in.)

	Column Strip			Middle Strip		
Long Direction	**Exterior**	**Positive**	**Interior**	**Exterior**	**Positive**	**Interior**
M_u (K·ft)	−107.06	±128.4	−216.0	0	±85.6	−72.0
b (in.)	120	120	120	120	120	120
$R_u = \frac{M_u}{bd^2}$ (psi)	172	206	346	0	138	116
Steel ratio ρ (%)	0.33	0.4	0.682	0	0.262	0.22
$A_s = \rho bd$	3.11	3.75	6.47	0	2.48	2.10
Min. $A_s = 0.0018bh_s$	1.94	1.94	1.94	1.94	1.94	1.94
Bars selected (Straight)	12 no. 5	12 no. 5	22 no. 5	10 no. 4	14 no. 4	14 no. 4
Spacing # 18 in.	10	10	5.5	12	8.5	8.5
Short Direction	**Column Strip**			**Middle Strip**		
M_u (K·ft)	−163.2	±70.0		−53.3	±46.6	
Width of strip b (in.)	120	120		168	168	
d(in.)	7.3	7.3		7.3	7.3	
$R_u = \frac{M_u}{bd^2}$ (psi)	306	131		71	63	
Steel ratio ρ (%)	0.6	0.25		0.133	0.12	
$A_s = \rho bd$ (in.^{2})	5.26	2.20		1.63	1.47	
Min. $A_s = 0.0018bh_s$	1.94	1.94		2.72	2.72	
Bars selected (Straight)	18 no. 5	8 no. 5		14 no. 4	14 no. 4	
Spacing 18 in.	6.67	15		12	12	

b. Middle strip:

$$\text{Negative moment} = -0.16M_o = -0.16(333) = -53.3 \text{ K·ft}$$

$$\text{Positive moment} = +0.14M_o = +0.14(333) = +46.6 \text{ K·ft}$$

The design procedure can be conveniently arranged in Table 17.9. The details for bars selected are shown in Fig. 17.22 using the straight-bar system in the long direction. Details of reinforcement in the short direction will be similar to Fig. 17.20 using the bars chosen in Table 17.9.

Note that all steel ratios are less than $\rho_{max} = 0.018$. Thus $\phi = 0.9$.

Example 17.5

Repeat Example 17.4 using the modified stiffness method. (Similar calculations are needed for the equivalent frame method, Section 17.12.)

Solution

1. Steps 1 through 4 will be the same as in Example 17.4.
2. Calculate the equivalent column stiffness, K_{ec}:

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{K_t}$$

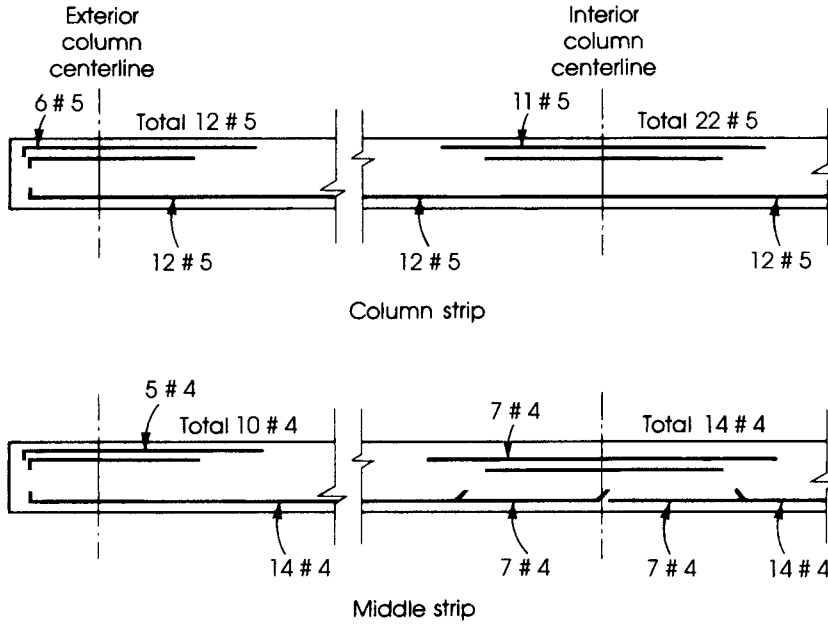


Figure 17.22 Example 17.4: reinforcement details (longitudinal direction). For bar lengths, refer to Fig. 17.16.

It can be assumed that the part of the slab strip between exterior columns acts as a beam resisting torsion. The section of the slab-beam is 20 in. (width of the column) × 9.0 in. (thickness of the slab), as shown in Fig. 17.21.

a. Determine the torsional stiffness, K_t, from Eq. 17.20:

$$C = \left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3} \qquad x = 9 \text{ in} \qquad y = 20 \text{ in.}$$

$$C = \left(1 - 0.63 \times \frac{9}{20}\right)\frac{9^3 \times 20}{3} = 3482 \text{ in.}^4$$

$$K_t = \frac{9E_c C}{l_2\left(l - \frac{c_2}{l_2}\right)^3} = \frac{9E_c \times 3482}{(20 \times 12)\left(1 - \frac{20}{20 \times 12}\right)^3} = 170E_c$$

For the two adjacent slabs (on both sides of the column) acting as transverse beams,

$$K_t = 2 \times 170E_c = 340E_c$$

b. Calculate the column stiffness, K_c; the column height $L_c = 12$ ft:

$$K_c = \frac{4E_c I_c}{L_c} = \frac{4E_c}{(12 \times 12)} \times \frac{(20)^4}{12} = 370.4E_c$$

For two columns above and below the floor slab,

$$K_c = 2 \times 370.14E_c = 740.18E_c$$

c. Calculate K_{ec}:

$$\frac{1}{K_{ec}} = \frac{1}{740.8E_c} + \frac{1}{340E_c}$$

To simplify the calculations, multiply by $1000E_c$:

$$\frac{1000E_c}{K_{ec}} = \frac{1000}{740.8} + \frac{1000}{340} = 4.29 \qquad K_{ec} = 233E_c$$

3. Calculate slab stiffness and the ratio α_{ec}:

$$K_s = \frac{4E_cI_s}{l_1} \qquad h_s = 9 \text{ in.} \qquad l_2 = 20 \text{ ft} \qquad I_s = \frac{l_2h_s^3}{12}$$

$$K_s = \frac{4E_c}{(24 \times 12)} \times \frac{(20 \times 12)(9.0)^3}{12} = 202.5E_c$$

$$\alpha_{ec} = \frac{K_{ec}}{\sum (K_s + K_b)} \tag{17.21}$$

$K_b = 0$ (no beams are provided)

thus

$$\alpha_{ec} = \frac{233E_c}{202.5E_c} = 1.15$$

Let

$$Q = \left(1 + \frac{1}{\alpha_{ec}}\right) = 1 + \frac{1}{1.15} = 1.87$$

4. Calculate the design moments in the long direction: $l_1 = 24$ ft. The distribution of moments in one panel is shown in Fig. 17.18. The interior negative moment is

$$M_{nt} = \left[0.75 - \frac{0.10}{Q}\right] M_{ol} = \left(0.75 - \frac{0.10}{1.87}\right)(411.4) = -286.6 \text{ K·ft}$$

The positive moment is

$$M_p = \left[0.63 - \frac{0.28}{Q}\right] M_{ol}$$

$$= \left(0.63 - \frac{0.28}{1.87}\right)(411.4) = 197.6 \text{ K·ft}$$

The exterior negative moment is

$$M_{ne} = \frac{0.65}{Q}(M_{ol}) = \frac{0.65}{1.87}(411.4) = -143.0 \text{ K·ft}$$

5. Calculate the distribution of panel moments in the transverse direction to column and middle strips. The moments M_{ni}, M_p, and M_{ne} are distributed as follows (refer to Table 17.6):

a. The interior moment $(M_{nl}) = -286.6$ K·ft is distributed 75% for the column strip and 25% for the middle strip.

$$\text{Column strip} = 0.75(-286.6) = -215 \text{ K·ft}$$

$$\text{Middle strip} = 0.25(-286.6) = -71.6 \text{ K·ft}$$

b. The positive moment, $M_p = 197.6$ K·ft, is distributed 60% for the column strip and 40% for the middle strip.

$$\text{Column strip} = 0.6(197.6) = 118.5 \text{ K·ft}$$

$$\text{Middle strip} = 0.4(197.6) = 79.1 \text{ K·ft}$$

c. The exterior negative moment, $M_{ne} = -143$ K·ft, is distributed according to Table 17.5:

$$\beta_t = \frac{E_c C}{2E_c I_s} = \frac{C}{2I_s}$$

The concrete of slab and column are the same.

$$I_s = (20 \times 12)\frac{(9.0)^3}{12} = 14{,}580 \text{ in.}^4$$

$$\beta_t = \frac{3482}{2 \times 14{,}580} = 0.119$$

$$\alpha_{f_1} = \frac{E_{cb} I_b}{E_{cs} I_s} = 0 \qquad \alpha_{f_1}\frac{l_2}{l_1} = 0 \qquad \frac{l_2}{l_1} = 0.83$$

From Table 17.5 and by interpolation between $\beta_t = 0$ (percentage = 100%) and $\beta_t = 2.5$ (percentage = 100%) for $\beta_t = 0.1119$, the percentage is 99%. The exterior negative moment in the column strip is $0.99(-143.10) = -142$ K·ft and in the middle strip, it is -1.10 K·ft. It is practical to consider that the column strip carries in this case 100% of $M_{ne} = -143$ K·ft.

6. Determine the reinforcement required in the long direction in a table form similar to Example 17.4. Results will vary slightly from those of Table 17.9.

7. Comparison of results between Examples 17.4 and 17.5 shows that the exterior moment in the column strip (− 143 K·ft) is greater than that calculated in Example 17.4 (− 107 K·ft) by about 34%, whereas the positive moment (± 118.15) is reduced by about 8% (relative to ± 128.14). Other values are almost compatible.

Example 17.6

Design an interior panel of the two-way slab floor system shown in Fig. 17.7. The floor consists of six panels in each direction, with a panel size of 24 by 20 ft. All panels are supported on 20- by 20-in. columns, 12 ft long. The slabs are supported by beams along the column lines with the cross-sections shown in the figure. The service live load is to be taken as 100 psf, and the service dead load consists of 22 psf of floor finish in addition to the slab weight. Use normal-weight concrete with $f'_c = 3$ ksi, $f_y = 60$ ksi, and the direct design method.

Solution

1. The limitations required by the ACI Code are met. Determine the minimum slab thickness using Eqs. 17.1 and 17.2. The slab thickness has been already calculated in Example 17.3, and a 7.0-in. slab can be adopted. Generally, the slab thickness on a floor system is controlled by a corner panel, as the calculations of h_{min} for an exterior panel give greater slab thickness than for an interior panel.

2. Calculate factored loads:

$$w_D = 22 + \text{weight of slab} = 22 + \frac{7}{12} \times 150 = 109.5 \text{ psf}$$

$$w_u = 1.2(109.5) + 1.6(100) = 292 \text{ psf}$$

3. The shear stresses in the slab are not critical. The critical section is at a distance d from the face of the beam. For a 1-ft width:

$$V_u = w_u\left(10 - \frac{1}{2}\text{beam width} - d\right)$$

$$= 0.292\left(10 - \frac{16}{2 \times 12} - \frac{6}{12}\right) = 2.58 \text{ K}$$

$$\phi V_c = \phi(2\lambda\sqrt{f'_c})bd = \frac{0.75 \times 2 \times 1 \times \sqrt{3000} \times 12 \times 6}{1000} = 6.3 \text{ K} > V_u$$

4. Calculate the total static moments in the long and short directions:

$$M_{ol} = \frac{w_u}{8} l_2 (l_{n1})^2 = \frac{0.292}{8}(20)(22.33)^2 = 364.0 \text{ K·ft}$$

$$M_{os} = \frac{w_u}{8} l_1 (l_{n2})^2 = \frac{0.292}{8}(24)(18.33)^2 = 294.3 \text{ K·ft}$$

5. Calculate the design moments in the long direction: $l_1 = 24$ ft.

   a. Distribution of moments in one panel:

$$\text{Negative moment } (M_n) = 0.65M_{ol} = 0.65 \times 364 = -236.6 \text{ K·ft}$$

$$\text{Positive moment } (M_p) = 0.35M_{ol} = 0.35 \times 364 = 127.4 \text{ K·ft}$$

   b. Distributions of panel moments in the transverse direction to the beam, column, and middle strips are as follows:

$$\frac{l_2}{l_1} = \frac{20}{24} = 0.83 \quad \alpha_{f_1} = \alpha_s = \frac{EI_b}{EI_s} = 3.27 \quad \text{(from Example 17.2)}$$

$$\alpha_{f_1}\frac{l_2}{l_1} = 3.27 \times 0.83 = 2.71 > 1.0$$

   c. Distribute the negative moment, M_n. The portion of the interior negative moment to be resisted by the column strip is obtained from Table 17.3 by interpolation and is equal to 80% (for $l_2/l_1 = 0.183$ and $\alpha_{f_1}(l_2/l_1) > 1.0$).

$$\text{Column strip} = 0.18M_n = 0.18 \times 236.16 = -189.13 \text{ K·ft}$$

$$\text{Middle strip} = 0.12M_n = 0.12 \times 236.16 = -47.13 \text{ K·ft}$$

   Because $\alpha_{f_1}(l_2/l_1) > 1.0$, the ACI Code, Section 13.6.5, indicates that 85% of the moment in the column strip is assigned to the beam and the balance of 15% is assigned to the slab in the column strip.

$$\text{Beam} = 0.85 \times 189.3 = -160.9 \text{ K·ft}$$

$$\text{Column strip} = 0.15 \times 189.3 = -28.4 \text{ K·ft}$$

$$\text{Middle strip} = -47.3 \text{ K·ft}$$

   d. Distribute the positive moment, M_p. The portion of the interior positive moment to be resisted by the column strip is obtained from Table 17.3 by interpolation and is equal to 80% (for $l_2/l_1 = 0.83$ and $\alpha_{f_1}(l_1/l_2) > 1.0$).

$$\text{Column strip} = 0.8M_p = 0.8 \times 127.4 = +101.9 \text{ K·ft}$$

$$\text{Middle strip} = 0.2M_p = 0.2 \times 127.4 = +25.5 \text{ K·ft}$$

Since $\alpha_{f_1}(l_2/l_1) > 1.0$, 85% of the moment in the column strip is assigned to the beam and the balance of 15% is assigned to the slab in the column strip:

$$\text{Beam} = 0.85 \times 101.9 = +86.6 \text{ K·ft}$$

$$\text{Column strip} = 0.15 \times 101.9 = +15.3 \text{ K·ft}$$

$$\text{Middle strip} = +25.5 \text{ K·ft}$$

Moment details are shown in Fig. 17.23.

6. Calculate the design moment in the short direction: span = 20 ft. The procedure is similar to step 5.

$$\text{Negative moment } (M_n) = 0.65M_{os} = 0.65 \times 294.3 = -191.3 \text{ K·ft}$$

$$\text{Positive moment } (M_p) = 0.35M_{os} = 0.35 \times 294.3 = +103.0 \text{ K·ft}$$

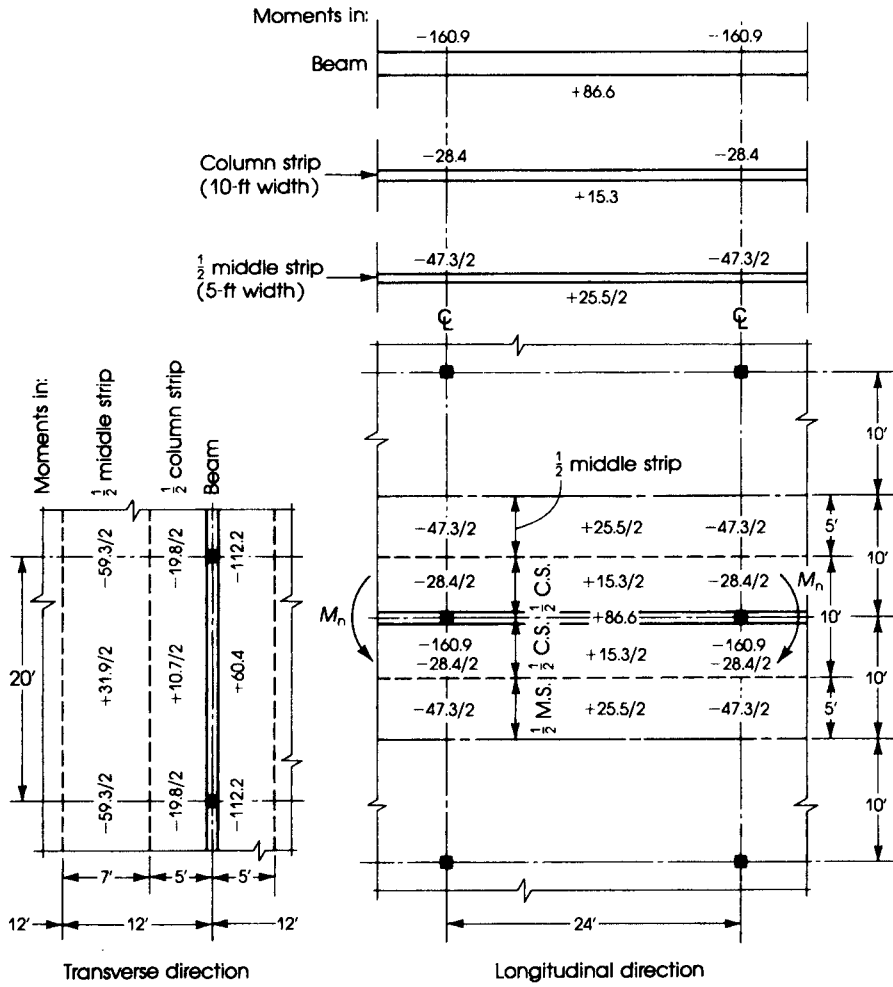


Figure 17.23 Example 17.6: interior slab with beams. All moments are in K·ft.

Distribution of M_n and M_p to beam, column, and middle strips:

$$\frac{l_2}{l_1} = \frac{24}{20} = 1.2 \quad \alpha_{f_1} = \alpha_s = \frac{EI_b}{EI_s} = 2.72 \quad \text{(from Example 17.2)}$$

$$\alpha_{f_1}\frac{l_2}{l_1} = 2.72 \times 1.2 = 3.26 > 1.0$$

The percentages of the column strip negative and positive moments are obtained from Table 17.3 by interpolation. (For $l_2/l_1 = 1.12$ and $\alpha_{f_1}(l_2/l_1) > 1.0$, the percentage is 69%.)

$$\text{Column strip negative moment} = 0.69M_n = 0.69 \times 191.3 = -132 \text{ K·ft}$$

$$\text{Middle strip negative moment} = 0.31M_n = 0.31 \times 191.3 = -59.3 \text{ K·ft}$$

Since $\alpha_{f_1}(l_2/l_1) > 1.0$, 85% of $-$ 132 K·ft is assigned to the beam. Therefore,

$$\text{Beam negative moment} = 0.85 \times 132 = -112.2 \text{ K·ft}$$

$$\text{Column strip negative moment} = 0.15 \times 132 = -19.8 \text{ K·ft}$$

$$\text{Beam positive moment} = (0.85)(0.69 \times 103.0) = +60.4 \text{ K·ft}$$

$$\text{Column strip positive moment} = (0.15)(0.69 \times 103.0) = +10.7 \text{ K·ft}$$

$$\text{Middle strip positive moment} = (1 - 0.169)(103.10) = \pm 31.19 \text{ K·ft}$$

7. The steel reinforcement required and number of bars are shown in Table 17.10. Note all steel ratios are less than $\rho_{max} = 0.0135$. Thus, $\phi = 0.9$.

Example 17.7

Using the direct design method, determine the negative and positive moments required for the design of the exterior panel (no. 2) of the two-way slab system with beams shown in Fig. 17.7. Use the loads and the data given in Example 17.6.

Solution

1. Limitations required by the ACI Code are satisfied in this problem. Determine the minimum slab thickness, h_s, using Eqs. 17.1 and 17.2 and the following steps: Assume $h_s = 7.10$ in. The sections of the interior and exterior beams are shown in Fig. 17.7. Note that the extension of the slab on each side of the beam $x = y = 15$ in.
2. **a.** The moments of inertia for the *interior* beams and slabs were calculated earlier in Example 17.2:

$$I_b(\text{in both directions}) = 22{,}453 \text{ in.}^4$$

$$I_s(\text{in the long direction}) = 6860 \text{ in.}^4$$

$$I_s(\text{in the short direction}) = 8232 \text{ in.}^4$$

b. Calculate I_b and I_s for the *edge* beam and end slab.

$$I_b(\text{edge beam}) = \left[\frac{27}{12}(7)^3 + (27 \times 7)(5.37)^2\right] + \left[\frac{12}{12}(15)^3 + (12 \times 15)(5.63)^2\right]$$

$$= 15{,}302 \text{ in.}^4$$

Calculate I_s for the end strip parallel to the edge beam, which has a width $= \frac{24}{2}$ ft $+$ $\frac{1}{2}$ column width $= 12 + \frac{10}{12} = 12.83$ ft.

$$I_s(\text{end slab}) = \frac{(12.83 \times 2)}{12}(7)^3 = 4401 \text{ in.}^4$$

Table 17.10 Design of an Interior Two-Way Slab with Beams

	Long Direction			
	Column Strip		Middle Strip	
M_u (K·ft)	−28.14	±15.13	−47.13	±25.15
Width of strip (in.)	120	120	120	120
Effective depth (in.)	6.0	6.0	6.0	6.0
$R_u = \frac{M_u}{bd^2}$ (psi)	79	43	132	71
Steel ratio ρ	0.0016	Low	0.0026	0.0015
$A_s = \rho bd$ (in.^{2})	1.15	Low	1.87	1.08
Min. $A_s = 0.10018bh_s$ (in.^{2})	1.52	1.52	1.52	1.52
Selected bars	8 no. 4	8 no. 4	10 no. 4	8 no. 4
	Short Direction			
	Column Strip		Middle Strip	
M_u (K·ft)	−19.18	±10.17	−59.13	±31.19
Width of strip (in.)	120	120	168	168
Effective depth (in.)	5.5	5.5	5.5	5.5
$R_u = \frac{M_u}{bd^2}$ (psi)	65	35	196	105
Steel ratio ρ	Low	Low	0.0039	0.002
$A_s = \rho bd$ (in.^{2})	Low	Low	3.60	1.85
Min. $A_s = 0.0018bh_s$ (in.^{2})	1.52	1.52	2.10	2.10
Selected bars	8 no. 4	8 no. 4	18 no. 4	10 no. 4

3. a. Calculate $\alpha_f (\alpha_f = EI_b/EI_s)$:

$$\alpha_l(\text{long direction}) = \frac{22{,}453}{6860} = 3.27$$

$$\alpha_s(\text{short direction}) = \frac{22{,}453}{8232} = 2.72$$

$$\alpha(\text{edge beam}) = \frac{15{,}302}{4401} = 3.48$$

$$\text{Average}\alpha = \alpha_{fm} = \frac{3.27 + 2.72 \times 2 + 3.48}{4} = 3.05$$

b. β = ration of long to short clear span.

$$\frac{22.33}{18.33} = 1.22$$

c. Calculate h_s:

$$\text{Min. } h_s = \frac{(22.33 \times 12)(0.8 + 0.005 \times 60)}{36 + (5 \times 1.22)[3.05 - 0.2]} = 5.52 \text{ in.}$$

but this value must not be less than

$$\text{Min. } h_s = \frac{294.756}{36 + 9(1.22)} = 6.30 \text{ in.} \quad \text{(controls)}$$

Use $h_s = 7$ in. > 3.15 in. (minimum code limitations).

4. Calculate factored loads:

$$w_u = 292 \text{ psf} \quad \text{(from Example 17.6)}$$

5. Calculate total static moments:

$$M_{ol} = 364.0 \text{ K·ft} \qquad M_{os} = 294.3 \text{ K·ft} \qquad \text{(from previous example)}$$

6. Calculate the design moments in the short direction (span = 20 ft): Because the slab is continuous in this direction, the moments are the same as those calculated in Example 17.6 and shown in Fig. 17.23 for an interior panel.

7. Calculate the moments in one panel using the coefficients given in Table 17.2 or Fig. 17.14 (Case 3):

$$\text{Interior negative moment } (M_{ni}) = 0.7M_o = 0.7 \times 364 = -254.8 \text{ K·ft}$$

$$\text{Positive moment within span } (M_p) = 0.57M_o = 0.57 \times 364 = +207.5 \text{ K·ft}$$

$$\text{Exterior negtive moment } (M_{ne}) = 0.16M_o = 0.16 \times 364 = -58.2 \text{ K·ft}$$

Note: If the modified stiffness method is used, then $C = 9528$, $K_t = 1520E_c$, $K_c = 370E_c$, $K_b = 312E_c$, $K_s = 95E_c$, $K_{ec} = 498E_c$, and $\alpha_{ec} = 1.22$. The interior negative moment becomes −253.13 K·ft (same as before). The positive moment becomes −173.19 K·ft (16% decrease) and the exterior moment becomes −128.16 K·ft (220% increase).

8. Distribute the panel moments to beam, column, and middle strips:

$$\frac{l_2}{l_1} = \frac{20}{24} = 0.83 \quad \alpha_{f_1} = \alpha_s = 3.27$$

$$\alpha_{f_1}\frac{l_2}{l_1} = 3.27 \times 0.83 = 2.71 > 1.0$$

Calculate C:

$$C = \sum\left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$$

Divide the section of the edge beam into two rectangles in such a way as to obtain maximum C. Use for a beam section 12 by 22 in., $x_1 = 12$ in., $y_1 = 22$ in., and a slab section 7 by 15 in., $x_2 = 7$ in., and $y_2 = 15$ in.

$$C = \left(1 - 0.63 \times \frac{12}{22}\right)\left(\frac{12^3 \times 22}{3}\right) + \left(1 - 0.63 \times \frac{7}{15}\right)\left(\frac{7^3 \times 15}{3}\right)$$

$$= 9528 \text{ in.}^4$$

$$\beta_t = \frac{E_{ch}C}{2E_{cs}I_s} = \frac{9528}{2 \times 6860} = 0.69$$

a. Distribute the interior negative moment, M_{ni}: Referring to Table 17.5 and by interpolation, the percentage of moment assigned to the column strip (for $l_2/l_1 = 0.83$ and $\alpha_{f_1} l_2/l_1 > 1.0$ is 80%.

$$\text{Column strip} = 0.8 \times 254.8 = -203.8 \text{ K/ft}$$

$$\text{Middle strip} = 0.2 \times 254.8 = -51.0 \text{ K·ft}$$

Because $\alpha_{f1}\, l_2/l_1 > 1.0$, 85% of the moment in the column strip is assigned to the beam. Therefore,

$$\text{Beam} = 0.85 \times 203.8 = -173.3 \text{ K·ft}$$

$$\text{Column strip} = 0.15 \times 203.18 = -30.6 \text{ K·ft}$$

$$\text{Middle strip} = -51.0 \text{ K·ft}$$

b. Distribute the positive moment, M_p: Referring to Table 17.5 and by interpolation, the percentage of moment assigned to the column strip is 80% (85% of this value is assigned to the beam). Therefore,

$$\text{Beam} = (0.85)(0.8 \times 207.5) = +141.1 \text{ K·ft}$$

$$\text{Column strip} = (0.15)(0.8 \times 207.5) = 24.9 \text{ K·ft}$$

$$\text{Middle strip} = 0.2 \times 207.5 = +41.5 \text{ K·ft}$$

c. Distribute the exterior negative moment, M_{ne}: Referring to Table 17.5 and by interpolation, the percentage of moment assigned to the column strip (for $l_2/l_1 = 0.83$, $\alpha_{f1}\, l_2/l_1 > 1.0$, and $\beta_t = 0.69$) is 94%, and 85% of the moment is assigned to the beam. Therefore,

$$\text{Beam} = (0.85)(0.94 \times 58.2) = -46.5 \text{ K·ft}$$

$$\text{Column strip} = (0.15)(0.94 \times 58.2) = -8.2 \text{ K·ft}$$

$$\text{Middle strip} = 0.06 \times 58.2 = -3.5 \text{ K·ft}$$

17.9 DESIGN MOMENTS IN COLUMNS

When the analysis of the equivalent frames is carried out by the direct design method, the moments in columns due to the unbalanced loads on adjacent panels are obtained from the following equation, which is specified by the ACI Code, Section 13.6.9:

$$M_u = 0.07[(w_d + 0.5w_l)l_2l_n^2 - w'_d l'_2(l'_n)^2] \tag{17.22a}$$

If the modified stiffness method using K_{ec} and α_{ec} is used, then the moment M_u is computed as follows:

$$M_u = \frac{0.08[(w_d + 0.5w_l)l_2l_n^2 - w'_d l'_2(l'_n)^2]}{\left(1 + \dfrac{1}{\alpha_{ec}}\right)} \tag{17.22b}$$

where

w_d and w_l = factored dead and live loads on the longer span
w'_d = factored dead load on the shorter span
l_n and l'_n = length of the longer and shorter spans, respectively

$$\alpha_{ec} = \frac{K_{ec}}{\sum(K_s + K_b)} \tag{17.21}$$

The moment in Eq. 17.22 should be distributed between the columns above and below the slab at the joint in proportion to their flexural stiffnesses (Fig. 17.24). For equal spans $l_2 = l'_2$ and $l_n = l'_n$,

$$M_u = 0.07(0.5w_l l_2 l_n^2) \tag{17.23a}$$

$$M_u = \frac{0.08(0.5w_l l_2 l_n^2)}{\left(1 + \dfrac{1}{\alpha_{ec}}\right)} \tag{17.23b}$$

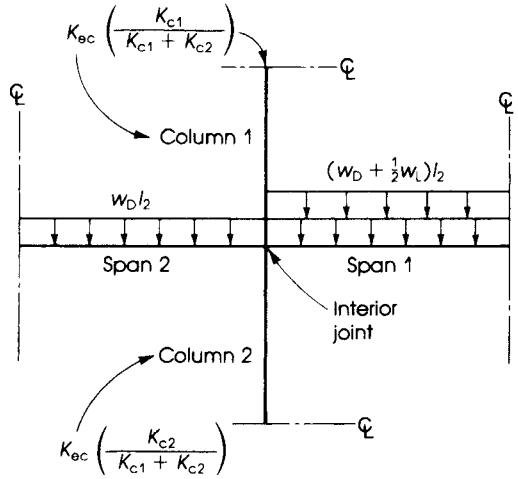


Figure 17.24 Interior column loading.

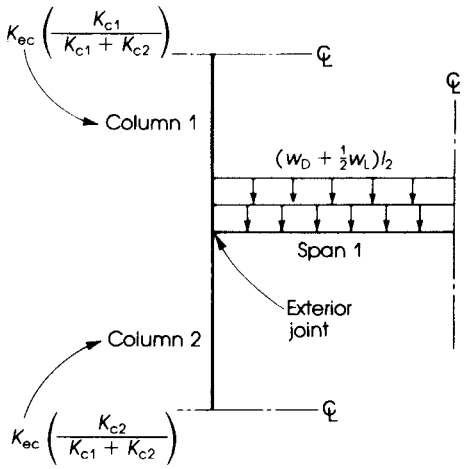


Figure 17.25 Exterior column loading.

The development of these equations is based on the assumption that half the live load acts on the longer span, whereas the dead load acts on both spans. Equation 17.22 can also be applied to an exterior column by assuming the shorter span length is 0 (Fig. 17.25).

17.10 TRANSFER OF UNBALANCED MOMENTS TO COLUMNS

17.10.1 Transfer of Moments

In the analysis of an equivalent frame in a building, moments develop at the slab-column joints due to lateral loads, such as wind, earthquakes, or unbalanced gravity loads, causing unequal moments in the slab on opposite sides of columns. A fraction of the unbalanced moment in the slabs must be transferred to the columns by flexure, and the balance must be transferred

by vertical shear acting on the critical sections for punching shear. Approximately 60% of the moment transferred to both ends of the column at a joint is transferred by flexure, and the remaining 40% is transferred by eccentric shear (or torque) at the section located at $d/2$ from the face of the column [14,15]. The ACI Code, Section 13.5.3, states that the fraction of the unbalanced moment transferred by flexure M_f at a slab-column connection is determined as follows (ACI Eq. 13.1):

$$M_f = \gamma_f M_u \tag{17.24}$$

$$\gamma_f = \frac{1}{1 + \left(\frac{2}{3}\sqrt{\frac{c_1 + d}{c_2 + d}}\right)} = \frac{1}{1 + \left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \tag{17.25}$$

and the moment transferred by shear is

$$M_v = (1 - \gamma_f)M_u = M_u - M_f \tag{17.26}$$

where c_1 and c_2 are the lengths of the two sides of a rectangular or equivalent rectangular column, $b_1 = (c_1 + d)$, and $b_2 = (c_2 + d)$. When $c_1 = c_2$, $M_f = 0.6M_u$, and $M_v = 0.4M_u$.

17.10.2 Concentration of Reinforcement Over the Column

For a direct transfer of moment to the column, it is necessary to concentrate part of the steel reinforcement in the column strip within a specified width over the column. The part of the moment transferred by flexure, M_f, is considered acting through a slab width equal to the transverse column width c_2 plus $1.15h_s$ on each side of the column or to the width $(c_2 + 3h_s)$ (ACI Code, Section 13.5.3). Reinforcement can be concentrated over the column by closer spacing of bars or the use of additional reinforcement.

17.10.3 Shear Stresses Due to M_v

The shear stresses produced by the portion of the unbalanced moment, M_v, must be combined with the shear stresses produced by the shearing force, V_u, due to vertical loads. Both shear stresses are assumed acting around a periphery plane located at a distance $d/2$ from the face of the column [16], as shown in Fig. 17.26. The equation for computing the shear stresses is

$$v_{1,2} = \frac{V_u}{A_c} \pm \frac{M_v C}{J_c} \tag{17.27}$$

where

A_c = area of critical section around the column
J_c = polar moment of inertia of the areas parallel to the applied moment in addition to that of the end area about the centroidal axis of the critical section

For an interior column,

$$A_c = 2d(x + y) \tag{17.28}$$

and

$$J_c = \frac{d}{2}\left(\frac{x^3}{3} + x^2 y\right) + \frac{xd^3}{6} \tag{17.29}$$

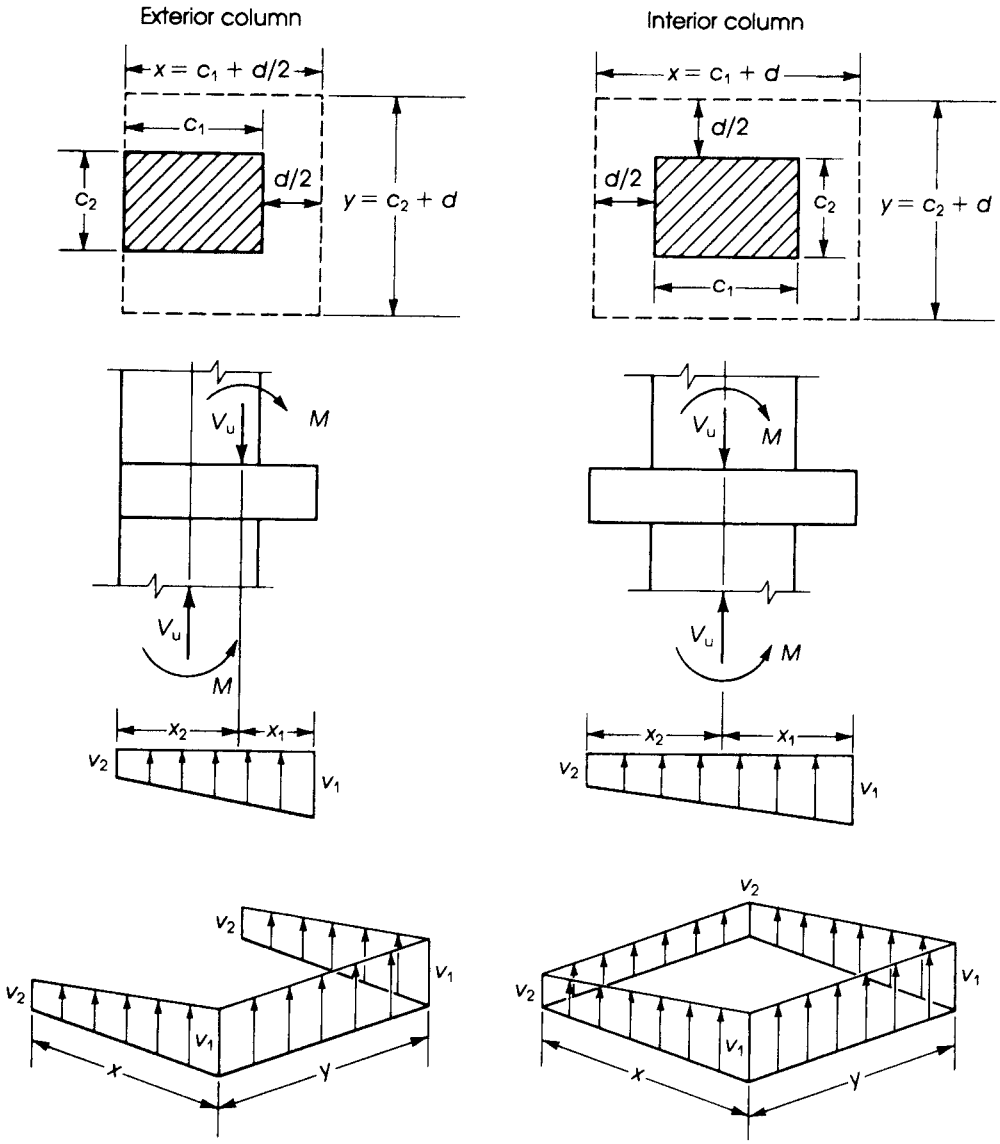


Figure 17.26 Shear stresses due to V_u and M.

For an exterior column,

$$A_c = d(2x + y) \tag{17.30}$$

and

$$J_c = \frac{2dx^3}{3} - (2x + y)dx_1^2 + \frac{xd^3}{6} \tag{17.31}$$

where x, x_1, and y are as shown in Fig. 17.26. The maximum shear stress, $v_1 = V_u/A_c + M_v C/J_c$, must be less than $\phi(4\sqrt{f'_c})$; otherwise, shear reinforcement should be provided.

Example 17.8

Determine the moments at the exterior and interior columns in the long direction of the flat plate in Example 17.4.

Solution

1. Find the exterior column moment. From Examples 17.4 and 17.5,

$$w_d = (136.5)(1.2) = 0.16 \text{ ksf}$$

$$0.5w_l = 0.5 \times (1.6 \times 100) = 80\text{psf}$$

$$l_2 = l_2' = 20 \text{ ft} \qquad l_n = l_n' = 22.33 \text{ ft} \qquad \left(1 + \frac{1}{\alpha_{ec}}\right) = 1.87$$

The unbalanced moment to be transferred to the exterior column using Eq. 17.22b is

$$M_u = \frac{0.08}{1.87}[(0.16 + 0.08)(20)(22.33)^2 - 0] = 102 \text{ K·ft}$$

If Eq. 17.22a is used, $M_u = 168$ K·ft, which is a conservative value.

2. At an interior support, the slab stiffness on both sides of the column must be used to compute α_{ec}:

$$\alpha_{ec} = \frac{K_{ec}}{\sum (K_s + K_b)} \tag{17.21}$$

From Example 17.5, $K_{ec} = 233E_c$, $K_s = 202.5E_c$, and $K_b = 0$. Therefore,

$$\alpha_{ec} = \frac{233E_c}{(2)202.5E_c} = 0.58$$

$$\left(1 + \frac{1}{\alpha_{ec}}\right) = 1 + \frac{1}{0.58} = 2.72$$

From Eq. 17.22b, the unbalanced moment at an interior support is

$$M_u = \frac{0.08}{2.72}[(0.16 + 0.08)(20)(22.33)^2 - 0.16(20)(22.33)^2] = 23 \text{ K·ft}$$

If Eq. 17.22a is used, $M_u = 42$ K·ft, which is a conservative value.

Example 17.9

For the flat plate in Example 17.4, calculate the shear stresses in the slab at the critical sections due to unbalanced moments and shearing forces at an interior and exterior column. Check the concentration of reinforcement and torsional requirements at the exterior column. Use $f_c' = 4$ ksi and $f_y = 60$ ksi.

Solution

1. The unbalanced moment at the interior support is $M_u = 20$ K·ft (Example 17.8), where $\gamma_f = 0.6$ (because $c_1 = c_2 = 20$ in.). The moment to be transferred by flexure is

$$M_f = \gamma_f M_u = 0.6 \times 23 = 13.8 \text{ K·ft}$$

The moment to be transferred by shear is

$$M_v = 23 - 13.8 = 9.2 \text{ K·ft}$$

Alternatively, moments calculated from Eq. 17.22a may be used producing higher shear stresses.

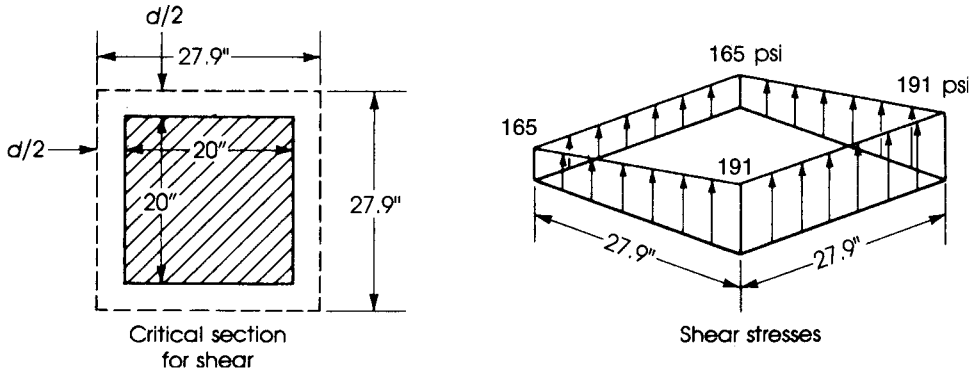


Figure 17.27 Example 17.9: shear stresses at interior column due to unbalanced moment.

Using $d = 7.9$ in. (Example 17.4),

$$V_u = 0.33\left[20 \times 24 - \left(\frac{27.9}{12}\right)^2\right] = 156.6 \text{ K}$$

From Fig. 17.27,

$$A_c = 4(27.9)(7.9) = 882 \text{ in.}$$

$$J_c = \frac{d}{2}\left(\frac{x^3}{3} + x^2 y\right) + \frac{xd^3}{6}$$

$$= \frac{7.9}{2}\left[\frac{(27.9)^3}{3} + (27.9)^2(27.9)\right] + \frac{27.9}{6}(7.9)^3 = 114{,}670 \text{ in.}^4$$

$$v_{\max} = \frac{156{,}600}{882} + \frac{9.2(12{,}000)(27.9/2)}{114{,}670}$$

$$= 178 + 13 = 191 \text{ psi}$$

$$v_{\min} = 178 - 13 = 165 \text{ psi}$$

Allowable v_c is $\phi 4\sqrt{f'_c} = 0.75 \times 4\sqrt{4000} = 190$ psi > 190 psi

2. For the exterior column, the unbalanced moment to be transferred by flexure M_f at a slabcolumn joint is equal to $\gamma_f M_u$, where $M_u = 102$ K·ft. Note that $c_1 = c_2 = 20$ in., $d = 7.9$ in. in the longitudinal direction, and $\gamma_f = 0.6$ for square columns.

$$M_f = 0.6(102) = 61.2 \text{ K·ft}$$

The moment to be transferred by shear is

$$M_v = M_u - M_f = 102 - 61.2 = 40.8 \text{ K·ft}$$

3. For transfer by shear at exterior column, the critical section is located at a distance $d/2$ from the face of the column (Fig. 17.28).

$$W_u = 330 \text{ psf}$$

$$V_u = 0.33\left(20 \times 12.83 - \frac{23.95}{12} \times \frac{27.9}{12}\right) = 83.1 \text{ K}$$

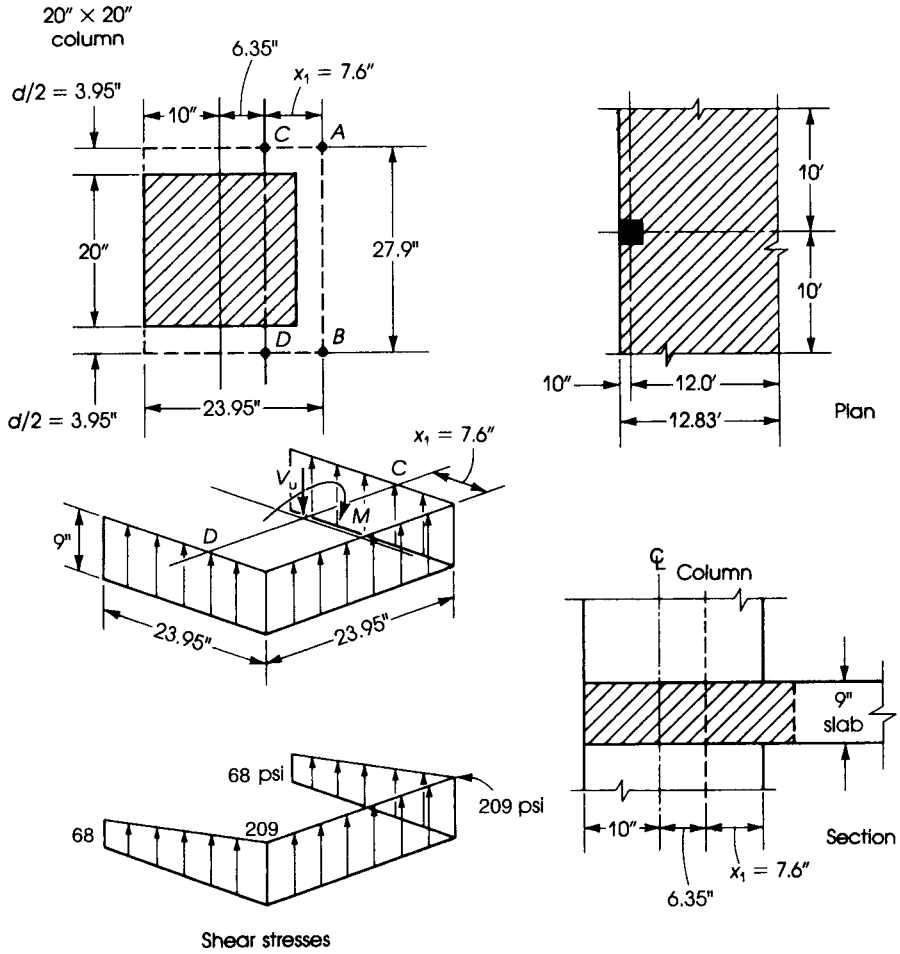


Figure 17.28 Example 17.9: shear stresses at exterior column due to unbalanced moment.

Locate the centroid of the critical section by taking moments about AB:

$$2\left(23.95 \times \frac{23.95}{2}\right) = (2 \times 23.95 + 27.9)x_l$$

Therefore, $x_l = 7.6$ in. The area of the critical section A_c is $2(23.95 \times 7.9) + (27.9 \times 7.9) = 599$ in.^{2} Calculate $J_c = I_x + I_y$ for the two equal areas (7.19×23.195) with sides parallel to the direction of moment and the area (7.19×27.19) perpendicular to the direction of moment, all about the axis through CD.

$$J_c = I_y + I_x = \sum\left(\frac{bh^3}{12} + Ax^2\right)$$

$$= 2\left[7.9\,\frac{(23.95)^3}{12} + (7.9 \times 23.95)\left(\frac{23.95}{2} - 7.6\right)^2\right]$$

$$+2\left[\frac{23.95}{12}(7.9)^3\right] + [(27.9 \times 7.9)(7.6)^2] = 52{,}760 \text{ in.}^4$$

or by using Eq. 17.31 for an exterior column. Calculate the maximum and minimum nominal shear stresses using Eq. 17.27:

$$v_{max} = \frac{V_u}{A_c} + \frac{M_v c}{J_c} = \frac{83{,}100}{599} + \frac{40.2(12{,}000)(7.6)}{52{,}760} = 209 \text{ psi}$$

$$v_{min} = 68 \text{ psi}$$

$$\text{Allowable } v_c = \phi 4\sqrt{f'_c} = 0.75 \times 4\sqrt{4000} = 190 \text{ psi.}$$

Shear stress is greater than the allowable v_c, so increase the slab thickness or use shear reinforcement.

4. Check the concentration of reinforcement at the exterior column; that is, check that the flexural capacity of the section is adequate to transfer the negative moment into the exterior column. The critical area of the slab extends $1.5h_s$ on either side of the column, giving an area $(20 + 3 \times 9) = 47$ in. wide and 9 in. deep. The total moment in the 120-in.-wide column strip is 107 K·ft, as calculated in Example 17.4 (step 5). The moment in a width, $c_2 + 3h_s = 47$ in., is equal to $107(\frac{47}{120}) = 41.9$ K·ft.

If equal spacing in the column strip is used, then the additional reinforcement within the 47-in. width will be needed for a moment equal to $M_f - 41.9 = 66 - 41.9 = 24.1$ K·ft. The required $A_s = 0.73$ in.^{2} and four no. 4 bars ($A_s = 0.8$ in.^{2}) may be used. An alternative solution is to arrange the reinforcement within the column strip to increase the reinforcement within a width of 47 in. The amount of steel needed within this width should be enough to resist a moment of 0.6 times the negative moment in the column strip, or $0.6 \times 107 = 64.2$ K·ft.

$$A_s = \frac{M_u}{\phi f_y \left(d - \frac{a}{2}\right)} \quad \text{assume} \quad a = 1.0 \text{ in.}$$

$$A_s = \frac{64.2(12)}{0.9 \times 60(7.9 - 0.5)} = 1.93 \text{ in.}^2$$

$$\text{Check: } a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.93 \times 60}{0.85 \times 4 \times 47} = 0.73 \text{ in.}$$

Use 10 no. 4 bars within a width 47 in. divided equally at both sides from the center of the column (Fig. 17.29). Additional reinforcement of four no. 4 bars, as indicated before, provides a better solution.

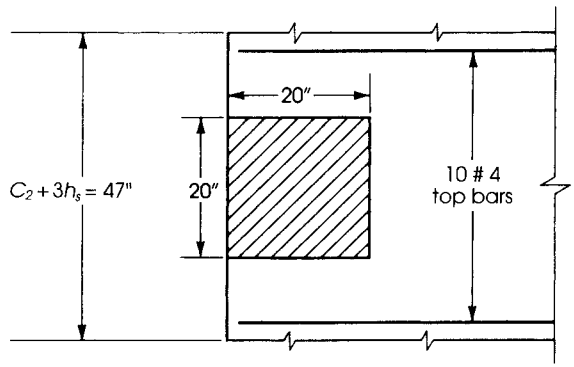


Figure 17.29 Example 17.9: concentration of reinforcement within exterior column strip.

5. Torque on slab: The torque from both sides of the exterior column is equal to 40% of the column strip moment.

$$T_u = 0.4(107) = 42.8 \text{ K·ft}$$

$$\text{Torque on each side : } \frac{42.8}{2} = 21.4 \text{ K·ft} = 257 \text{ K·in.}$$

A slab section of width equal to the column width will be assumed to resist the torsional stresses:

$$T_u = \frac{1}{3} v_{tu} \sum x^3 y$$

where $x = 9$ in. and $y = 20$ in. The critical section is at a distance d from the face of the column (Fig. 17.30). Assuming that the torque varies in a parabolic curve to the center of the slab, then the torque at a distance d is

$$T_u = 257\left(\frac{140 - 7.9}{140}\right)^2 = 229 \text{ K·in.}$$

For torsional strength of concrete, $A_{cp} = 9 \times 20 = 180$ in., $P_{cp} = 2(9 + 20) = 58$ in. By Eq. 15.19, $\phi T_{cp} = 0.75(4)\sqrt{4000}(180)^2/58 = 106$ K·in. $T_a = 106/4 = 26.5$ K·in. $< T_u$.
Torsional reinforcement is needed. The required closed stirrups and the additional longitudinal bars are determined as explained in Chapter 15. The final section is shown in Fig. 17.30. It is advisable to provide an edge beam between the exterior columns to increase the torsional stiffness of the slab.

Example 17.10

Determine the shear reinforcement required for an interior flat plate panel considering the following: Punching shear is $V_u = 195$ K, slab thickness $= 9$ in., $d = 7.5$ in., $f'_c = 4$ ksi, $f_y = 60$ ksi, and column size is 20×20 in.

Solution

1. Determine $\phi V_c = \phi 4\sqrt{f'_c} b_o d$ for two-way shear.

$$b_o = 4(20 + d) = 4(20 + 7.5) = 110 \text{ in.}$$

$$\phi V_c = 0.75(4)\sqrt{4000}(110)(7.5) = 156.3 \text{ K}$$

Because $V_u = 195$ K $> \phi V_c$, shear reinforcement is required.

2. Maximum allowable ϕV_n using shear reinforcement is equal to $\phi 6\sqrt{f'_c} b_o d = 1.5(\phi V_c) = 234.5$ K Because $\phi V_n > V_n$, shear reinforcement can be used.

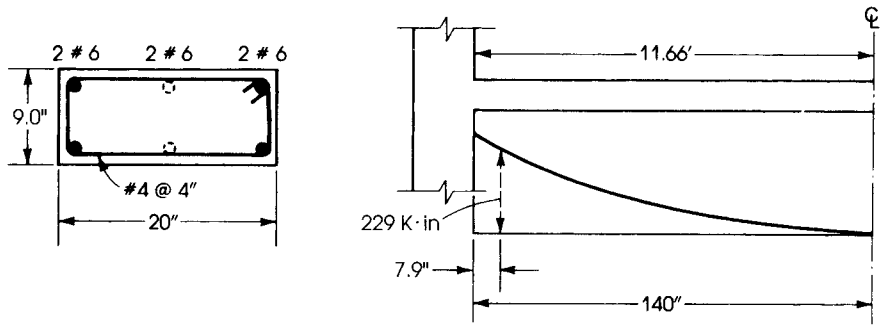


Figure 17.30 Example 17.9: reinforcement in edge of slab to resist torque.

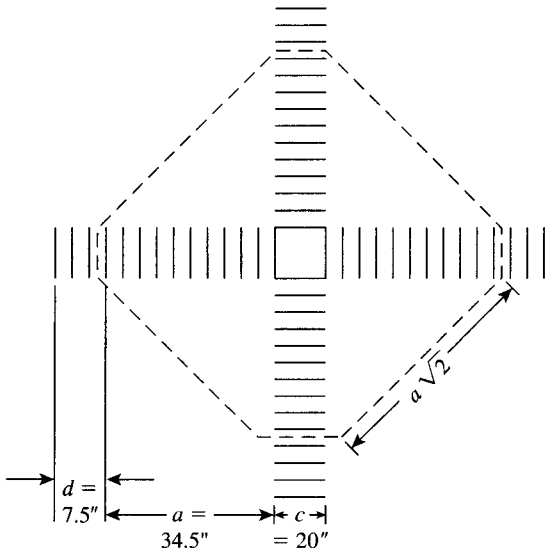


Figure 17.31 Example 17.10: Shear reinforcement no. 3 at 3.5 in.

3. Shear reinforcement may consist of reinforcing bars, structural steel sections such as I-beams, or special large-head studs welded to a steel strip. In this example, an inexpensive solution using normal shear reinforcement will be adopted. See Fig. 17.9*f*. Shear reinforcement must be provided on the four sides of the interior column (or three sides of an exterior column) for a distance of $d + a$. See Fig. 17.31. The distance a is determined by equating $\phi V_c = V_u$ at section b_o, indicated by the dashed line, and assuming $\phi V_c = \phi 2\lambda\sqrt{f'_c} b_o d$.

$$b_o = 4(c + \sqrt{2a}) = 4(20 + \sqrt{2a})$$

$$0.75(2)\sqrt{4000}(4)(20 + \sqrt{2a})(7.5) = 195{,}000 \text{ lb}$$

Here, $a = 34.3$ in., and $(a + d) = 34.3 + 7.5 = 41.8$ in., so use 42 in.

4. Calculate shear reinforcement:

$$\phi V_s = (V_u - \phi V_c) = 195 - 156.3 = 38.7 \qquad V_s = 51.6 \text{ K}$$

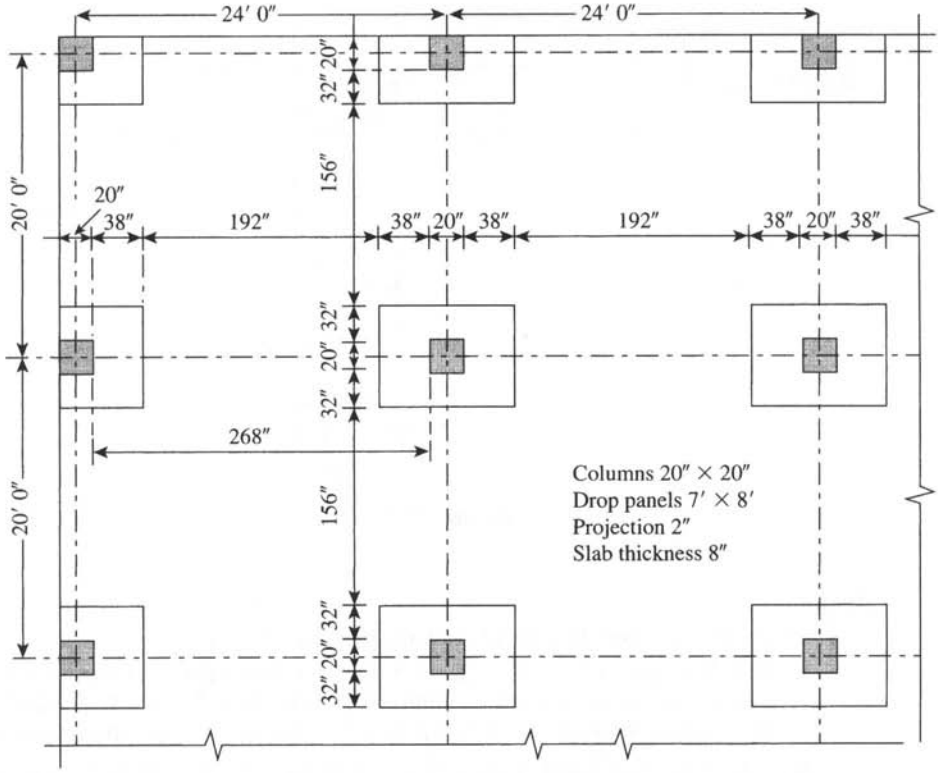
$$V_s(\text{for one face of critical section}) = \frac{V_s}{4} = \frac{51.6}{4} = 12.9 \text{ K}$$

Use no. 3 U-stirrups, $A_v = 0.22$ in.^{2} (for two legs). The spacing is $S = A_v f_y d/V_s = 0.22(60)(7.5)/12.9 = 7.7$ in. Maximum spacing is $d/2 = 7.5/2 = 3.75$ in.; let $s = 3.5$ in.

5. Distribution of stirrups: The number of stirrups per one side of column is 43/3.5 = 12.3, or 13 stirrups. Total distance is 13(3.5) = 45.5 in. (Fig. 17.31).

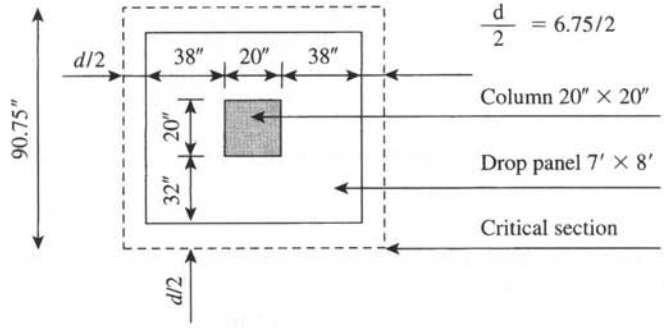
Example 17.11: Flat-Slab Floor System

Using the direct design method, design a typical 24 × 20-ft interior flat-slab panel with drop panels only (Fig. 17.32). All panels are supported by 20 × 20-in. columns, 12 ft long. The slab carries a uniform service live load of 100 psf and a service dead load of 24 psf, excluding self-weight. Use $f'_c = 4$ ksi, and $f_y = 60$ ksi. (The solution is similar to Example 17.3.)



Plan view of the flat slab

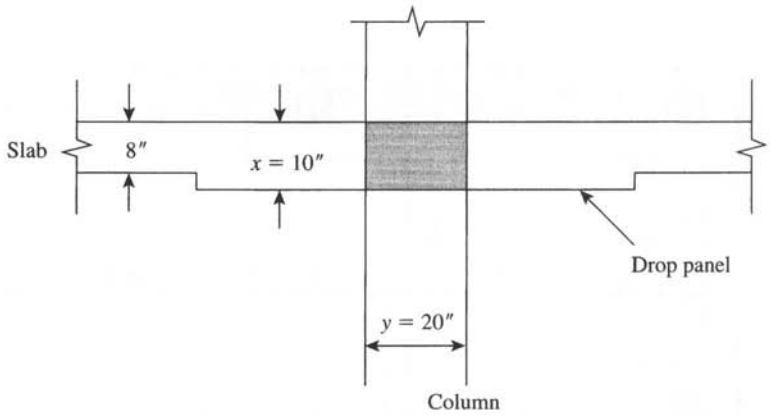
(a)



Critical section for
two-way shear in slab

(b)

Figure 17.32 Example 17.11: flat slab with drop panel.



(c)

Figure 17.32 (*continued*)

Solution

1. Determine slab and drop panel thicknesses using Table 17.1.
   - **a.** The clear span is $24 - \frac{20}{12} = 22.33$ ft. For an exterior panel, minimum $h = l_n/33 = 8.12$ in., whereas for an interior panel, minimum $h = l_n/36 = 7.44$ in. Use a slab thickness of 8 in. The projection below the slab is $h/4 = \frac{8}{4} = 2.0$ in.; thus, the drop panel thickness is 10 in.
   - **b.** Extend the drop panels $L/6 = \frac{24}{6} = 4$ ft in each direction from the centerline of support in the long direction and $\frac{20}{6} = 3.33$ ft, or 3.5 ft, in the short direction. Thus, the total size of one drop panel is 8 × 7 ft (Fig. 17.32).
2. Calculate factored loads:

$$\text{Slab load} = 24 + \frac{8(150)}{12} = 124 \text{ psf}$$

$$W_u = 1.2(124) + 1.6(80) = 277 \text{ psf}$$

$$\text{Drop panel load} = 24 + \frac{10(150)}{12} = 149 \text{ psf}$$

$$W_u = 1.2(149) + 1.6(80) = 307 \text{ psf}$$

   Because the drop panel length is $L/3$ in each direction, the average W_u is $\left(\frac{2}{3}\right)(277) + \left(\frac{1}{3}\right)(307) = 287$ psf.
3. Check two-way shear (at distance $d/2$ from the face of column):
   - **a.** In the drop panel: $d = 10 - 0.75 - 0.5 = 8.75$ in.

$$b_o = 4(20 + 8.75) = 115 \text{ in.}$$

$$V_u = 0.287\left[24 \times 20 - \left(\frac{28.75}{12}\right)^2\right] = 136.1 \text{ K}$$

$$\phi V_c = \phi 4\sqrt{f'_c} b_o d = 0.85(4)\sqrt{4000}(115)(8.75) = 214.4 \text{ K} > V_u$$

b. In the slab: $d = 8 - 0.75 - 0.5 = 6.75$ in. and b_o is measured at 6.175/2 in. (in slab) beyond the drop panel.

$$b_o = 2(8 \times 12 + 6.75) + 2(7 \times 12 + 6.75) = 387 \text{ in.}$$

$$V_u = 0.287[24 \times 20 - (102.75)(90.75)/144] = 119.2 \text{ K}$$

$$\psi V_c = 0.75(4)\sqrt{4000}(387)(6.75) = 495.6 \text{ K} > V_u$$

c. One-way shear is not critical.

4. Calculate the total static moments in the long and short directions:

$$M_{\text{ol}} = \frac{0.287(20)(22.33)^2}{8} = 357.8 \text{ K·ft}$$

$$M_{\text{os}} = \frac{0.287(24)(18.33)^2}{8} = 289.3 \text{ K·ft}$$

The width of column strip in each direction is $\frac{20}{2} = 10$ ft, whereas the width of the middle strip is 10 ft in the long direction and 14 ft in the short direction.

5. Calculations of moments and steel reinforcement are shown in Table 17.11. Use an average $d = 10 - 1.5 = 8.5$ in. in the column strip and $d = 8 - 1.5 = 6.5$ in. in the middle strip.

Bars are chosen for adequate distribution in both the column and middle strip. Reinforcement details are similar to those in flat-plate examples.

Table 17.11 Design of an Interior Flat-Slab Floor System

	Long Direction			
$M_{\text{ol}} = 358$ K·ft	**Column Strip**		**Middle Strip**	
M factor	$-0.49M_o$	$0.21M_o$	$-0.16M_o$	$0.14M_o$
M_u (K·ft)	−175.4	±75.2	−57.3	±50.1
Width of strip (in.), b	120	120	120	120
Effective depth (in.), d	8.5	6.5	6.5	6.5
$R_u = \frac{M_u}{bd^2}$ (psi)	243	178	129	119
Steel ratio ρ (%)	0.48	0.34	0.25	0.23
$A_s = \rho bd$ (in.^{2})	4.9	2.65	1.95	1.79
Min. $A_s = 0.0018bh_s$ (in.^{2})	2.16	2.16	1.73	1.73
Selected bars	16 no. 5	14 no. 4	10 no. 4	9 no. 4
	Short Direction			
$M_{\text{os}} = 289.3$ K·ft	**Column Strip**		**Middle Strip**	
M factor	$-0.49M_o$	$0.21M_o$	$-0.16M_o$	$0.14M_o$
M_u (K·ft)	−142	±60.8	−46.3	±40.5
Width of strip (in.), b	120	120	168	168
Effective depth (in.), d	8.5	6.5	6.5	6.5
$R_u = \frac{M_u}{bd^2}$ (psi)	196	144	78	68
Steel ratio ρ (%)	0.38	0.28	0.15	0.13
$A_s = \rho bd$ (in.^{2})	3.9	2.2	1.64	1.42
Min. $A_s = 0.0018bh_s$ (in.^{2})	2.16	2.16	2.42	2.42
Selected bars	13 no. 5	11 no. 4	12 no. 4	12 no. 4

17.11 WAFFLE SLABS

A two-way waffle slab system consists of concrete ribs that normally intersect at right angles. These slabs might be constructed without beams, in which case a solid column head is made over the column to prevent any punching due to shear. Wide beams can also be used on the column centerlines for uniform depth construction. Square metal or fiberglass pans are commonly used to form these joists. A thin slab of 3 to 5 in. is cast with these joists to form the waffle slab.

Each panel is divided into a column and a middle strip. The column strip includes all joists that frame into the solid head; the middle strip is located between consecutive column strips. Straight or bent bars could be used as a reinforcement in a waffle slab. The design of a two-way waffle slab is similar to that of flat slabs by considering the solid head as a drop panel. To prevent any excess in the diagonal tension in the head, a sufficient size of column must be used or a shear cap must be provided.

In the design of a waffle slab, the top slabs with each rib form a T-section, with considerable depth relative to flat plates. Consequently, long spans carrying heavy loads may be designed with great savings in concrete. Waffle slabs also provide an attractive ceiling, which is achieved by leaving the rib pattern or by integrating lighting fixtures. The standard pans that are commonly used in waffle slabs can be one of the following two types:

1. 30 × 30-in. square pans with a 3-in. top slab, from which 6-in.-wide ribs at 36 in. (3 ft) on centers are formed. These are available in standard depths of 8 to 20 in. in 2-in. increments. Refer to Example 17.12 and Fig. 17.33.
2. 19 × 19-in. square pans with a 3-in. top slab, from which 5-in.-wide ribs at 24 in. (2 ft) on centers are formed. These are available in standard depths of 4, 6, 8, 10, and 12 in. Other information about pans is shown in Table 17.12 [17].Other types, ranging from 19 × 19-in. pans to 40 × 40-in. pans, are available in the construction industry.

Example 17.12: Waffle Slab

Design a waffle floor system that consists of square panels without beams considering the following data (Fig. 17.33):

Span, center to center of columns = 33 ft

Width of rib = 6 in., spaced at 36 in. on centers

Depth of rib = 14in. and slab thickness = 3 in.

Column size = 20 × 20 in.

Dead load (excluding self-weight) = 50 psf

Live load = 100 psf $\quad f'_c = 5$ ksi $\quad f_y = 60$ ksi

Solution

1. Determine minimum slab thickness using Table 17.1: Minimum $h = l_n/30$, $l_n = 33 - \frac{20}{12} = 31.33$ ft, $h = 31.33(12)/30 = 12.5$ in. for exterior panels, and $h = l_n/33 = 11.4$ in. for interior

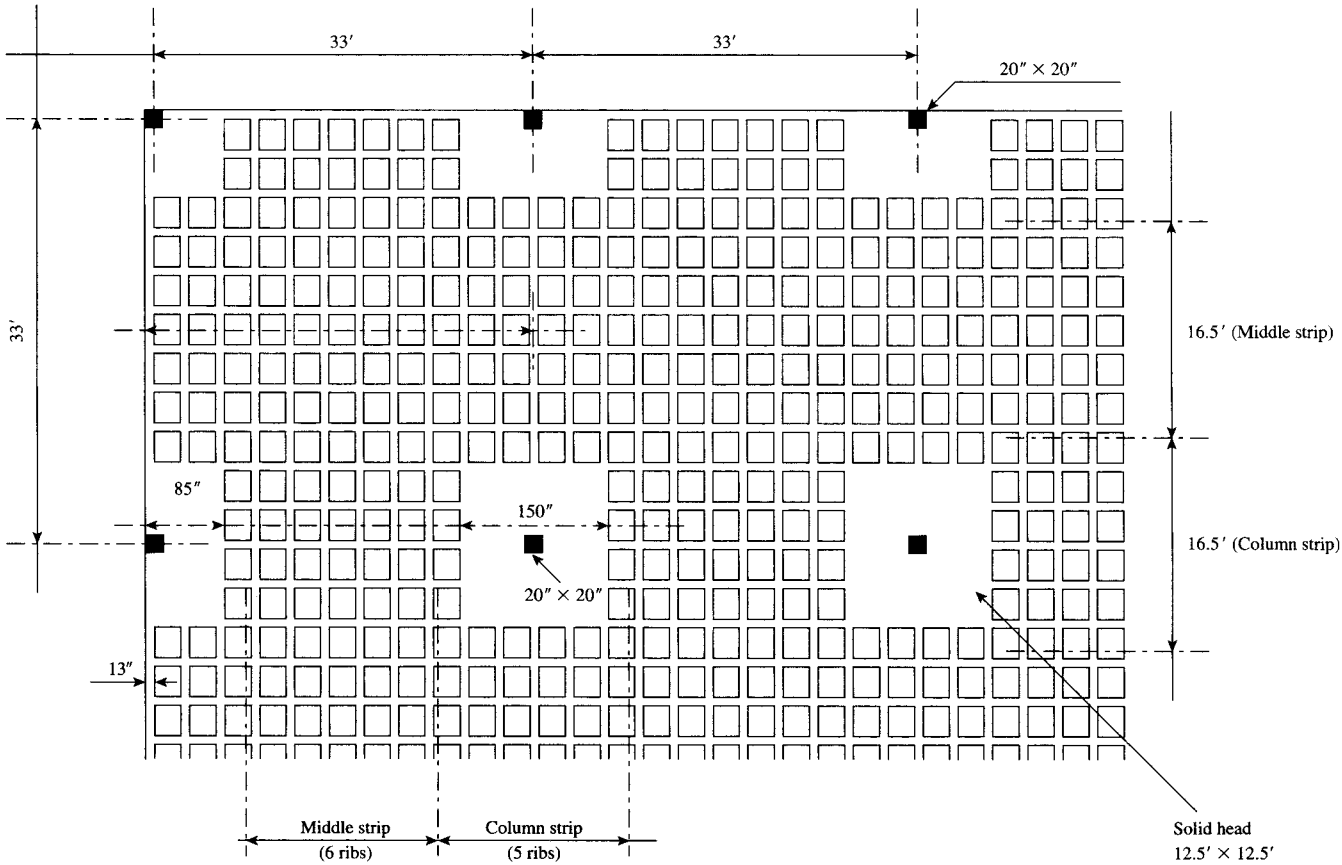


Figure 17.33 (*a*) Plan of the waffle slab, (*b*) cross section, (*c*) pan and rib dimensions, and (*d*) spacing and dimensions of solid heads (Example 17.12).

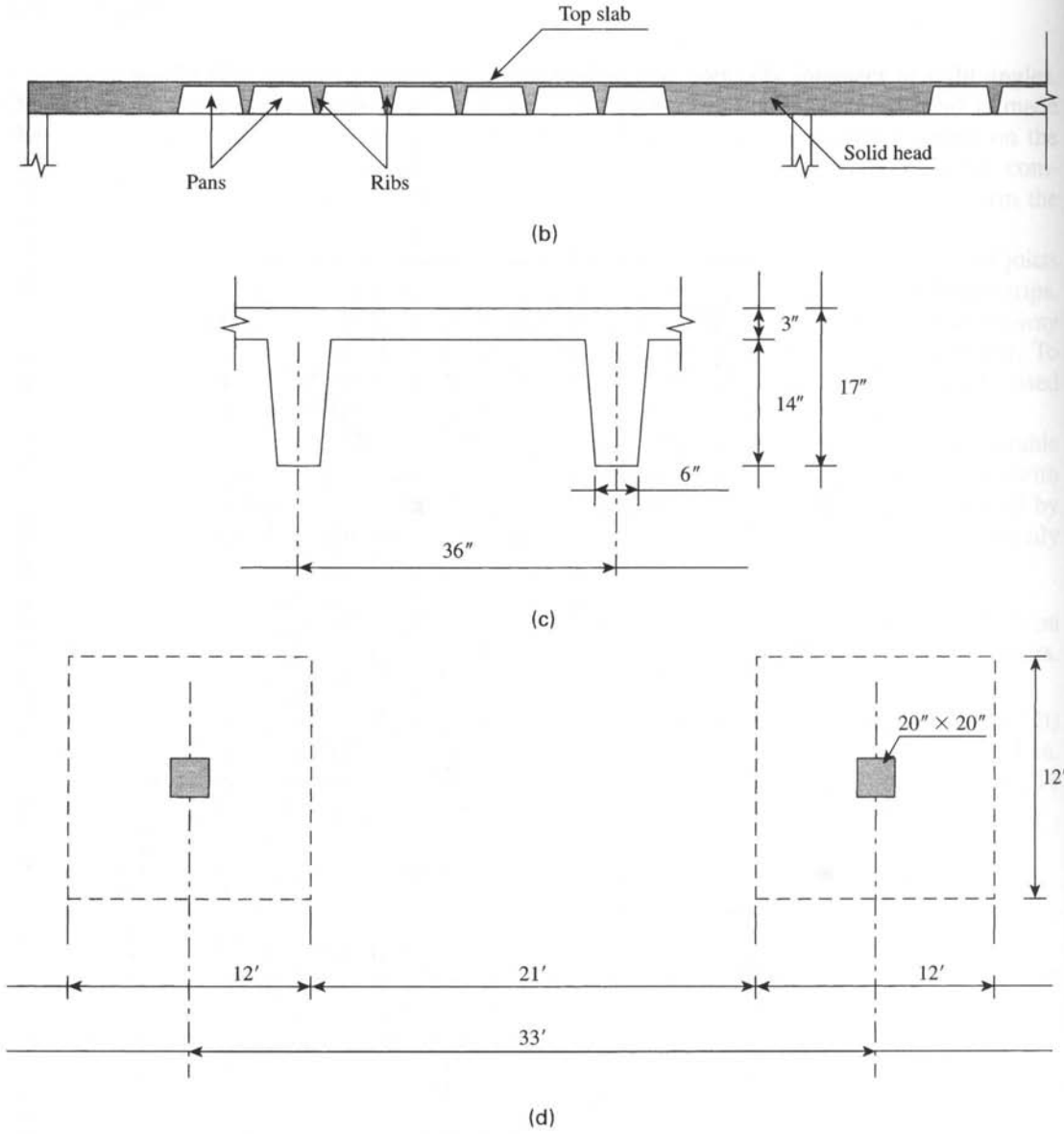


Figure 17.33 (*continued*)

panels. Equations 17.1 and 17.2 may be used. Assume the total depth is 17 in. consisting of 3-in. slab thickness and 14-in. rib depth.

2. Calculate loads on the waffle slab:
   - **a.** Factored load of solid head part = 1.2(150)(17/12) = 255 psf.
   - **b.** Voided volume of 14-in. rib = 6.54 ft^3 on 3 × 3-ft^2 area. Total weight of 9-ft^2 area is $1.2(150)(9 \times \frac{17}{12} - 6.54) = 1118$ lb. Weight per square foot is $\frac{1118}{9} = 125$ psf.

Table 17.12 Gross Section Properties [17]

Top Slab (in.)	Rib Depth (in.)	Volume (cf/pan)	Gross Area (in.^{2})	Y_{cg} (in.)	I_g (in.^{4})
For the Joists (30 x 30-in. pans)					
3	8	3.85	161.3	3.28	1393
3	10	4.78	176.3	3.95	2307
3	12	5.53	192	4.66	3541
3	14	6.54	208.3	5.42	5135
3	16	7.44	223.3	6.20	7127
3	20	9.16	261.3	7.83	12,469
4.5	8	3.85	215.3	3.77	2058
4.5	10	4.78	230.3	4.35	3227
4.5	12	5.53	246.0	4.97	4783
4.5	14	6.54	262.3	5.66	6773
4.5	16	7.44	279.3	6.36	9238
4.5	20	9.16	315.3	7.86	15,768
For the Joists (19 x 19-in. pans)					
3	6	1.09	105	2.886	598
3	8	1.41	117.4	3.564	1098
3	10	1.9	130.4	4.303	1824
3	12	2.14	144	5.083	2807
4.5	6	1.09	141	3.457	957
4.5	8	1.41	153	4.051	1618
4.5	10	1.9	166.4	4.709	2550
4.5	12	2.14	180	5.417	3794

c. Factored additional dead plus live load is $1.2(50) + 1.6(100) = 220$ psf. Uniform w_u (at solid head) $= 255 + 220 \approx 500$ psf. Uniform w_u (at ribbed area) $= 125 + 220 = 345$ psf.

d. Loads on one panel (refer to Fig. 17.34): At the solid head, $W = 0.5(12) + 0.345(21) = 13.22$ K/ft. At the ribbed area, $W = 0.345(33) = 11.39$ K/ft.

3. Calculate shear and total static moment:

$$V_u \text{ (at the face of column)} = 13.22(5.17) + \frac{(11.39)(21)}{2} = 188 \text{ K}$$

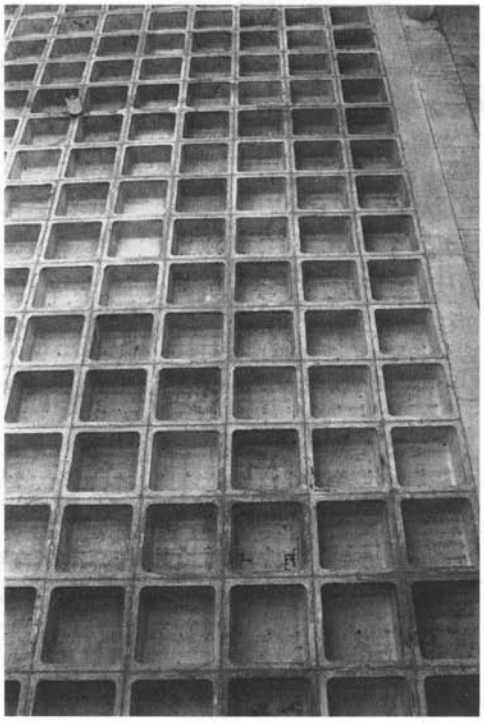
$$M_o \text{ (at midspan)} = 188(15.67) - 13.22(5.17)(13.09) - \frac{11.39(10.5)^2}{2} = 1424 \text{ K-ft}$$

4. Check punching shear (refer to Fig. 17.35):

a. In solid head at $d/2$ from column face, $h = 17$ in., $d = 17 - 1.25 = 15.75$ in., c (column) $= 20$ in., $b_o = 4(20 + 15.75) = 143$ in., $V_u = 11.39(21 \text{ ft}) + 13.22(12 \text{ ft}) - 0.5(37.75/12)^2 = 393.4$ K and $\phi V_c = \phi 4\sqrt{f'_c} b_o d = 0.75(4)(\sqrt{5000})(143)(15.75) = 478 \text{ K} > V_u$.

b. In the slab at distance $d/2$ from the edge of the solid head, slab thickness is 3 in.; let $d = 2.15$ in. Then

$$b_o = 4(150 + 2.5) = 610 \text{ in.}$$



Waffle slab (looking upward).

$$V_U = 11.39(21) + 13.22(12) - 0.5\left(\frac{152.5}{12}\right)^2 = 317.4 \text{ K}$$

$$\phi V_c = 0.75(4)(\sqrt{5000})(610)(2.5) = 324 \text{ K} > V_u$$

5. Design moments and reinforcement:
   a. Exterior panel: $M_o = 1424$ K·ft

$$\text{Exterior negative moment} = 0.26M_o = -370 \text{ K·ft}$$

$$\text{Positive moment} = 0.52M_o = +740 \text{ K·ft}$$

$$\text{Interior negative moment} = 0.7M_o = -997 \text{ K·ft}$$

   b. Interior panel: $M_o = 1424$ K·ft

$$\text{Negative moment} = 0.65(1424) = -925.6 \text{ K·ft}$$

$$\text{Positive moment} = 0.35(1424) = 498.4 \text{ K·ft}$$

   Design details are shown in Table 17.13 and Fig. 17.36. Note that all steel ratios are low and $\phi = 0.9$.

6. Calculate the unbalanced moments in columns and check shear for V_u and M_v, as in Examples 17.8 and 17.9.

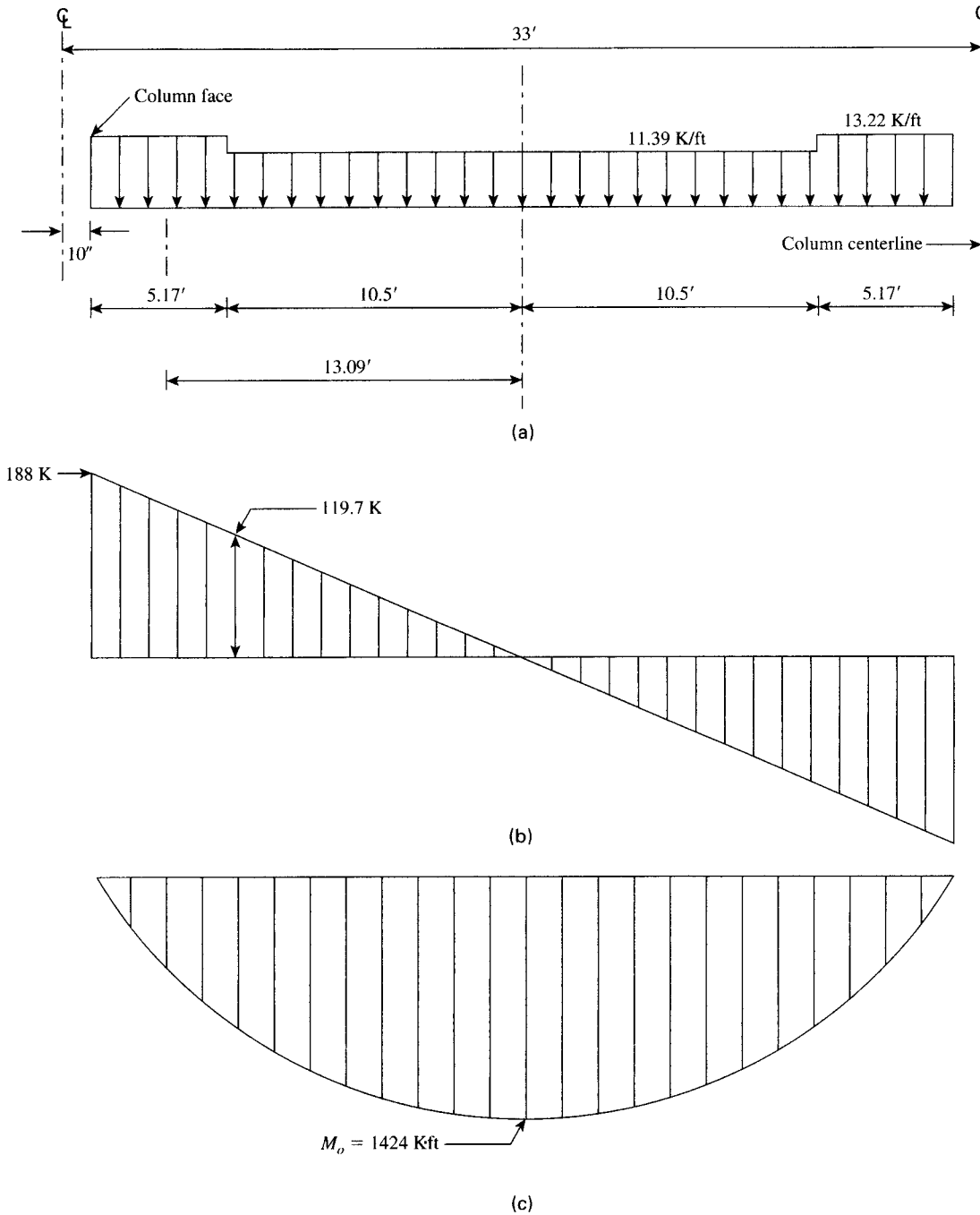


Figure 17.34 Load, shear, and moment diagrams: (*a*) load distribution on the span, (*b*) shear force diagram, and (*c*) bending moment diagram.

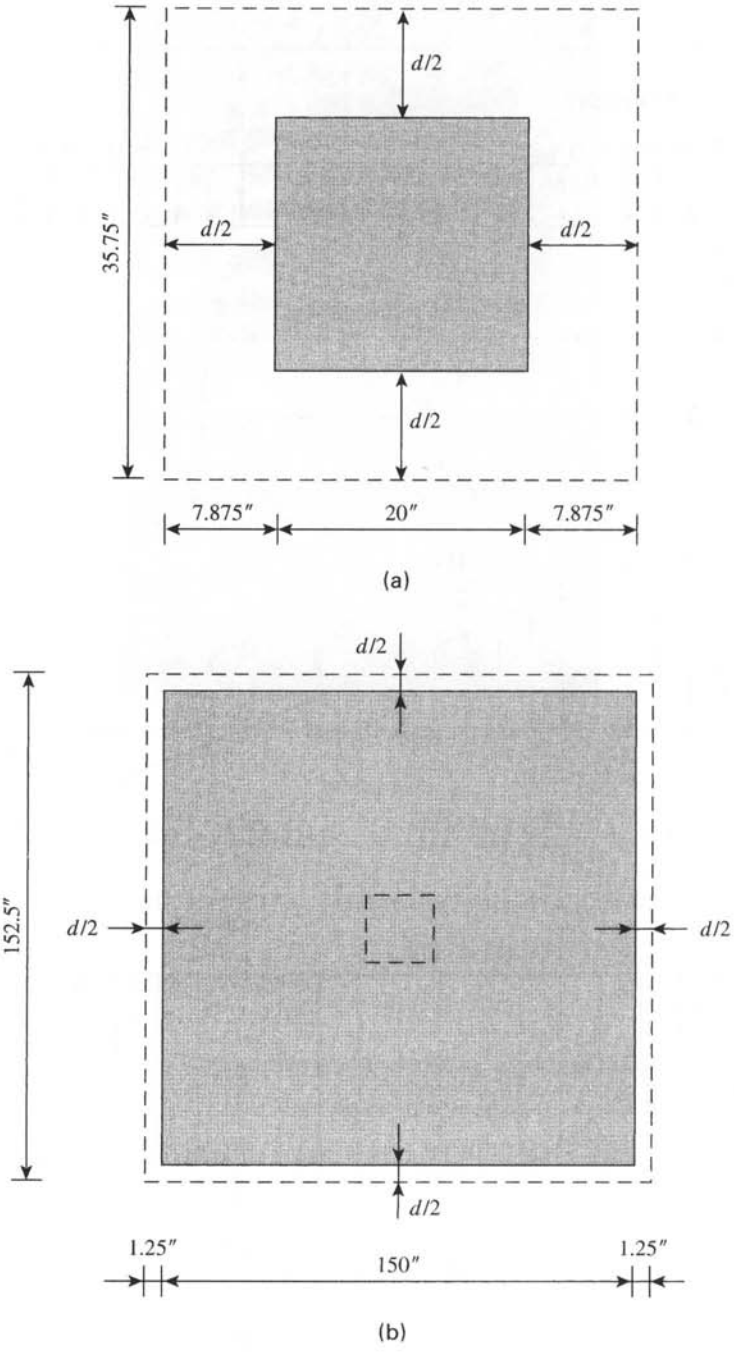


Figure 17.35 Punching shear locations: (*a*) punching shear in column head and (*b*) punching shear in slab.

Table 17.13 Design of an Exterior and an Interior Waffle Slab (5 Ribs in Column Strip and 6 Ribs in Middle Strips)

	Column Strip			Middle Strip	
Exterior Panel	**Exterior (−M)**	**±M**	**Interior −M**	**−M**	**±M**
Moment factor (%)	100	60	75	25	40
M_u (K·ft)	370	444	748	249	296
Strip width, b (in.)	150	198	150	36 (6 ribs)	198
d (in.)	15.75	15.75	15.75	15.75	15.75
$R_u = \frac{M_u}{bd^2}$ (psi)	120	108	241	334	72
Steel ratio, ρ (%)	0.226	0.204	0.465	0.657	0.135
$A_s = \rho bd$ (in.^{2})	5.33	6.36	11.0	3.73	4.2
Min. $A_s = 0.0018bh$	2.6	1.22	4.6	1.1	1.47
Bars selected	14 no. 6	2 no. 8/rib	26 no. 6	10 no. 6	2. no. 7/rib

	Column Strip			Middle Strip	
Exterior Panel	**Exterior (−M)**	**±M**	**Interior −M**	**−M**	**±M**
Moment factor (%)	—	60	75	25	40
M_u (K·ft)	—	299	694.2	231.4	200
Strip width, b (in.)	—	198	150	36 (6 ribs)	198
d (in.)	—	15.75	15.75	15.75	15.75
$R_u = \frac{M_u}{bd^2}$ (psi)	—	73	224	311	49
Steel ratio, ρ (%)	—	0.137	0.431	0.61	0.091
$A_s = \rho bd$ (in.^{2})	—	4.27	10.18	3.45	2.84
Min. $A_s = 0.10018bh$	—	1.22/rib	4.6	1.1	1.47
Bars selected	—	2 no. 7/rib	24 no. 6	10 no. 6	2 no. 6/rib

17.12 EQUIVALENT FRAME METHOD

When two-way floor systems do not satisfy the limitations of the direct design method, the design moments must be computed by the equivalent frame method. In the latter method, the building is divided into equivalent frames in two directions and then analyzed elastically for all conditions of loadings. The difference between the direct design and equivalent frame methods lies in the way by which the longitudinal moments along the spans of the equivalent rigid frame are determined. The design requirements can be explained as follows.

1. Description of the equivalent frame: An equivalent frame is a two-dimensional building frame obtained by cutting the three-dimensional building along lines midway between columns (Fig. 17.4). The resulting equivalent frames are considered separately in the longitudinal and transverse directions of the building. For vertical loads, each floor is analyzed separately, with the far ends of the upper and lower columns assumed to be fixed. The slab-beam may be assumed to be fixed at any support two panels away from the support considered, because the vertical loads contribute very little to the moment at that support. For lateral loads, the equivalent frame consists of all the floors and extends for the full

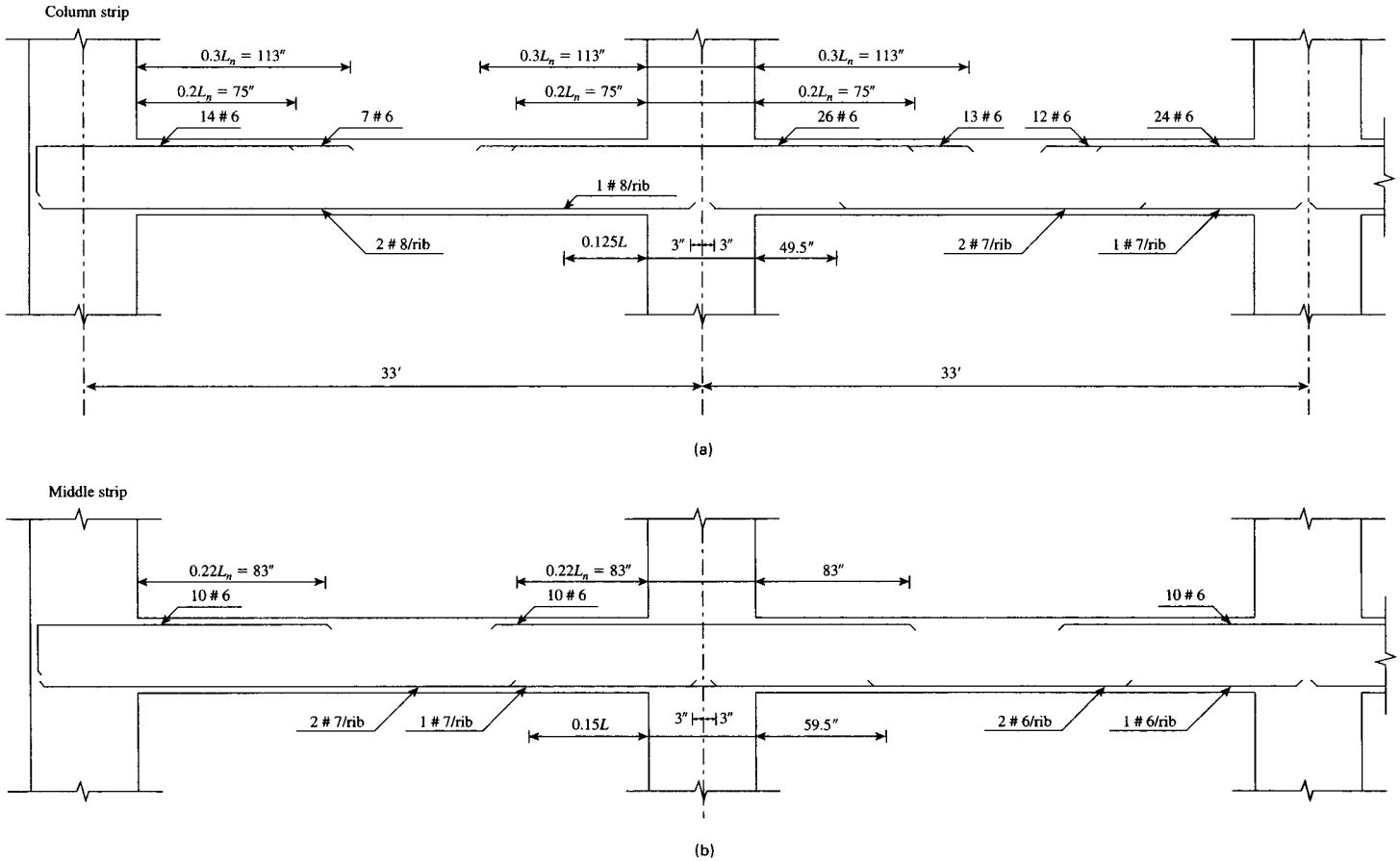


Figure 17.36 Example 17.12: reinforcement details of the waffle slab.

height of the building, because the forces at each floor are a function of the lateral forces on all floors above the considered level. Analysis of frames can also be made using computer programs.

2. Load assumptions: When the ratio of the service live load to the service dead load is less than or equal to 0.75, the structural analysis of the frame can be made with the factored dead and live loads acting on all spans instead of a pattern loading. When the ratio of the service live load to the service dead load is greater than 0.75, pattern loading must be used, considering the following conditions:
   a. Only 75% of the full-factored live load may be used for the pattern loading analysis.
   b. The maximum negative bending moment in the slab at the support is obtained by loading only the two adjacent spans.
   c. The maximum positive moment near a midspan is obtained by loading only alternate spans.
   d. The design moments must not be less than those occurring with a full-factored live load on all panels (ACI Code, Section 13.7.6).
   e. The critical negative moments are considered to be acting at the face of a rectangular column or at the face of the equivalent square column having the same area for nonrectangular sections.
3. Slab-beam moment of inertia: The ACI Code specifies that the variation in moment of inertia along the longitudinal axes of the columns and slab beams must be taken into account in the analysis of frames. The critical region is located between the centerline of the column and the face of the column, bracket, or capital. This region may be considered as a thickened section of the floor slab. To account for the large depth of the column and its reduced effective width in contact with the slab beam, the ACI Code, Section 13.7.3.3, specifies that the moment of inertia of the slab beam between the center of the column and the face of the support is to be assumed equal to that of the slab beam at the face of the column divided by the quantity $(1 - c_2/l_2)^2$, where c_2 is the column width in the transverse direction and l_2 is the width of the slab beam. The area of the gross section can be used to calculate the moment of inertia of the slab beam.
4. Column moment of inertia: The ACI Code, Section 13.7.4, states that the moment of inertia of the column is to be assumed infinite from the top of the slab to the bottom of the column capital or slab beams (Fig. 17.37).
5. Column stiffness, K_{ec}, is defined by

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{K_t} \tag{17.17}$$

where ΣK_c is the sum of the stiffness of the upper and lower columns at their ends,

$$K_t = \sum \frac{9E_{es}C}{l_2\left(1 - \frac{c_2}{l_2}\right)} \tag{17.19}$$

$$C = \sum \left(1 - 0.63\frac{x}{y}\right)\left(\frac{x^3 y}{3}\right) \tag{17.20}$$

6. Column moments: In frame analysis, moments determined for the equivalent columns at the upper end of the column below the slab and at the lower end of the column above the slab must be used in the design of a column.

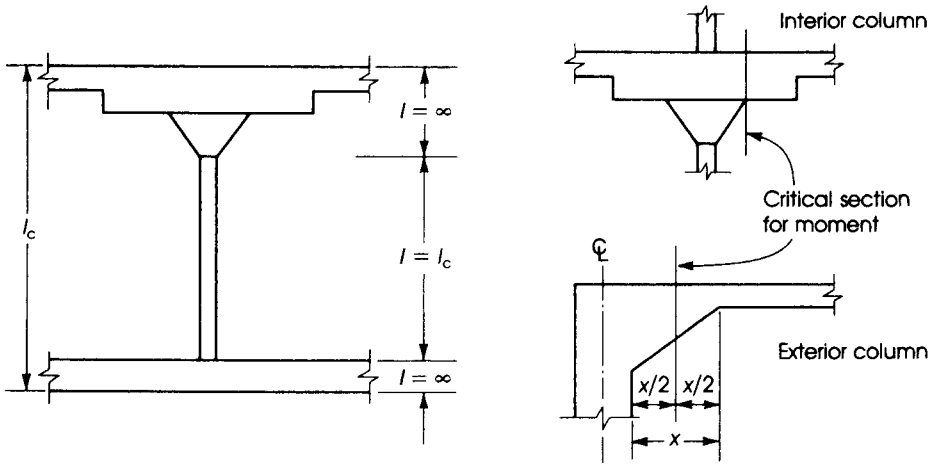


Figure 17.37 Critical sections for column moment, equivalent frame method.

7. Negative moments at the supports: The ACI Code, Section 13.7.7, states that for an interior column, the factored negative moment is to be taken at the face of the column or capital but at a distance not greater than $0.1175l_1$ from the center of the column. For an exterior column, the factored negative moment is to be taken at a section located at half the distance between the face of the column and the edge of the support. Circular section columns must be treated as square columns with the same area.
8. Sum of moments: A two-way slab floor system that satisfied the limitations of the direct design method can also be analyzed by the equivalent frame method. To ensure that both methods will produce similar results, the ACI Code, Section 13.7.7, states that the computed moments determined by the equivalent frame method may be reduced in such proportion that the numerical sum of the positive and average negative moments used in the design must not exceed the total statical moment, M_o.

Example 17.13: Waffle Slab

By the equivalent frame method, analyze a typical interior frame of the flat-plate floor system given in Example 17.3 in the longitudinal direction only. The floor system consists of four panels in each direction with a panel size of 25 by 20 ft. All panels are supported by a 20- by 20-in. columns, 12 ft long. The service live load is 80 psf and the service dead load is 124 psf (including the weight of the slab). Use $f'_c = 3$ ksi and $f_y = 60$ ksi. Edge beams are not used. Refer to Fig. 17.38.

Solution

1. A slab thickness of 8.0 in. is chosen, as explained in Example 17.3.
2. Factored load is $w_u = 1.2 \times 124 + 1.6 \times 60 = 245$ psf. The ratio of service live load to service dead load is $60/124 = 0.48 < 0.75$; therefore, the frame can be analyzed with the full factored load, w_u, acting on all spans instead of pattern loading.
3. Determine the slab stiffness, K_s:

$$K_s = k\frac{EI_s}{l_s}$$

where k is the stiffness factor and

$$I_s = \frac{l_2 h_s^3}{12} = \frac{(20 \times 12)}{12}(8)^3 = 10{,}240 \text{ in.}^4$$

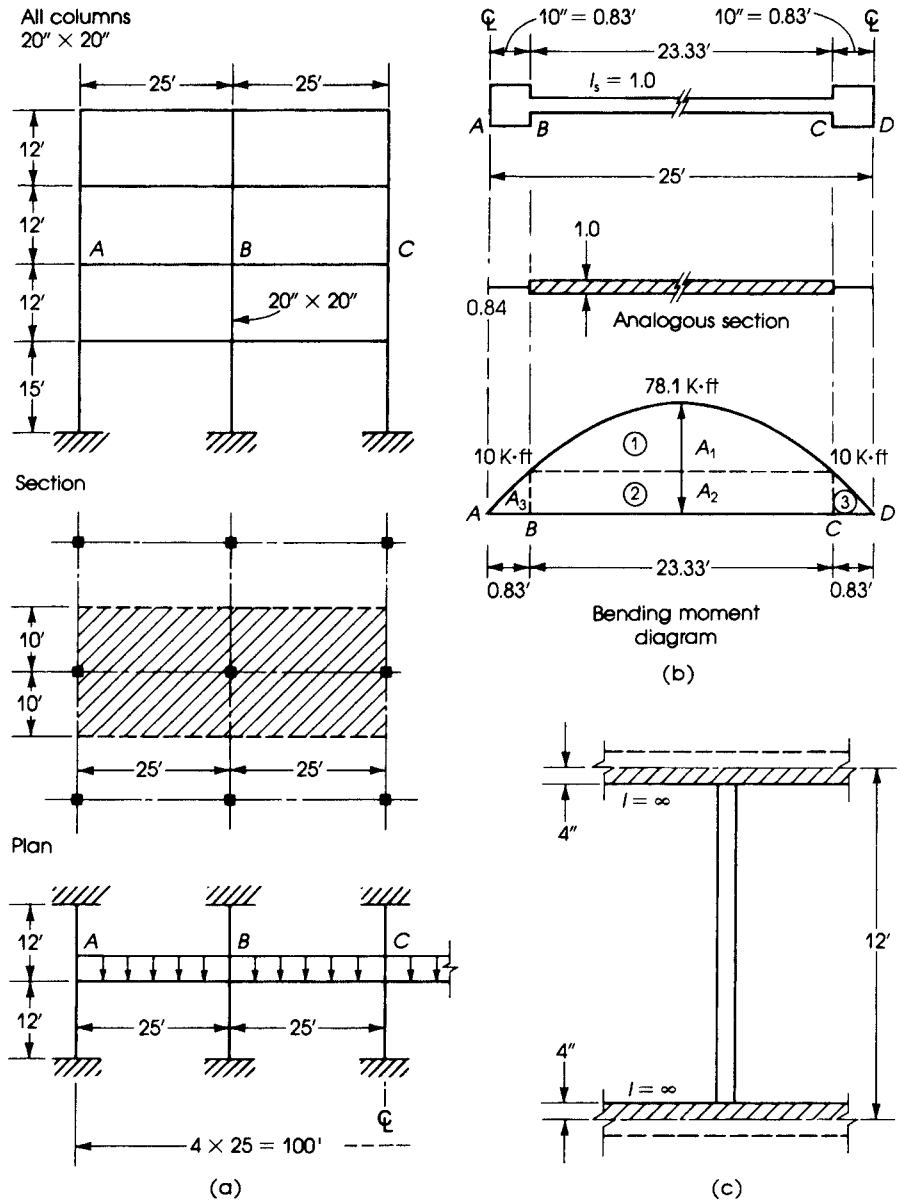


Figure 17.38 Example 17.13.

The stiffness factor can be determined by the column analogy method described in books on structural analysis. Considering the moment of inertia for the slab I_s to be 1.0 as a reference, the moment of inertia between the column centerline and the face of the column is

$$\frac{1.0}{\left(1-\frac{c_2}{l_2}\right)^2}=\frac{1.0}{\left(1-\frac{20}{(20\times 12)}\right)^2}=1.19$$

The width of the analogous column varies with $1/I$, as shown in Fig. 17.38*b*: $(1/1.19) = 0.84$

$$\text{Slab stiffness factor } k = l_1 \left(\frac{1}{A_a} + \frac{Mc}{I_a}\right)$$

where

A_a = area of the analogous column section

I_a = moment of inertia of analogous column

M = moment due to a unit load at the extreme fiber of the analogous column located at the center of the slab

$M = 1.0 \times \frac{l_1}{2}$

$A_a = 23.33 + 2 \times (0.83 \text{ ft})(0.84) = 23.33 + 1.40 = 24.72$

$I_a = I$ (for slab portion of 23.33) + I (of end portion) about the centerline

$$I_a = \frac{(23.33)^3}{12} + 1.4\left(12.5 - \frac{0.83}{2}\right)^2 = 1263$$

neglecting the moment of inertia of the short end segments about their own centroid.

$$\text{Stiffness factor } k = 25\left[\frac{1}{24.72} + \frac{1.0 \times 12.5(12.5)}{1263}\right]$$

$$= 1.01 + 3.09 = 4.1$$

$$\text{Carryover factor} = \frac{3.09 - 1.01}{4.1} = 0.509$$

Therefore, slab stiffness is

$$K_s = \frac{4.1E \times 10{,}240}{(25 \times 12)} = 140E$$

4. Determine the column stiffness, K_c:

$$K_c = k'\left(\frac{EI_c}{l_c}\right) \times 2$$

for columns above and below the slab.

k' = column stiffness factor

$l_c = 12$ ft $\qquad I_c = \dfrac{(20)^4}{12} = 13{,}333 \text{ in.}^4$

The stiffness factor, k', can be determined as follows:

$$k' = l_c\left(\frac{1}{A_a} + \frac{Mc}{I_a}\right)$$

For the column, $c = l_c/2$ and $M = 1.0(l_c/2) = l_c/2$.

$$A_a = l_c - h_s = 12 - \frac{8}{12} = 11.33$$

$$I_a = \frac{(l_c - h_s)^3}{12} = \frac{(11.33)^3}{12} = 121.2$$

$$k' = 12\left[\frac{1}{11.33} + \frac{(1 \times \frac{12}{2})(\frac{12}{2})}{121.2}\right] = 4.62$$

$$K_c = 4.62E \times \frac{13{,}333}{12 \times 12} \times 2 = 856E$$

In a flat-plate floor system, the column stiffness, K_c can be calculated directly as follows:

$$\frac{K_c}{E_c} = \frac{I_c}{(l_c - h_s)} + \frac{3I_c l_c^2}{(l_c - h_s)^3} \tag{17.33}$$

5. Calculate the torsional stiffness, K_t, of the slab at the side of the column:

$$K_t = \frac{\sum 9R_{cs}C}{l_2\left(1 - \frac{c_2}{l_2}\right)} \quad \text{and} \quad C = \sum \left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$$

In this example, $x = 8.0$ in. (slab thickness) and $y = 20$ in. (column width). See Fig. 17.17.

$$c = \left(1 - 0.63 \times \frac{8}{20}\right)\left(\frac{(8)^3 \times 20}{3}\right) = 2553 \text{ in.}^4$$

$$K_t = \frac{9E_{cs} \times 2553}{(20 \times 12)\left(1 - \frac{20}{(20-12)}\right)^3} = 124\,E_c$$

For two adjacent slabs, $K_t = 2 \times 124E_c = 248E_c$.

6. Calculate the equivalent column stiffness, K_{ec}:

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{K_t} = \frac{1}{856E_c} + \frac{1}{248E_c}$$

or $K_{ec} = 192E_c$.

7. Moment distribution factors (D.F.): For the exterior joint,

$$\text{D.F. (slab)} = \frac{K_s}{K_s + K_{ec}} = \frac{140}{140 + 192} = 0.42$$

$$\text{D.F. (columns)} = \frac{K_{ec}}{\sum K} = 0.58$$

The columns above and below the slab have the same stiffness; therefore, the distribution factor of 0.58 is divided equally between both columns, and each takes a D.F. of 0.58/2 = 0.29. For the interior joint,

$$\text{D.F. (slab)} = \frac{K_s}{2K_s + K_{ec}} = \frac{140}{2 \times 140 + 192} = 0.295$$

$$\text{D.F. (columns)} = \frac{K_{ec}}{\sum K} = \frac{192}{2 \times 140 + 192} = 0.41$$

Each column will have a D.F. of 0.41/2 = 0.205.

8. Fixed-end moments: Because the actual L.L./D.L. is less than 0.75, the full-factored load is assumed to act on all spans.

$$\text{Fixed-end moment} = k'' w_u l_2 (L_1)^2$$

The factor k'' can be determined by the column analogy method: For a unit load $w = 1.0$ K/ft over the longitudinal span of 25 ft, the simple moment diagram is shown in Fig. 17.38*b*. The area of the bending moment diagram, considering the variation of the moment of inertia along

the span, is

$$\text{Total area } (A_m) = A_1 + A_2 + 2A_3$$
$$= \frac{2}{3} \times 23.33(78.1 - 10) + 23.33 \times 10$$
$$+2\left(\frac{1}{2} \times 0.83 \times 10\right)(0.84) = 1300$$

$$\text{Fixed-end moment coefficient} = \frac{A_m}{A_a l_1^2}$$

where A_a for the slab is 24.72, as calculated in step 3.

$$k'' = \frac{1300}{27.32(25)^2} = 0.084$$

It can be seen that the fixed-end moment coefficient, $k'' = 0.084$, is very close to the coefficient $\frac{1}{12} = 0.0833$ usually used to calculate the fixed-end moments in beams. This is expected, because the part of the span that has a variable moment of inertia is very small in flat plates where no column capital or drop panels are used. In this example, only parts AB and CD, each equal to 0.83 ft, have a higher moment of inertia than I_s. In flat plates where the ratio of the span to column width is high, say, at least 20, the coefficient 0.0833 may be used to calculate approximately the fixed-end moments. Fixed-end moment (due to $w_u = 276$ psf) = $0.084(0.245)(20)(25)^2 = 256$ K·ft. The factors K, K'_s, and K'' can be obtained from tables prepared by Simmonds and Misic [18] to meet the ACI requirements for the equivalent frame method.

9. Moment distribution can be performed on half the frame due to symmetry. Once the end negative moments are computed, the positive moments at the center of any span can be obtained by subtracting the average value of the negative end moments from the simple beam positive moment. The moment distribution is shown in Fig. 17.39. The final bending moments and shear forces are shown in Fig. 17.40.
10. Slabs can be designed for the negative moments at the face of the columns as shown in Fig. 17.40.

Example 17.14: SI Units

Use the direct design method to design a typical interior flat slab with drop panels to carry a dead load of 8.6 kN/m^2 and a live load of 11 kN/m^2. The floor system consists of six panels in each direction, with a panel size of 6.0 by 5.4 m. All panels are supported by 0.4-m-diameter columns with 1.0-m-diameter column capitals. The story height is 3.0 m. Use $f'_c = 28$ MPa and $f_y = 400$ MPa.

Solution

1. All the ACI limitations to using the direct design method are met. Determine the minimum slab thickness, h_s, using Eqs. 17.1 and 17.2. The diameter of the column capital equals 1.0 m. The equivalent square column section of the same area will have a side of $\sqrt{\pi r^2} = \sqrt{\pi(500)^2} =$ 885 mm or 900 mm.

$$\text{Clear span (long direction)} = 6.0 - 0.19 = 5.1 \text{ m}$$

$$\text{Clear span (short direction)} = 5.4 - 0.9 = 4.5 \text{ m}$$

Because no beams are used $\alpha_{fm} = 0$, $\beta_s = 1.0$, and $\beta = 6.0$ m/5.4 m = 1.11. From Table 17.1, minimum $h_s = l_n/33 = 5100/33 = 155$ mm, but because a drop panel is used, h_s may be reduced by 10% if drop panels extend a distance of at least $l/6$ in each direction from the

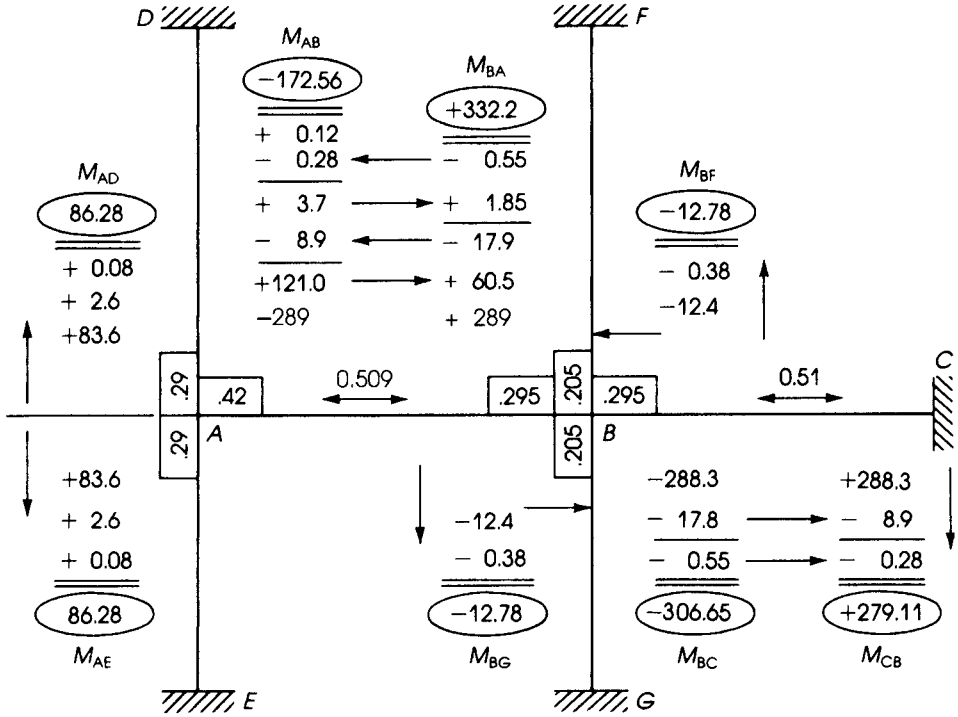


Figure 17.39 Example 17.13: Analysis by moment distribution. All moments are in K·ft.

centerline of support and project below the slab a distance of at least $h_s/4$. Therefore, use a slab thickness $h_s = 0.9 \times 155 \times 140$ mm and a drop panel length and width as follows:

$$\text{Long direction } \frac{l_1}{3} = \frac{6.0}{3} = 2.0 \text{ m}$$

$$\text{Short direction } \frac{l_2}{3} = \frac{5.4}{3} = 1.8 \text{ m}$$

The thickness of the drop panel is $1.25h_s = 1.25 \times 140 = 175$ mm. Increase drop panel thickness to 220 mm to provide adequate thickness for punching shear and to avoid the use of a high percentage of steel reinforcement. All dimensions are shown in Fig. 17.41.

2. Calculate factored loads:

$$w_u = 1.2 \times 8.6 + 1.6 \times 11 = 28 \text{ kN/m}^2$$

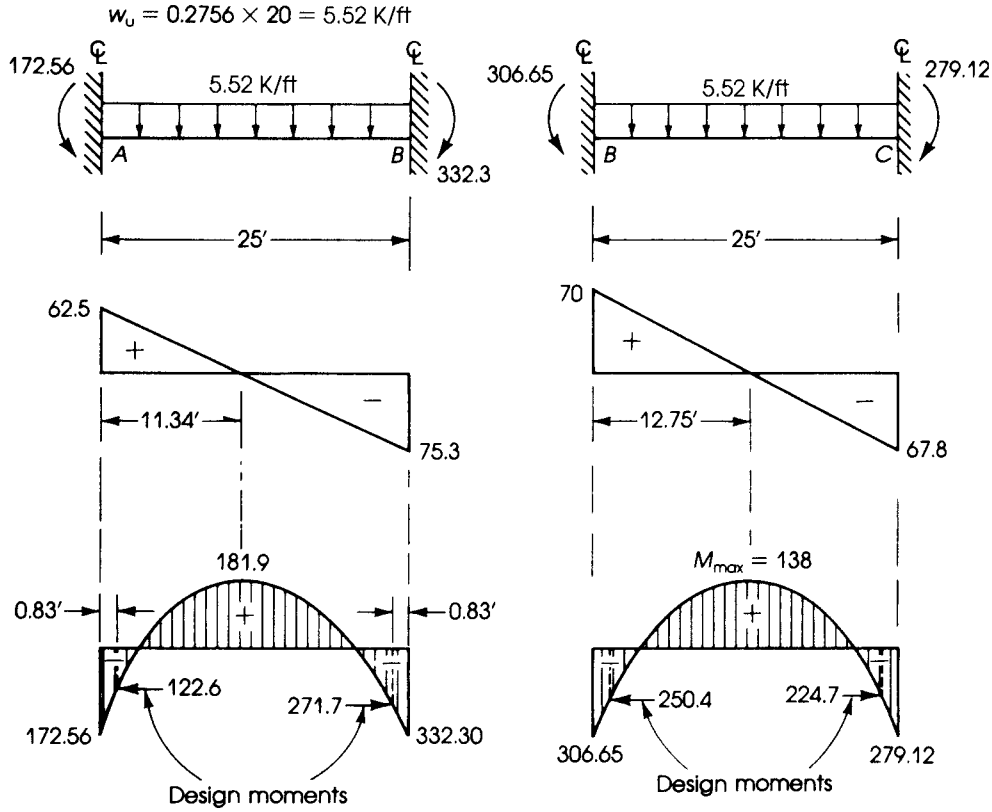
3. Check two-way shear, first in the drop panel: The critical section is at a distance $d/2$ around the column capital. Let $d = 220 - 30$ mm $= 190$ mm. Diameter of shear section $= 1.0$ m $+$ $d = 1.19$ m

$$V_u = 28\left[6.0 \times 5.4 - \frac{\pi}{4}(1.19)^2\right] = 876 \text{ kN}$$

$$b_o = 2\pi\left(\frac{1.19}{2}\right) = 3.74 \text{ m}$$

$$\phi V_c = \phi \times 0.33 \times \sqrt{f'_c} b_o d$$

$$= \frac{0.75 \times 0.33}{1000}\sqrt{28} \times 3740 \times 190 = 930 \text{ kN}$$



$$V_A = 5.52 \times 12.5 - \tfrac{1}{25}(332.3 - 172.56) = 62.5 \text{ K}$$

$$V_B \text{ (left)} = 5.52 \times 12.5 + \tfrac{1}{25}(332.3 - 172.56) = 75.3 \text{ K}$$

$$V_B \text{ (right)} = 5.52 \times 12.5 + \tfrac{1}{25}(306.65 - 279.12) = 70 \text{ K}$$

$$V_C = 5.52 \times 12.5 - \tfrac{1}{25}(306.65 - 279.12) = 67.8 \text{ K}$$

Figure 17.40 Example 17.13: equivalent frame method — final bending moments and shear forces. (Slabs can be designed for the negative moments at the face of the columns as shown.)

which is greater than V_u of 876 kN. Then check the two-way shear in the slab; the critical section is at a distance $d/2$ outside the drop panel.

$$d(\text{slab}) = 140 - 30 = 110 \text{ mm}$$

$$\text{Critical area} = (2.0 + 0.11)(1.8 + 0.11) = 4.03 \text{ m}^2$$

$$b_o = 2(2.11 + 1.91) = 8.04 \text{ m}$$

$$V_u = 28(6 \times 5.4 - 4.03) = 794 \text{ kN}$$

$$\phi V_c = \frac{0.75 \times 0.33}{1000}\sqrt{21} \times 8040 \times 110 = 1003 \text{ kN} > V_u$$

One-way shear is not critical.

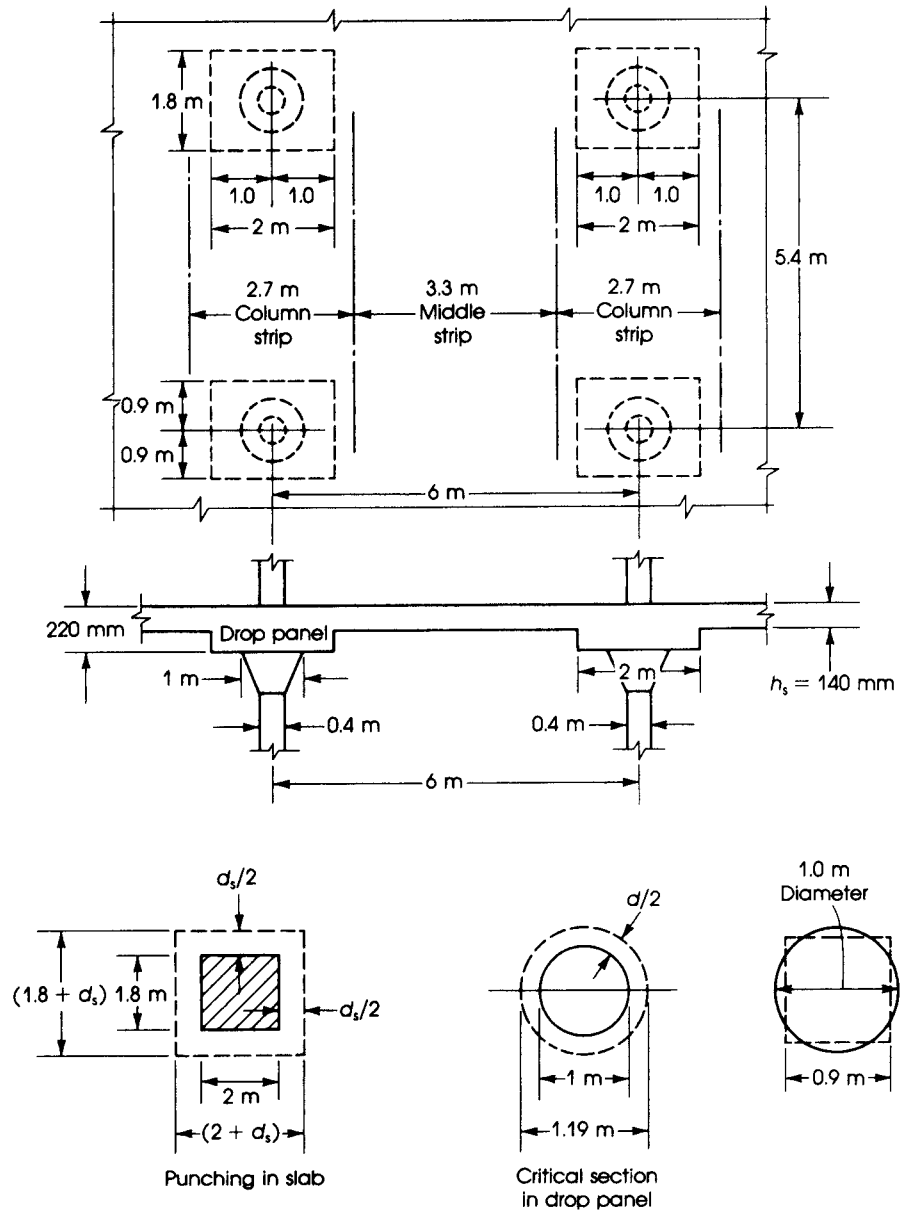


Figure 17.41 Example 17.14: interior flat slab with drop panel.

4. Calculate the total static moments in the long and short directions:

$$M_{ol} = \frac{w_u}{8} l_2 l_{n1}^2 = \frac{28}{8}(5.4)(5.1)^2 = 491.6 \text{ kN·m}$$

$$M_{os} = \frac{w_u}{8} l_1 l_{n2}^2 = \frac{28}{8}(6)(4.5)^2 = 425.2 \text{ kN·m}$$

Table 17.14 Design of an Interior Flab Slab With Drop Panels

	$M_o = 491.6$ kN·m $M_o = +0.35M_o = -319.5$ kN·m $M_p = +0.35M_o = +172.1$ kN·m			
Long Direction	**Column Strip**		**Middle Strip**	
Moment factor	$0.75M_o$	$0.60M_p$	$0.25M_n$	$0.40M_p$
M_u (kN·m)	− 239.6	± 103.3	−79.9	± 68.8
d (mm)	190	110	110	110
Strip width b (m)	2.7	2.7	2.7	2.7
$R_u = \frac{M_u}{bd^2}$ (MPa)	2.46	3.16	2.44	2.10
Steel ratio, ρ (%)	0.71	0.93	0.7	0.6
$A_s = \rho bd$ (mm^2)	3642	2762	2079	1782
Min. $A_s = 0.0018$bh (mm^2)	1070	680	680	680
Bars selcted (straight bars)	18 × 16 mm	14 × 16 mm	20 × 12 mm	16 × 12 mm
Spacing (mm)	150	193	135	170
	$M_o = 425.2$ kN·m $M_n = -0.65M_o = -276.4$ kN·m $M_p = +0.35M_o = +148.8$ kN·m			
Short Direction	**Column Strip**		**Middle Strip**	
Moment factor	$0.75M_n$	$0.60M_p$	$0.25M_n$	$0.40M_p$
M_u (kN·m)	−207.3	± 89.3	−69.1	± 59.5
d (mm)	180	100	100	100
Strip width b (m)	2.7	2.7	3.3	3.3
$R_u = \frac{M_u}{bd^2}$ (MPa)	2.37	3.30	2.10	1.80
Steel ratio, ρ (%)	0.69	1.00	0.6	0.5
$A_s = \rho\, bd$ (mm^2)	3353	2700	1980	1650
Min. $A_s = 0.0018bh$ (mm ^{2})	1070	680	832	832
Bars selected (straight bars)	18 × 16 mm	14 × 16 mm	18 × 12 mm	16 × 12 mm
Spacing (mm)	150	195	185	205

Because $l_2 < l_1$, the width of the column strip in the long direction is $2(0.25 \times 5.4) = 2.7$ m. The width of the column strip in the short direction is 2.7 m. Assuming that the steel bars are 12 mm in diameter and those in the short direction are placed on top of the bars in the long direction, then the effective depth in the short direction will be about 10 mm less than the effective depth in the long direction. The d values and the design procedure are shown in Table 17.14. Minimum lengths of the selected reinforcement bars should meet the ACI Code length requirements shown in Fig. 17.16. Note that all steel ratios are less than ρ_{max}. Thus, $\phi = 0.9$.

5. The column stiffness is

$$\text{Ratio } \frac{\text{D.L.}}{\text{L.L.}} = \frac{8.6}{11} = 0.782 \text{ and } \frac{l_2}{l_2} = 1.11$$

Determine α_{min} from Table 17.7, taking into account that the relative beam stiffness is 0 because no beams are used. By interpolation, $\alpha_{min} = 1.15$. An approximate method is used here to determine the stiffness of the column with its capital.

I_s (moment of inertia of slab, short direction)

$$= 6000\frac{(140)^3}{12} = 1372 \times 10^6 \text{mm}^4$$

$$K_s = \frac{4E_cI_s}{l_2} = \frac{4E_c \times 1372 \times 10^6}{5400} = 1016 \times 10^3 E_c$$

I_c (for circular column, diameter 400 mm)

$$= \frac{\pi D^4}{64} = \frac{\pi}{64}(400)^4 = 1257 \times 10^6 \text{mm}^4$$

$$K_c = \frac{4E_cI_c}{l_c} = \frac{4E_c \times 1257 \times 10^6}{3000\text{mm}} = 1676 \times 10^3 E_c$$

Ratio of column stiffness/slab stiffness

$$= \frac{K_c}{K_s} = \frac{1676 \times 10^3}{1016 \times 10^3} = 1.65$$

which is greater than $\alpha_{\min}$ of 1.15. If I_s in the long direction is used, the calculated ratio of column to slab stiffness will be greater than 1.65. Therefore, the column is adequate.

6. Determine the balanced moment in the column and check the shear stresses in the slab, as explained in Examples 17.8 and17.9.

SUMMARY

Sections 17.1–17.5

1. A two-way slab is one that has a ratio of length to width less than 2. Two-way slabs may be classified as flat slabs, flat plates, waffle slabs, or slabs on beams.
2. The ACI Code specifies two methods for the design of two-way slabs: the direct design method and the equivalent frame method. In the direct design method, the slab panel is divided (in each direction) into three strips, one in the middle (referred to as the *middle strip*) and one on each side (referred to as *column strips*).

Section 17.6

To control deflection, the minimum slab thickness, h, is limited to the values computed by Table 17.1 or Eqs. 17.1 and 17.2 and as explained in Examples 17.1 and 17.2.

Section 17.7

For two-way slabs without beams, the shear capacity of the concrete section in one-way shear is

$$V_c = 2\lambda\sqrt{f'_c}bd$$

The shear capacity of the concrete section in two-way shear is

$$V_c = \left(2 + \frac{4}{\beta_c}\right)\lambda\sqrt{f'_c}b_od \leq 4\sqrt{f'_c}b_od$$

When shear reinforcement is provided, $V_n \leq 6\sqrt{f'_c}b_od$.

Section 17.8

In the direct design method, approximate coefficients are used to compute the moments in the column and middle strips of two-way slabs. The total factored moment is

$$M_o = (w_u l_2)\frac{l_1^2}{8} \tag{17.11}$$

The distribution of M_o into negative and positive span moments is given in Fig. 17.14. A summary of the direct design method is given in Section 17.8.8. The modified stiffness method is explained in Section 17.8.7.

Sections 17.9–17.11

1. Unbalanced loads on adjacent panels cause a moment in columns that can be computed by Eq. 17.22.
2. Approximately 60% of the moment transferred to both ends of a column at a joint is transferred by flexure, M_f, and 40% is transferred by eccentric shear, M_v. The fraction of the unbalanced moment transferred by flexure, M_f, is $\gamma_f M_u$, where γ_f is computed from Eq. 17.25. The shear stresses produced by M_v must be combined with shear stresses produced by the shearing force V_u.
3. Waffle slabs are covered in Section 17.11.

Section 17.12

1. In the equivalent frame method, the building is divided into equivalent frames in two directions and then analyzed for all conditions of loadings. Example 17.13 explains this procedure.
2. Example 17.14 is an example of a two-way flat slab with drop panel (SI units).

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PROBLEMS

17.1 (Flat plates) Determine the minimum slab thickness according to the ACI Code for the flat-plate panels shown in Fig. 17.42 and Table 17.15. The floor panels are supported by 24 × 24-in. columns,

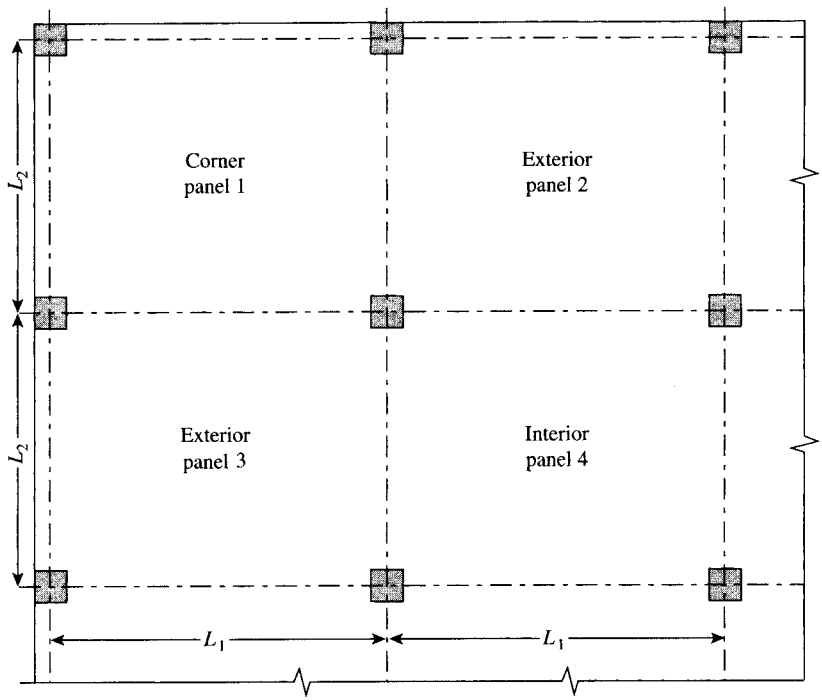


Figure 17.42 Problem 17.1.

Table 17.15 Problem 17.1

Number (Flat Plate)	Panel Dimensions (ft)		Panel Numbers
	L_1	L_2	
(a)	20	20	1 and 4
(b)	24	24	2 and 4
(c)	26	26	3 and 4
(d)	20	16	1 and 2
(e)	24	20	3 and 4
(f)	26	22	1 and 4
(g)	30	24	1 and 2
(h)	30s	30	1 and 4

12 ft long, with no edge beams at the end of the slab. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, dead load (excluding self-weight) = 55 psf, and live load = 120 psf.

17.2 (Flat plates) Use the direct design method to design the interior flat-plate panel (no. 4) of Problems 17.1*a*, *b*, *c*, and *e*, using the data given earlier. Check the shear and moment transfer at an interior column. Draw sketches showing the reinforcement distribution and the shear stresses.

17.3 (Flat plates) Repeat Problem 17.2 for the exterior panel no. 3. Check the shear and moment transfer at the exterior column. If shear stresses are not adequate, use shear reinforcement involving stirrups.

17.4 (Flat slabs with drop panels) Determine the minimum slab and drop panel thicknesses according to the ACI Code for the slabs shown in Fig. 17.42 and Table 17.15. The floor panels are supported by 24 × 24 -in. columns with no edge beams. Use $f'_c = 4$ ksi, $f_y = 60$ ksi, additional dead load (excluding self-weight) = 60 psf, and live load = 120 psf.

17.5 (Flat slabs) Use the direct design method to design the interior flat slab panel no. 4, of Problem 17.4*a*, *b*, *c*, and *e*, using the data given in Problem 17.4. Check the shear and moment transfer at an interior column. Draw sketches showing the reinforcement distribution and the shear stresses. Use a 4-ft-column capital diameter for part *c* only.

17.6 (Flat slabs) Repeat Problem 17.5 for the exterior panel no. 3.

17.7 (Slabs on beams) Redesign the slabs in Problem 17.2, using the same data when the slabs are supported by beams on all four sides. Each beam has a width $b_w = 14$ in. and a projection below the bottom of the slab of 18 in.

17.8 (Slabs on beams) Redesign the slabs in Problem 17.7 as exterior panels.

17.9 (Waffle slabs) Repeat Example 17.12 when the spans are (a) 36 ft and (b) 42 ft. Use the same data and 24 × 24-in. columns.

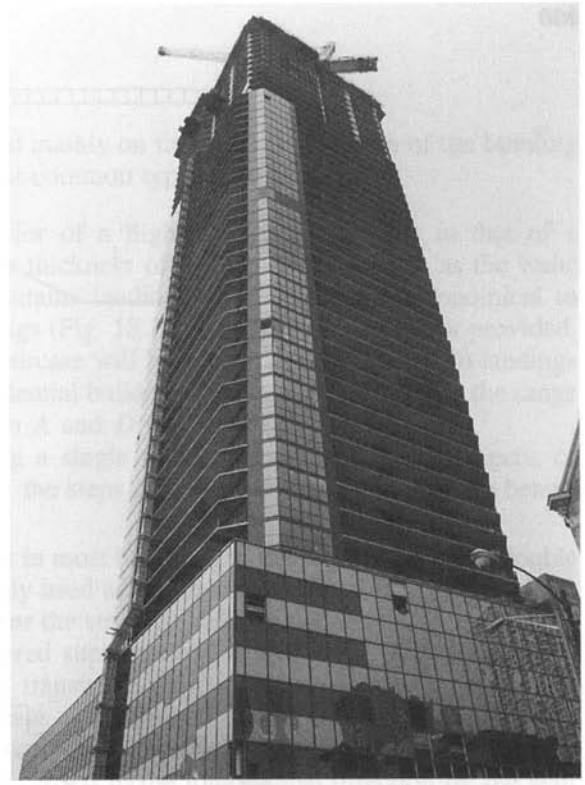
17.10 (Waffle slabs) Redesign the waffle slabs in Problem 17.9 as exterior panels.

17.11 (Equivalent frame method) Redesign the flat-plate floor system of Problem 17.2*a* and *b* using the equivalent frame method.

17.12 (Equivalent frame method) Redesign the waffle slabs of Problem 17.9 using the equivalent frame method.

CHAPTER 18

STAIRS



Office building under construction, Chicago, Illinois.

18.1 INTRODUCTION

Stairs must be provided in almost all buildings, either low-rise or high-rise, even if adequate numbers of elevators are provided. Stairs consist of rises, runs (or treads), and landings. The total steps and landings are called a *staircase*. The *rise* is defined as the vertical distance between two steps, and the *run* is the depth of the step. The *landing* is the horizontal part of the staircase without rises (Fig. 18.1).

The normal dimensions of the rises and runs in a building are related by some empirical rules.

$$\text{Rise} + \text{run} = 17 \text{ in}$$

$$2 \times \text{rise} + \text{run} = 25 \text{ in. (635 mm)}$$

$$\text{rise} \times \text{run} = 75 \text{ in.}^2 \ (0.05 \text{ m}^2)$$

The rise depends on the use of the building. For example, in public buildings the rise is about 6 in., whereas in residential buildings it varies between 6 and 7.5 in. The run is about 1 ft in public buildings and varies between 9 in. and 12 in. in residential buildings. In general, a rise should not exceed 8 in. or be less than 4 in., and the number of rises is obtained by dividing the structural floor-to-floor dimension by the assumed rise.

The finishing on the stairs varies from troweling Alundum grits to adding asphalt tiles, terrazzo tiles, marble, or carpets. In addition to dead loads, stairs must be designed for a minimum live load of 100 psf.

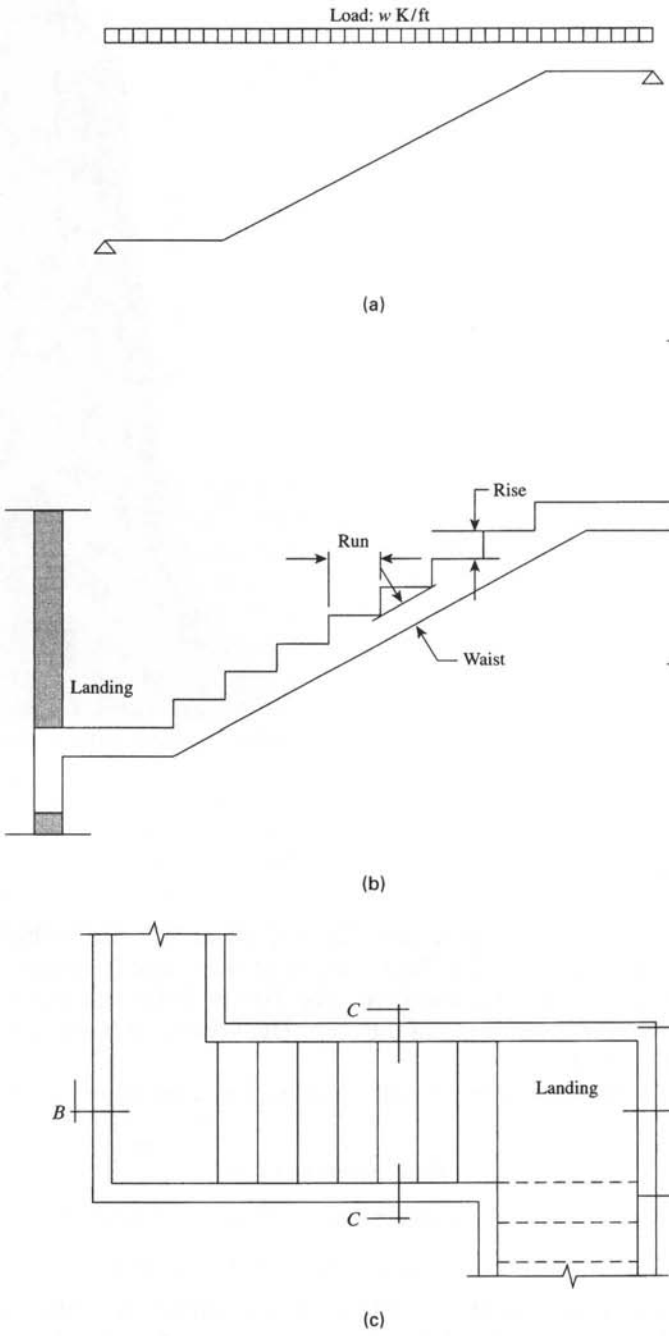


Figure 18.1 Plan of a single-flight staircase: (*a*) loads, (*b*) section *B*-*B*, and (*c*) plan.

18.2 TYPES OF STAIRS

There are different types of stairs, which depend mainly on the type and function of the building and on the architectural requirements. The most common types are as follows.

1. *Single-flight stairs:* The structural behavior of a flight of stairs is similar to that of a one-way slab supported at both ends. The thickness of the slab is referred to as the waist (Fig. 18.1). When the flight of stairs contains landings, it may be more economical to provide beams at B and C between landings (Fig. 18.2). If such supports are not provided, which is quite common, the span of the staircase will increase by the width of two landings and will extend between A and D. In residential buildings, the landing width is in the range of 4 to 6 ft, and the total distance between A and D is about 20 ft.

   An alternative method of supporting a single flight of stairs is to use stringers, or edge beams, at the two sides of the stairs; the steps are then supported between the beams (Fig. 18.3).
2. *Double-flight stairs:* It is more convenient in most buildings to build the staircase in double flights between floors. The types commonly used are quarter-turn (Fig. 18.4) and closed-or open-well stairs, as shown in Fig. 18.5. For the structural analysis of the stairs, each flight is treated as a single flight and is considered supported on two or more beams, as shown in Fig. 18.2. The landing extends in the transverse direction between two supports and is designed as a one-way slab. In the case of open-well stairs, the middle part of the landing carries a full load, whereas the two end parts carry half-loading only, as shown in Fig. 18.5(*d*). The other half-loading is carried in the longitudinal direction by the stair flights, sections A-A and B-B.
3. *Three or more flights of stairs:* In some cases, where the overall dimensions of the staircase are limited, three or four flights may be adopted (Fig. 18.6). Each flight will be treated separately, as in the case of double-flight staircases.
4. *Cantilever stairs:* Cantilever stairs are used mostly in fire-escape stairs, and they are supported by concrete walls or beams. The stairsteps may be of the full-flight type, projecting from one side of the wall, the half-flight type, projecting from both sides of the supporting wall, or of the semispiral type, as shown in Fig. 18.7. In this type of stairs, each step acts as a cantilever, and the main reinforcement is placed in the tension side of the run and the bars are anchored within the concrete wall. Shrinkage and temperature reinforcement is provided in the transverse direction.

   Another form of a cantilever stair is that using open-riser steps supported by a central beam, as shown in Fig. 18.8. The beam has a slope similar to the flight of stairs and receives the steps on its horizontally prepared portions. In most cases, precast concrete steps are used, with special provisions for anchor bolts that fix the steps into the beam.
5. *Precast flights of stairs:* The speed of construction in some projects requires the use of precast flights of stairs (Fig. 18.8). The flights may be cast separately and then fixed to cast-in-place landings. In other cases, the flights, including the landings, are cast and then placed in position on their supporting walls or beams. They are designed as simply supported one-way slabs with the main reinforcement at the bottom of the stair waist. Adequate reinforcement must be provided at the joints, as shown in Fig. 18.9.

   Provisions must be made for lifting and handling the precast stair units by providing lifting holes or inserting special lifting hooks into the concrete. Special reinforcement must be provided at critical locations to account for tensile stresses that will occur in the stairs from the lifting and handling process.

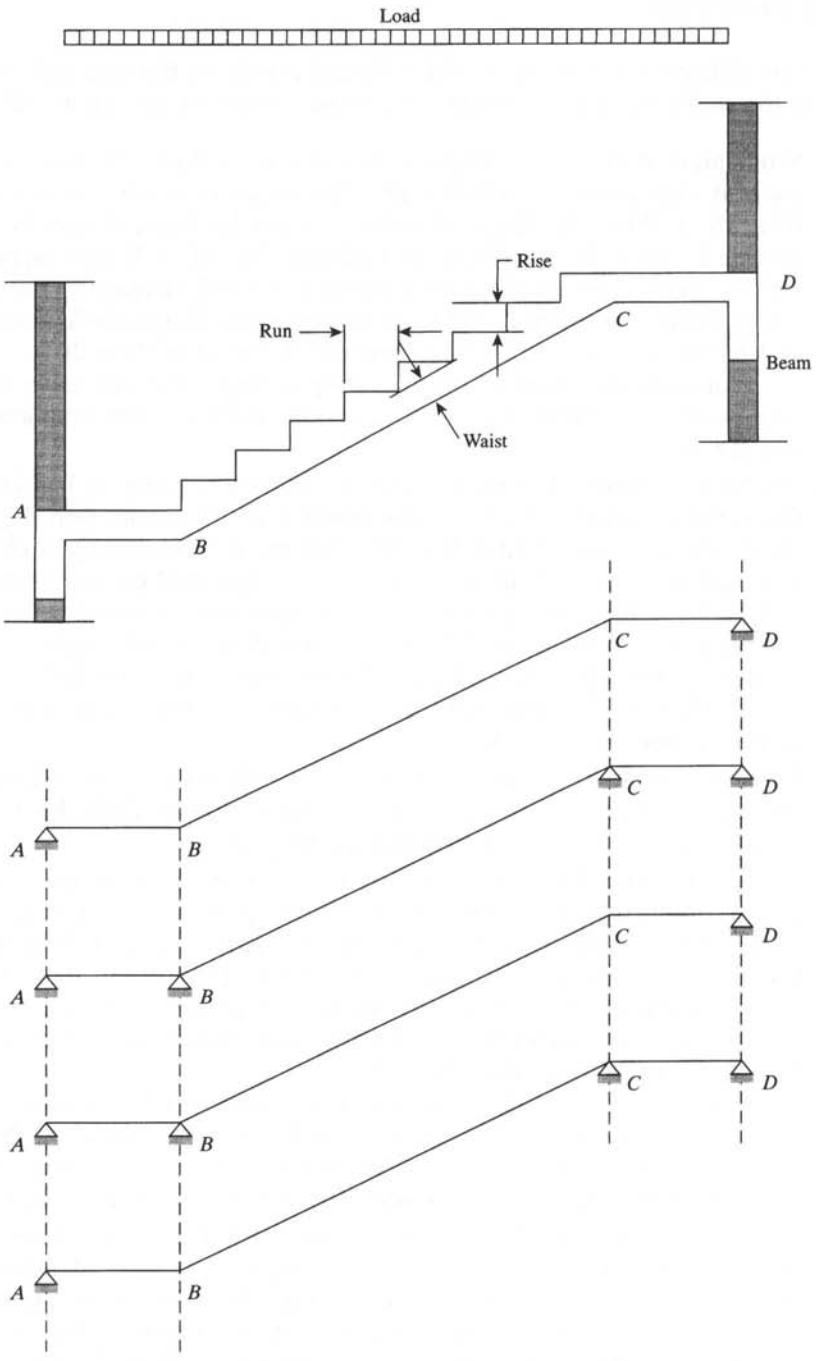


Figure 18.2 Supporting systems of one flight.

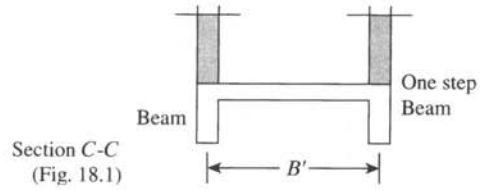


Figure 18.3 Steps supported by stringer beams.

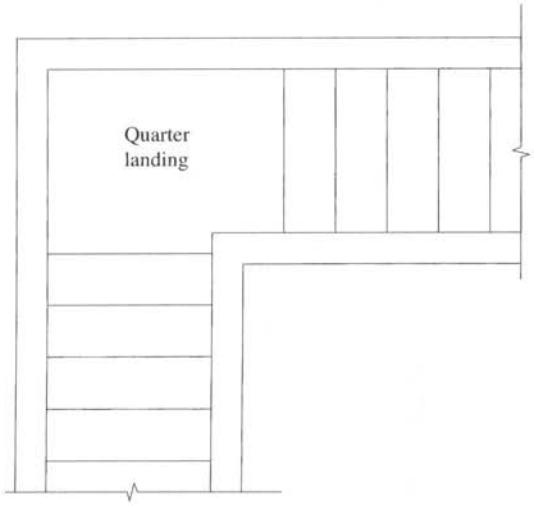


Figure 18.4 Quarter-turn staircase.

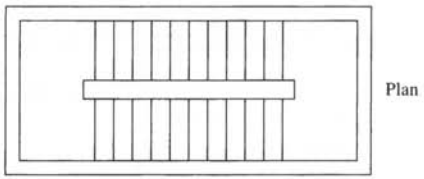
6. *Free-standing staircase:* In this type of stairs, the landing projects into the air without any support at its end (Fig. 18.10). The stairs behave in a springboard manner, causing torsional stresses in the slab.

   Three systems of loading must be considered in the design of this type of stairs, taking into consideration that torsional moments will develop in the slab in all cases:

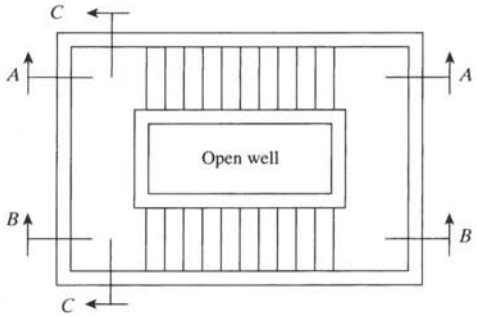
   a. When the live load acts on the upper flight and half the landing only (Fig. 18.11), the upper flight slab will be subjected to tensile forces in addition to bending moments, whereas the lower flight will be subjected to compression forces, which may cause buckling of the slab.

   b. When the live load acts on the lower flight and half the landing only (Fig. 18.12), the upper flight slab will be subjected to tensile forces, whereas the lower flight will be subjected to bending moment and compression forces.

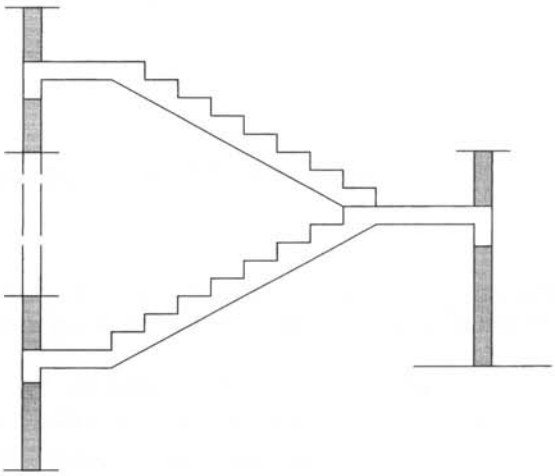
   c. When the live load acts on both upper and lower flights, the loading of one flight will cause the twisting of the other. The torsional stresses developed in the stairs require adequate reinforcement in both faces of the stair slabs and the landing. Transverse reinforcement in the slab and the landing must be provided in both faces of the concrete in the shape of closed U-bars lapping at midwidth of the stairs. Typical reinforcement details are shown in Fig. 18.13.



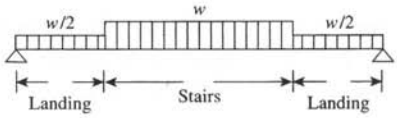
(a)



(b)

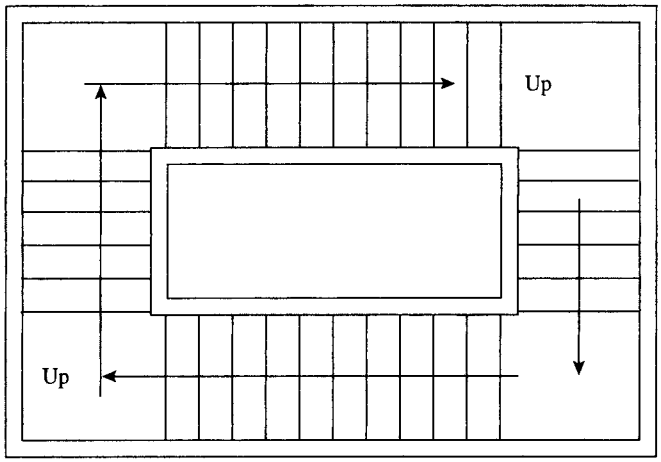


(c)

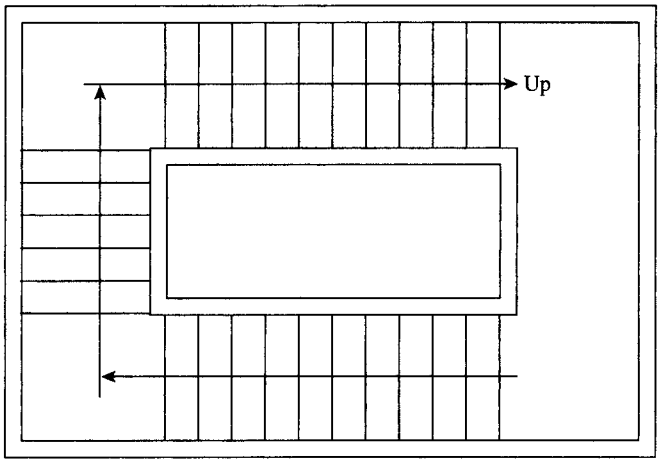


(d)

Figure 18.5 Double-flight stairs: (*a*) closed-well staircase, (*b*) open-well staircase, (*c*) section *B*-*B*, and (*d*) section *C*-*C*.



Four-stair flight



Three-stair flight

Figure 18.6 Three- and four-stair flights.

This type of stairs is favored by architects and sometimes called a pliers-shaped staircase or jackknife staircase.

A study was made to determine the effect of the following parameters on the free-standing staircases forces and moments considering a live load of 100 psf (Figs. 18.10 and 18.13).

1. *The width of the stairs (Fig. 18.10).* An increase in the width from 4 to 10 ft, will increase the forces and moments sharply. For example, the torsional moment along the flight increases by about 1,400%. Therefore, it is desirable to restrict the flight width between 4.0 and 6.0 ft. Other moments increase by about 450%.
2. *The span length* L. An increase in the span L will increase the forces and moments in the stair flight and landing significantly. For example, if L is increased from 8 ft to

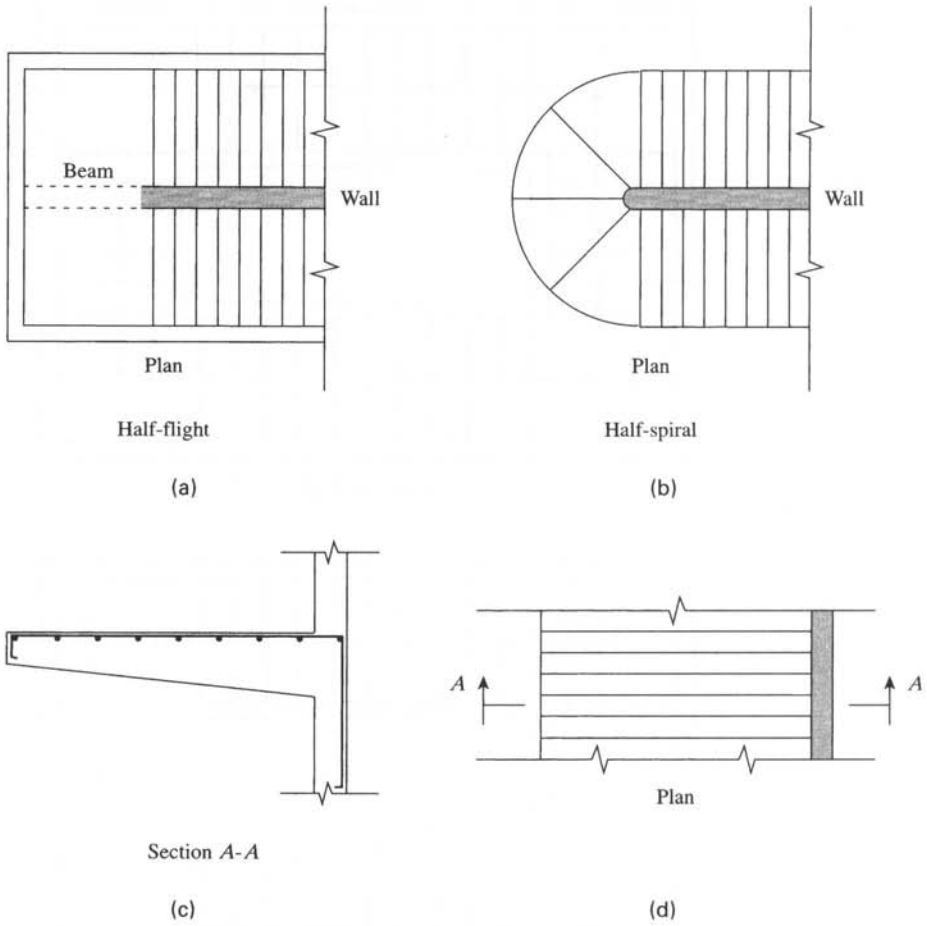


Figure 18.7 Steps projecting from one or two sides of the supporting wall.

16 ft, the shearing forces at the top edge of the stairs increases by about 230%. Moments increase by about 100% to 150%.

3. *The total flight height* h. If h is increased from 10 ft to 16 ft, the shearing force at the top edge increases by about 150%. Moments increase by about 50 to 100%.
4. *The flight slab thickness* t. This parameter has the least effect on forces and moments. For example, if t is increased from 6 to 10 in., the moments increase by about 25% and the shearing force by about 20%.
5. For practical design, the parameters may be chosen as follows: flight width between 4- and 6 ft, horizontal span L between 9- and 12 ft, total flight height between 10- and 15 ft, and slab thickness between 6- and 10 in.

The above information is a guide to help the designer to choose the right parameters for an economical design.

7. *Run-riser stairs:* Run-riser stairs are stepped underside stairs that consist of a number of runs and risers rigidly connected without the provision of the normal waist slab (Fig. 18.14*a*). This type of stairs has an elegant appearance and is sometimes favored by architects. The

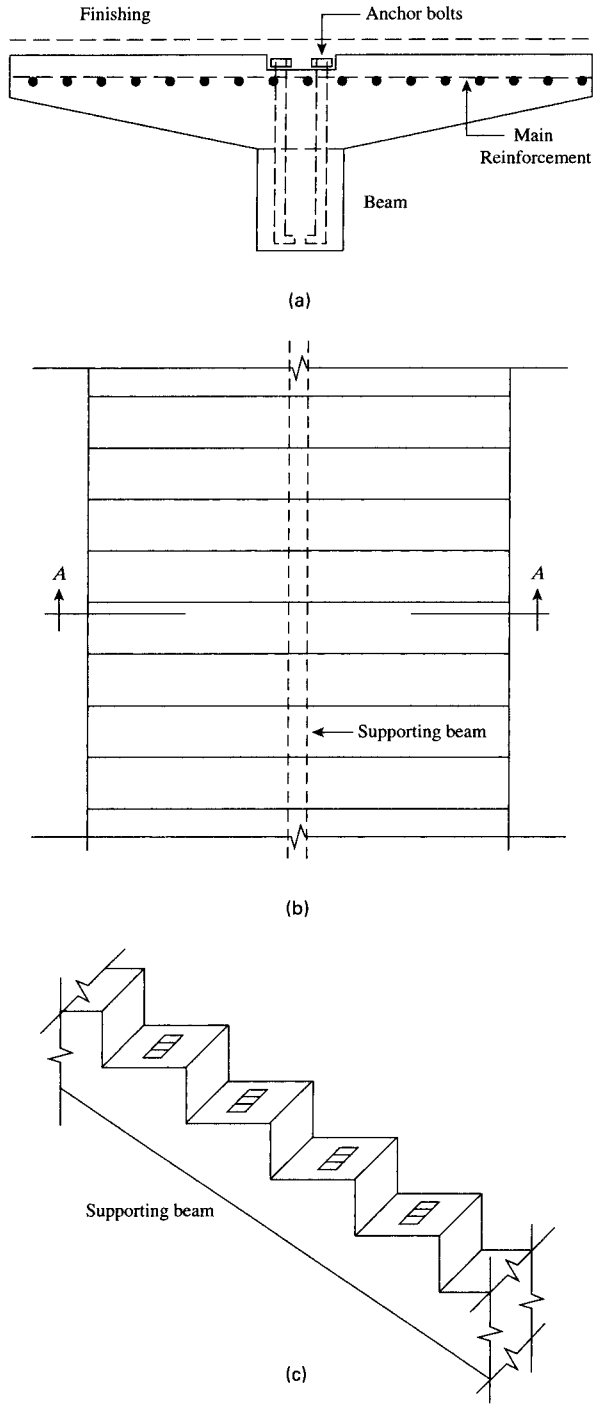


Figure 18.8 Precast cantilever stair supported by central beam: (*a*) section *A*-*A*, (*b*) part plan, and (*c*) supporting beam.

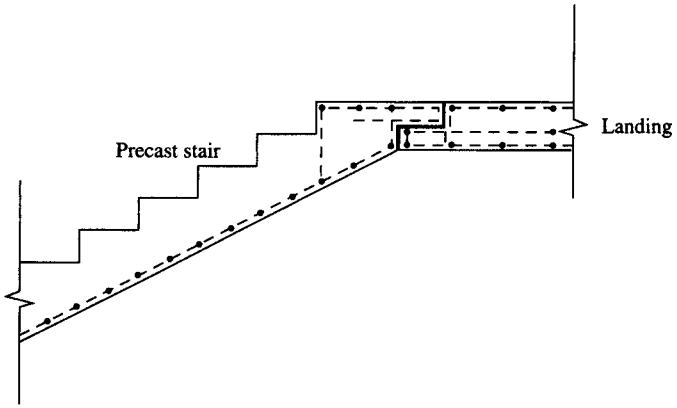


Figure 18.9 Joint of a precast concrete flight of stairs.

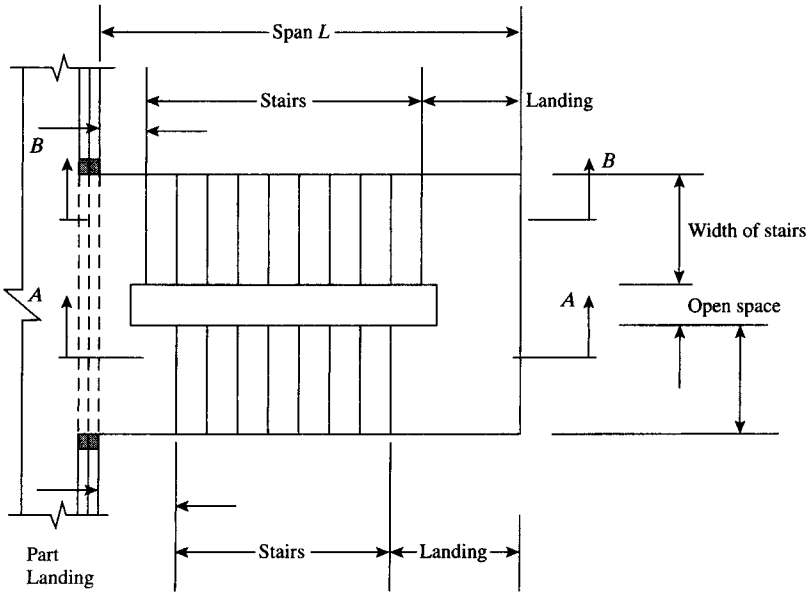


Figure 18.10 Plan of a free-standing staircase.

structural analysis of run-riser stairs can be simplified by assuming that the effect of axial forces is negligible and that the load on each run is concentrated at the end of the run (Fig. 18.14*b*). For the analysis of a simply supported flight of stairs, consider a simple flight of two runs, ABC, subjected to a concentrated load P at B' (Fig. 18.14*b*). Because joints B and B' are rigid, the moment at joint B is equal to the moment at B', or

$$M_B = M'_B = \frac{PS}{2}$$

where S is the width of the run. The moment in rise, BB', is constant and is equal to $PS/2$.

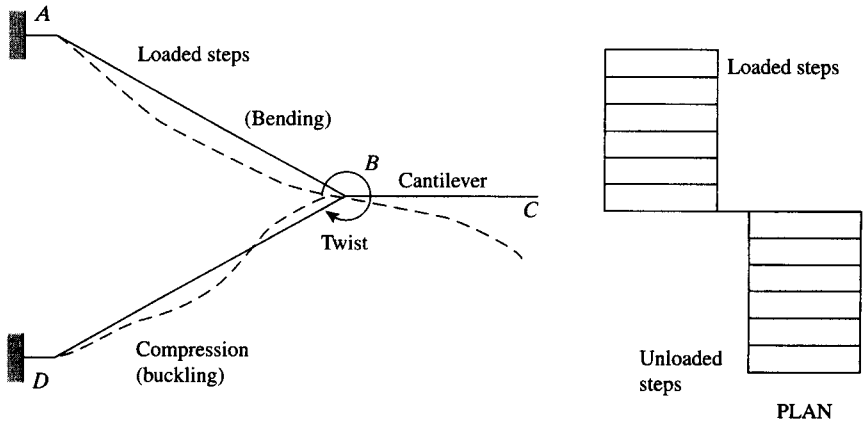


Figure 18.11 Case 1, *ABC* loaded.

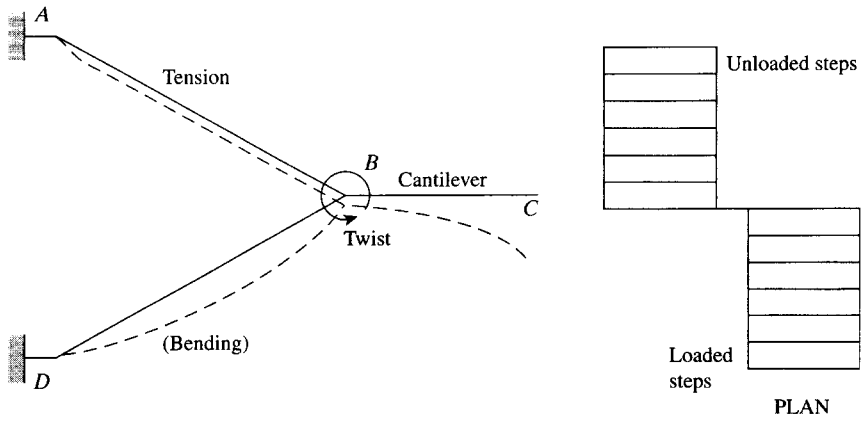


Figure 18.12 Case 2, *DBC* loaded.

When the rise is absent, the stairs, ABC, act as a simply supported beam, and the maximum bending moment occurs at midspan with value

$$M_B = \frac{PL}{4} = \frac{PS}{2}$$

For a flight of stairs that consists of a number of runs and risers, the same approach can be used; the bending moment diagram is shown in Fig. 18.15*a*. The moment in BB' is constant and is equal to the moment at joint B, or $2PS$. Similarly, $M_C = M'_C = 3PS$, $M_D = M'_D = 3PS$, and $M_E = M'_E = 2PS$

If a landing is present at one or both ends, the load on the landing practically may be represented by concentrated loads similar to the runs. The structural analysis may also be performed by considering a load uniformly distributed on the flight of stairs. The moment in every riser is constant and is obtained from the bending moment diagram of a simply supported beam subjected to a uniform load (Fig. 18.15*b*). Example 18.3 illustrates the design of a staircase using the two assumptions of concentrated loads and uniform loads.

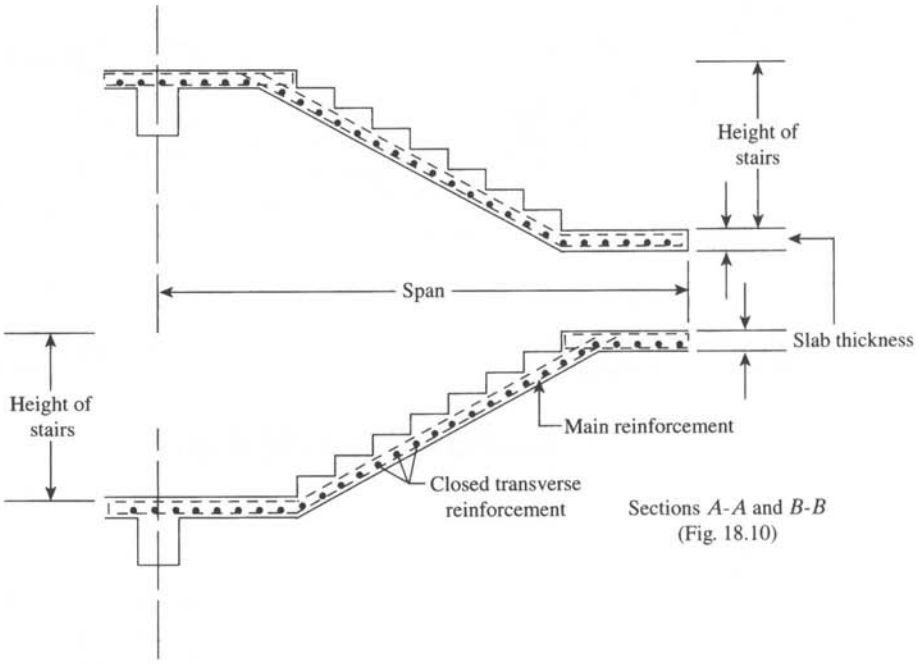


Figure 18.13 Section of a free-standing staircase.



Free-standing staircase.

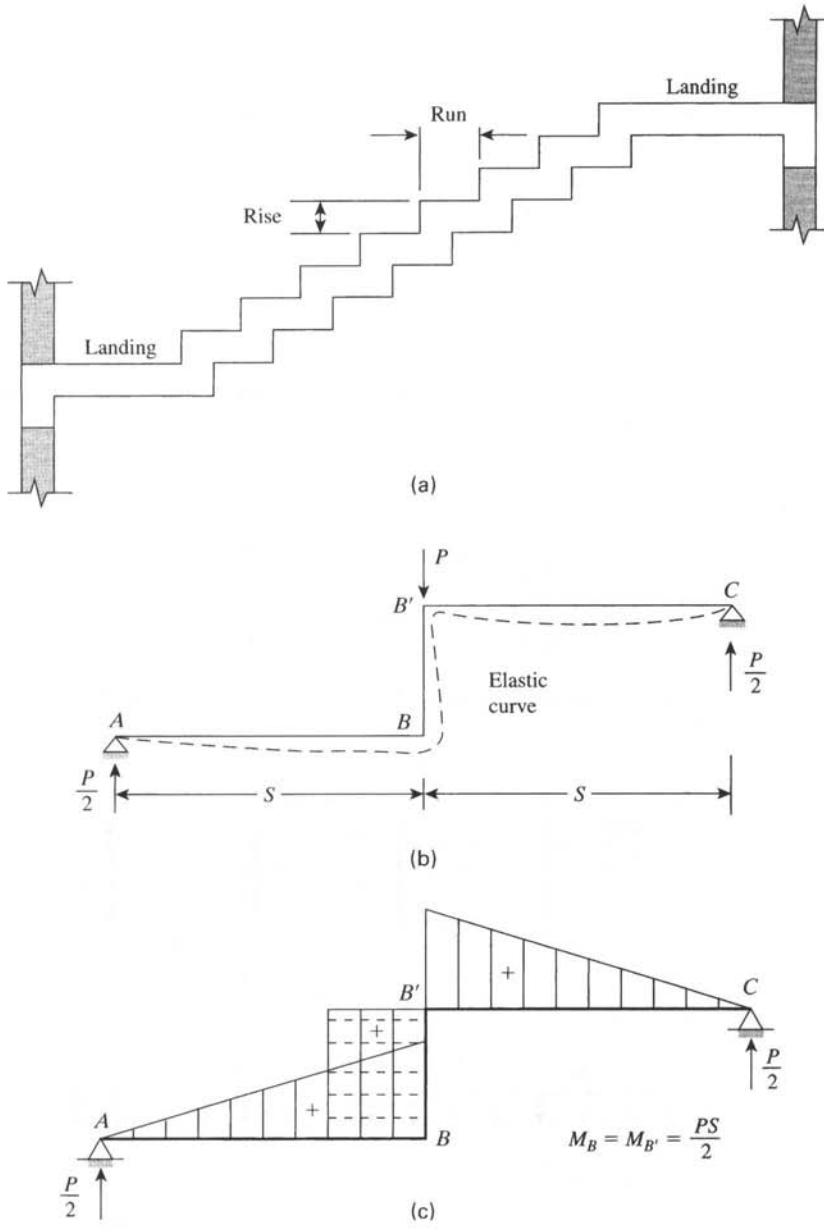
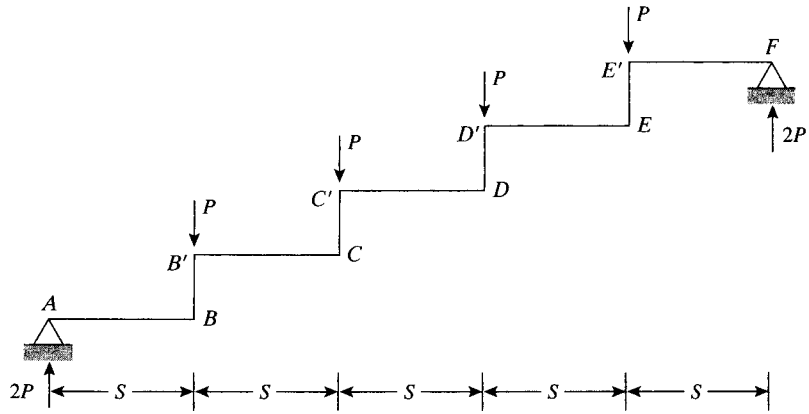
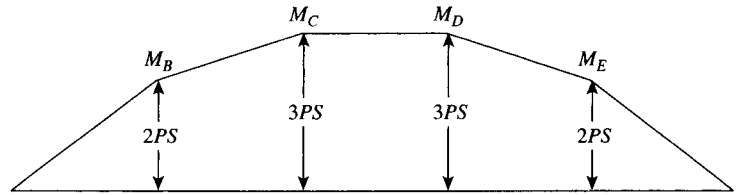
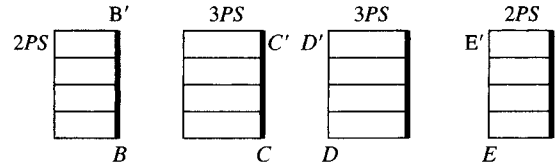


Figure 18.14 Run-riser staircase: (*a*) cross-section, (*b*) elastic curve, and (*c*) bending moment diagram.

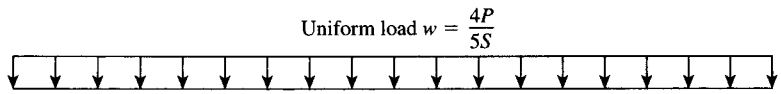
If the stair flight is fixed or continuous at one or both ends, the moments can be obtained using any method of structural analysis. To explain this case, consider a flight of stairs that consists of two runs and is fixed at both ends (Fig. 18.16*a*). The moments at the fixed ends, A and B, due to a concentrated load at B are equal to $PL/8 = PS/4$. This result is obtained by assuming that the rise does not exist and the stairs, ABC, act as a

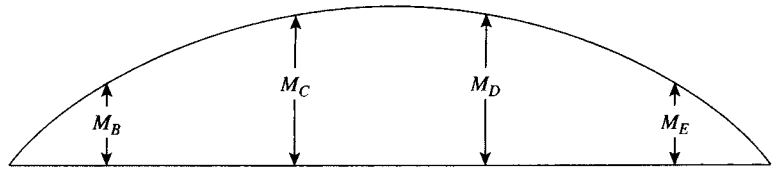






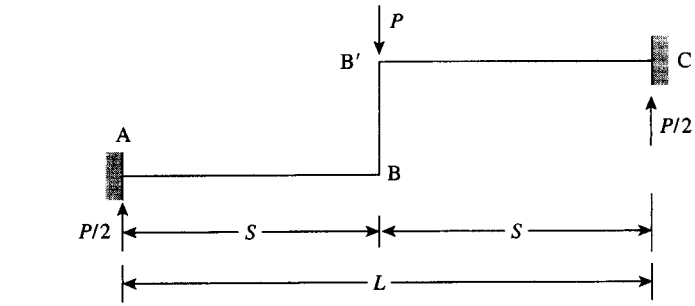
(a)

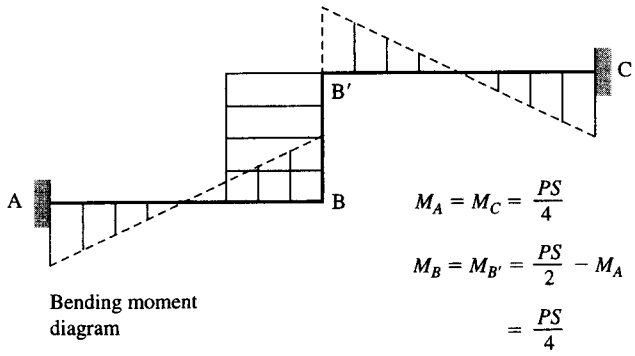




(b)

Figure 18.15 Distribution of moments: (*a*) bending moment due to concentrated loads and (*b*) bending moment due to uniform load.



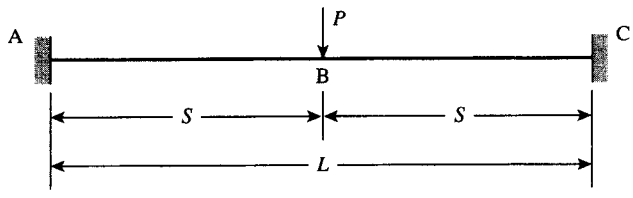


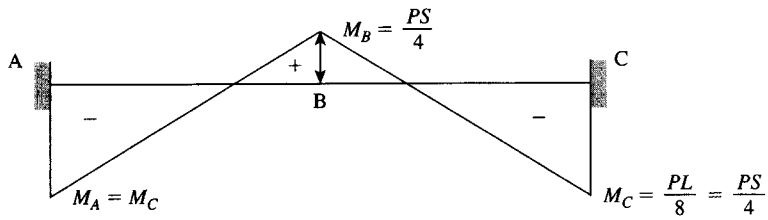
$$M_A = M_C = \frac{PS}{4}$$

$$M_B = M_{B'} = \frac{PS}{2} - M_A$$

$$= \frac{PS}{4}$$

(a)





(b)

Figure 18.16 Fixed-end staircase: (*a*) loaded steps and (*b*) loaded beam.

fixed-end beam subjected to a concentrated load at midspan (Fig. 18.16*b*). The moment at midspan, section B, is equal to

$$\frac{PL}{4} - M_A = \frac{PS}{2} - \frac{PS}{4} = \frac{PS}{4}$$

The bending moment of a flight of stairs with one riser is shown in Fig. 18.16*a*. Note that the moment in the riser BB' is constant, and $M_B = M'_B = PS/4$.

For a symmetrical stair flight, fixed at both ends and subjected to a number of concentrated loads at the node of each run, the moment at the fixed end can be calculated as follows:

$$M \text{ (fixed end)} = \frac{PS}{12}(n^2 - 1)$$

where

P = concentrated load at the node of the run
S = width of run
n = number of runs

When $n = 2$, then

$$M \text{ (fixed end)} = \frac{PS}{12}(4 - 1) = \frac{PS}{4}$$

which is the same result obtained earlier.

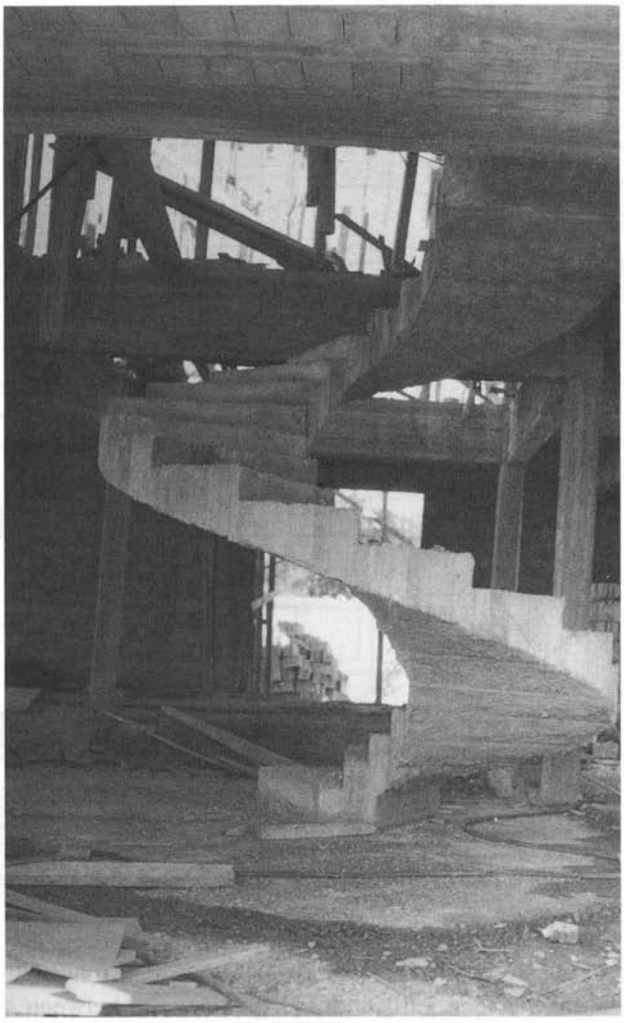
If a landing is present at one or both ends, the load on the landing may be represented by concentrated loads at spacing S.

8. *Helical stairs (open-spiral stairs):* A helical staircase is a three-dimensional structure, which usually has a circular shape in plan (Fig. 18.17). It is a distinctive type of stairs used mainly in entrance halls, theater foyers, and special low-rise office buildings. The cost of a helical stair is much higher than that of a normal staircase.

The stairs may be supported at some edges within adjacent walls or may be designed as a free-standing helical staircase, which is most popular. The structural analysis of helical staircases is complicated and was discussed by Morgan [1] and Scordelis [2] using the principles of strain energy. Design charts for helical stairs are also prepared by Cusens and Kuang [3]. Under load, the flight slab will be subjected to torsional stresses throughout. The upper landing will be subjected to tensile stresses, whereas compressive stresses occur at the bottom of the flight. The forces acting at any section may consist of vertical moment, lateral moment, torsional moment, axial force, shearing force across the waist of the stairs, and radial horizontal shearing force. The main longitudinal reinforcement consists of helical bars placed in the concrete waist of the stairs and runs from the top landing to the bottom support. The transverse reinforcement must be in a closed stirrup form to resist torsional stresses or in a U-shape lapped at about the midwidth of the stairs.

A study was made to determine the effect of the following parameters on the forces and moments that develop on helical staircases. These parameters are:

1. The total arc subtended by the helix with an angle that normally ranges from 240° to 360°. Referring to Fig. 18.17, for 16 equal runs at 20° pitch, the total arc equals 320°. If the arc is increased from 240° to 360° the vertical moment may increase by about 1,200% for a live load of 100 psf. Other forces increase appreciably.



Reinforced concrete helical staircase.

2. The width of stairs that normally ranges from 4 to 8 ft. All other parameters are constant. The increase of stair width by 100%, from 4 to 8 ft, increases the torsional moment by about 700%.
3. Variation in the interior and exterior radii (R_i and R_e) keeping the stair width of 6 ft constant. The increase in R_e (from 9 to 12 ft) and R_i (from 3 to 6 ft) with a ratio of R_e/R_i that varies between 3 and 2, increases the lateral moment by about 230%.
4. The thickness of stair slab is not as critical as the other parameters. For a variation in slab thickness between 6 and 12 in., the lateral moment increases by about 70%, while the torsional moment increases by about 170%.
5. The total height of the helical stair, h, has the least effect on all forces (for h between 9 and 15 ft) The increase in lateral moment is about 70% and in torsional moment is about 40%. Other forces decrease by about 80%.

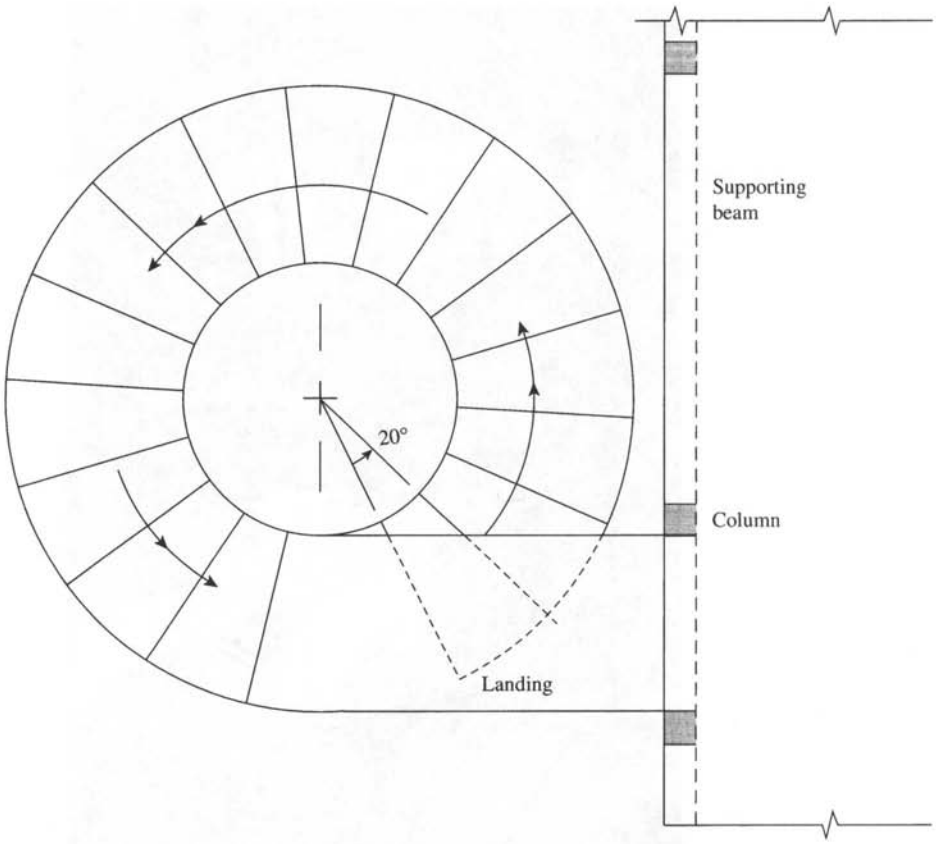


Figure 18.17 Plan of a helical staircase (16 equal runs at 20° pitch).

6. Based on this study, the possible practical dimensions may be chosen as follows: Total subtended arc between 120° and 320°, stair width between 4 and 6 ft, stairs slab thickness between 6 and 10 in., and stair height between 10 and 15 ft.

The above information can be used as a guide to achieve a proper and economical design of helical staircase.

An alternative method of providing a helical stair is to use a central helical girder located at the midwidth of the stairs and have the steps project equally on both sides of the girder. Each step is analyzed as a cantilever, and the reinforcement bars extend all along the top of the run. Precast concrete steps may be used and can be fixed to specially prepared horizontal faces at the top surfaces of the girder.

18.3 EXAMPLES

Example 18.1

Design the cantilever stairs shown in Fig. 18.18 to carry a uniform live load of 100 psf. Assume the rise of the steps equals 6.0 in. and the run equals 12 in. Use normal-weight concrete with $f'_c = 3$ ksi and $f_y = 60$ ksi.

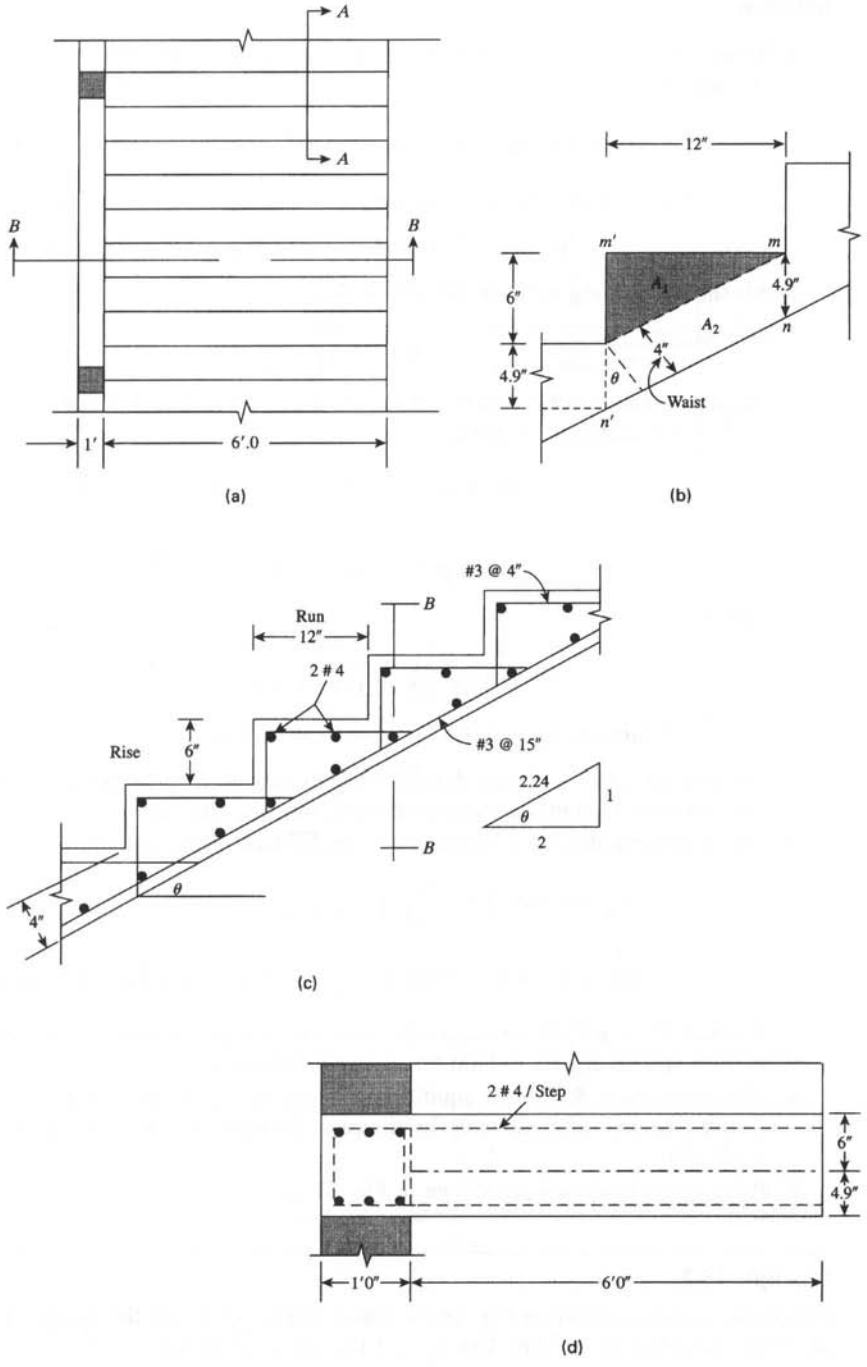


Figure 18.18 Example 18.1: cantilever stairs: (*a*) plan, (*b*) section in one step, (*c*) section *A*-*A*, and (*d*) section *B*-*B*.

Solution

1. Loads: Assume the thickness of the slab (waist) is 4.0 in. Weight of the assumed slab (areas A_1 and A_2) is

$$\text{trapezoidal area } mnn'm' = \left(\frac{4.9 + 10.9}{2 \times 12}\right)(1)(150) = 98.8 \text{ lb/ft}$$

Refer to Fig. 18.18*b*. Assume the weight of the step cover is 5 lb/ft. Total D.L. = 119 lb/ft.

$$W_u = 1.2D + 1.6L = 1.2 \times 119 + 1.6 \times 100 = 302 \text{ lb/ft}$$

2. Maximum bending moment per step is $W_u l^2/2$.

$$M_u = \frac{0.302}{2}(6)^2 = 5.44 \text{ K·ft}$$

Average thickness of a step is (10.9 + 4.9)/2 = 7.9 in. Let d = 7.9 − 0.75 (concrete cover) − 0.25 ($\frac{1}{2}$ bar diameter) = 6.9 in.

$$M_u = \phi A_s f_y \left(d - \frac{a}{2}\right) \qquad \text{Assume } a = 0.5 \text{ in.}$$

$$A_s = \frac{5.44 \times 12}{0.9 \times 60(6.9 - 0.25)} = 0.19 \text{ in.}^2$$

Check

$$a = \frac{A_s f_y}{0.85\, f'_c b} = \frac{0.19 \times 60}{0.85 \times 3 \times 12} = 0.38 \text{ in.} \qquad \text{(close to 0.5 in.)}$$

$$\text{Minimum } A_s = 0.00333(12)(6.9) = 0.28 \text{ in.}^2$$

Use two no. 4 bars per step. A smaller depth may be adopted, but to avoid excessive deflection and vibration of stairs, a reasonable depth must be chosen.

3. Check flexural shear at a distance d from the face of the support.

$$V_u = 0.315\left(6 - \frac{6.9}{12}\right) = 1.7 \text{ K}$$

$$\phi V_c = 0.75(2\lambda\sqrt{f'_c}bd) = \frac{0.75}{1000} \times 2 \times 1 \times \sqrt{3000} \times 12 \times 6.9 = 6.8 \text{ K}$$

Because $V_u < \phi V_c/2$, no shear reinforcement is required. But it is recommended to use no. 3 stirrups spaced at 4 in. to hold the main reinforcement.

4. The stairs must remain in equilibrium either by the weight of the wall or by a reinforced concrete beam within the wall. In this case, the beam will be subjected to torsional moment of 5.7 K·ft/ft.

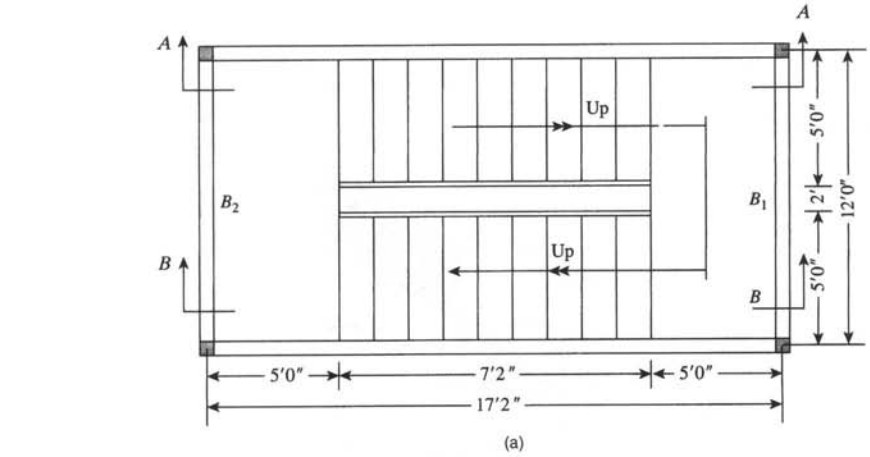
5. Reinforcement details are shown in Fig. 18.18.

Example 18.2

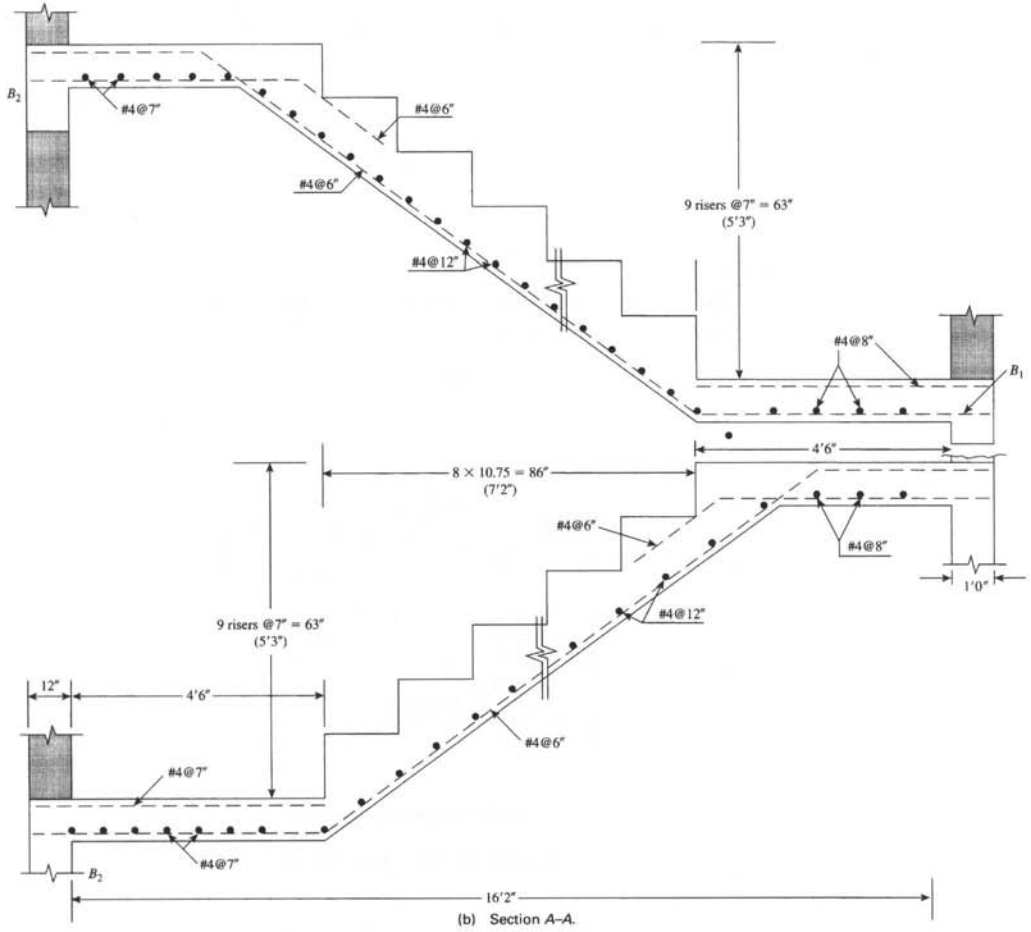
Design the staircase shown in Fig. 18.19, which carries a uniform live load of 120 psf. Assume a rise of 7.0 in. and a run of 10.75 in. Use f'_c = 3 ksi and f_y = 60 ksi.

Solution

1. Structural system: If no stringer beam is used, one of the four possible solutions shown in Fig. 18.2 may be adopted. When no intermediate supports are used, the flight of stairs will be supported at the ends of the upper and lower landings. This structural system will be adopted in this example.

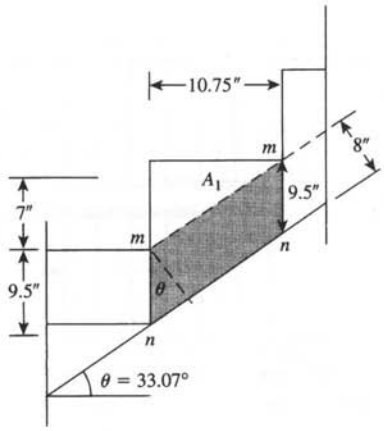


(a)

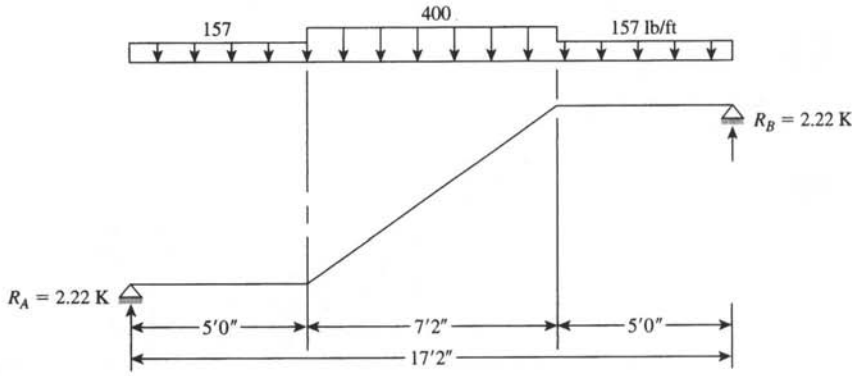


(b) Section A–A.

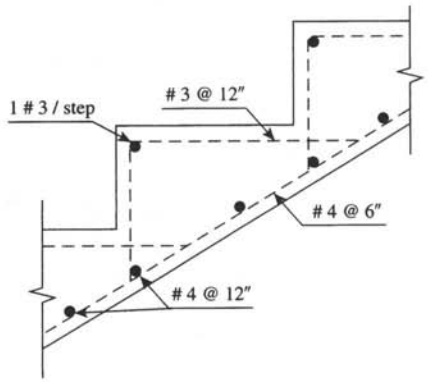
Figure 18.19 Example 18.2.



(c) Section in steps.



(d) Loads.



(e) Reinforcement details.

Figure 18.19 (*continued*)

2. Loads: Assume the thickness of the slab (waist) is 8.0 in.

$$\text{Weight of one step} = \text{trapezoidal area} \times 150 \text{ pcf}$$

$$= \left(\frac{9.5 + 16.5}{2 \times 12}\right)\left(\frac{10.75}{12}\right)(150) = 145.6 \text{ lb per step}$$

$$\text{Average weight per foot length} = 145.6\left(\frac{12}{10.75}\right) = 162.5 \text{ lb/ft}$$

$$\text{Weight of 8 in. landing} = \frac{8}{12} \times 150 = 100 \text{ lb/ft}$$

Assume weight of step cover is 7.5 lb/ft and weight of landing = 2 lb/ft. The total D.L. on stairs is 162.5 + 7.5 = 170 lb/ft. The total D.L. on landing is 100 + 2 = 102 lb/ft.

$$W_u \text{ (on stairs)} = 1.2 \times 170 + 1.6 \times 120 = 400 \text{ lb/ft}$$

$$W_u \text{ (on landing)} = 1.2 \times 102 + 1.6 \times 120 = 314 \text{ lb/ft}$$

Because the load on the landing is carried into two directions, only half the load will be considered in each direction.

3. Calculate the maximum bending moment and steel reinforcement (Fig. 18.19*d*):

a. The moment at midspan is

$$M_u = 2.22\left(\frac{17.2}{2}\right) - (0.157 \times 5)(6.1) - (0.400)\frac{(3.6)^2}{2} = 11.71 \text{ K·ft}$$

Let $d = 8.0 - 0.75$ (concrete cover) $- 0.25$ $\left(\frac{1}{2} \text{ bar diameter}\right) = 7.0$ in.

b. $M_u = \phi A_s f_y\,(d - a/2)$; assume $a = 0.8$ in.

$$A_s = \frac{11.71 \times 12}{0.9 \times 60(7 - 0.4)} = 0.4 \text{ in.}^2$$

Check:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.4 \times 60}{0.85 \times 3 \times 12} = 0.78 \text{ in.}, \qquad c = 0.92 \text{ in.}$$

$$\text{Minimum } A_s = 0.0033 \times 12 \times 8 = 0.32 \text{ in.}^2 < 0.4 \text{ in.}^2$$

Use no. 4 bars spaced at 6 in. ($A_s = 0.4$ in.^{2}). For 5-ft-wide stairs, use 10 no. 4 bars.

$$d_t = 7 \text{ in.} \qquad c = 0.92 \text{ in.}$$

Net tensile strain,

$$\varepsilon_t = \left(\frac{d_t - c}{c}\right) = 0.0198 \text{ in.}$$

$$\varepsilon_t > 0.005 \qquad \phi = 0.9$$

c. Transverse reinforcement must be provided to account for shrinkage.

$$A_s = 0.0018 \times 12 \times 8 = 0.18 \text{ in.}^2/\text{ft}$$

Use no. 4 bars spaced at 12 in. ($A_s = 0.2$ in.^{2}).

d. If the slab will be cast monolithically with its supporting beams, additional reinforcement must be provided at the top of the upper and lower landings. Details of stair reinforcement are shown in Fig. 18.19.

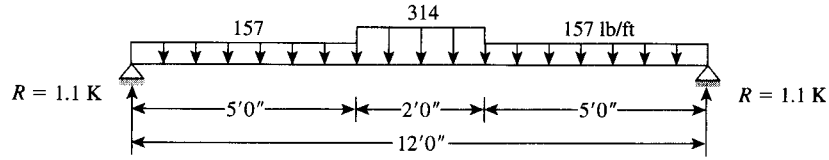


Figure 18.20 Example 18.2: loads on landing.

4. Minimum slab thickness for deflection is

$$\frac{L}{25} = \frac{17.2 \times 12}{25} = 8.26 \text{ in.}$$

(for a simply supported slab). In the case presented here, where the slab ends are cast with the supporting beams and additional negative reinforcement is provided, minimum thickness can be assumed to be

$$\frac{L}{28} = 7.4 \text{ in.} < 8 \text{ in. used}$$

5. Design of landings: Considering a 1-ft length of the landing, the load on the landing is as shown in Fig. 18.20. The middle 2 ft will carry a full load, whereas the two 5-ft lengths on each side will carry half the ultimate load.

$$\text{Maximum bending moment} = (1.1 \times 6) - (0.157 \times 5)(3.5) - (0.314)\frac{(1)^2}{2} = 3.7 \text{ K·ft}$$

Because the bars in the landing will be placed on top of the main stair reinforcement,

$$d = 8.0 - 0.75 - \frac{4}{8} - 0.25 = 6.375 \text{ in.} \qquad \text{say, } 6.3 \text{ in.}$$

Assume $a = 0.4$ in.

$$A_s = \frac{3.7 \times 12}{0.9 \times 60(6.3 - 0.2)} = 0.14 \text{ in.}^2 < A_s(\text{min}) \text{ of } 0.32 \text{ in.}^2$$

Use $A_s = 0.32 \text{ in.}^2$ Use no. 4 bars spaced at 7 in. ($A_s = 0.34 \text{ in.}^2$).

6. The transverse beams at the landing levels must be designed to carry loads from stairs (2.3 K/ft) in addition to their own weight and the weight of the wall above.
7. Check shear as usual.

Example 18.3

Design the simply supported run-riser stairs shown in Fig. 18.21 for a uniform live load of 120 psf. Use $f'_c = 3$ ksi and $f_y = 60$ ksi.

Solution

1. Loads: Assume the thickness of runs and risers is 6 in. The concentrated load at each riser is calculated as follows (refer to Fig. 18.21*b*). Due to dead load per foot depth of run,

$$P_D = \left(\frac{16}{12} \times \frac{6}{12} + \frac{1}{12} \times \frac{6}{12}\right) 150 = 106 \text{ lb}$$

Note that the node dead load on the landing is less than 106 lb but can be assumed to be equal to P_D to simplify calculations. Due to live load peer foot depth of run, $P_L = \frac{10}{12} \times (120) = 100$ lb.

$$\text{Factored load, } P_u = 1.2P_D + 1.6P_L$$
$$= 1.2 \times 106 + 1.6 \times 100 = 290 \text{ lb}$$

2. Calculate the bending moments at midspan: Loads in this example are symmetrical about midspan section B. Reaction at A, R_A is $\frac{1}{2}(15)(290) = 2175$ lb $= (7\frac{1}{2}P \times 290)$

$$\text{Moment at } B = R_A(8S) - 7P_u(4S)$$
$$= 2.175(8 \times 10) - 7(0.29)(4 \times 10) = 92.8 \text{ K·in.}$$

3. Calculate the reinforcement required at midspan section: For $h = 6$ in., $d = 6 - 1.0 = 5.0$ in.,

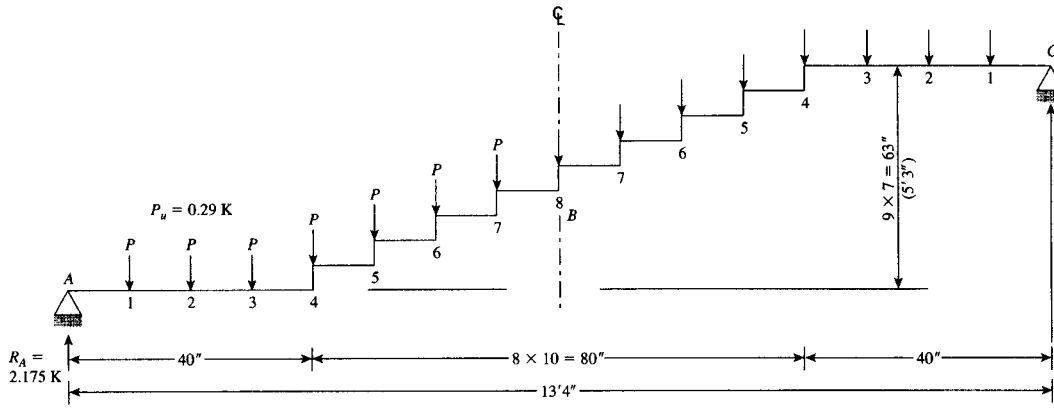
$$R_u = \frac{M_u}{bd^2} = \frac{92.8 \times 1000}{12(5.0)^2} = 309 \text{ psi}$$

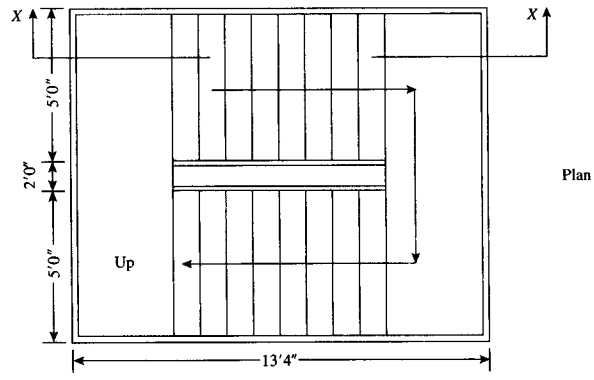
For $f'_c = 3$ ksi, $f_y = 60$ ksi, and $R_u = 309$ psi, the steel ratio is $\rho = 0.0061 < \rho_{\max} = 0.0135$ ($\phi = 0.9$).

$$A_s = 0.0061 \times 12 \times 5.0 = 0.366 \text{ in.}^2$$

Use no. 4 bars spaced at 6 in. ($A_s = 0.39$ in.^{2}) horizontally and vertically in closed stirrup form. For distribution bars, use minimum ρ of 0.0018.

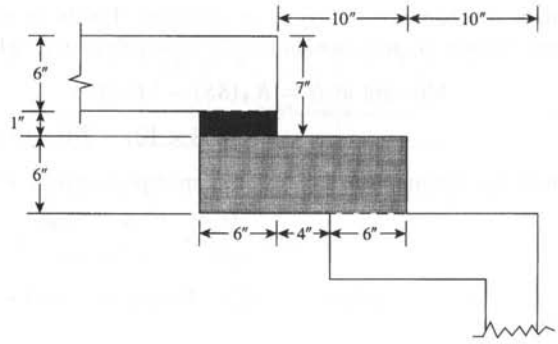
$$A_s = 0.0018 \times 16 \times 6 = 0.18 \text{ in.}^2$$



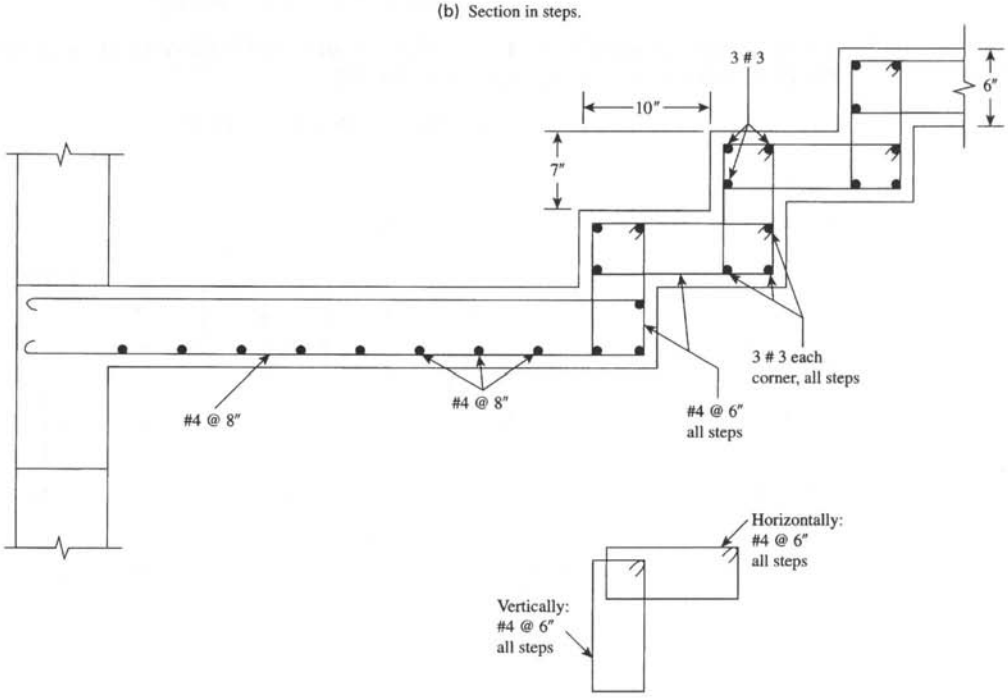


(a) Plan and section X–X

Figure 18.21 Example 18.3.



(b) Section in steps.



(c) Reinforcement details.

Figure 18.21 (*continued*)

Use no. 3 bars spaced at 6 in. ($A_s = 0.22\,\text{in.}^2$). For each step corner, use three no. 3 bars ($A_s = 0.33\,\text{in.}^2$), as shown in Fig. 18.21*c*.

4. The moments and reinforcement required for other sections can be prepared in table form, as follows:

Location	A	1	2	3	4	5	6	7	8
B.M. (K in.)	0	22	41	57	70	80	87	91	92.8
R_u (psi)	0	73	137	190	233	267	290	303	309
ρ (%)	0	0.18	0.26	0.38	0.46	0.52	0.58	0.60	0.61
A_s (in.^{2})	0	0.11	0.16	0.23	0.28	0.31	0.35	0.36	0.37

Use no. 4 bars at 8 in. for the landing and no. 4 bars at 6 in. for the steps. For distribution bars, use minimum ρ of 0.0018. For $A_s = 0.18\,\text{in.}^2$, use no. 4 bars spaced at 8 in. in the landing. Details of reinforcement are shown in Fig. 18.21*c*.

5. Check reinforcement required in the transverse direction of landing: Load per square foot on the landing is $\frac{290}{10} \times 12 = 348$ psf.

$$M_u = \frac{0.348}{8}(12)^2 \times 12 = 75\ \text{K·in.}$$

$$R_u = \frac{75 \times 1000}{12(5.0)^2} = 250\ \text{psi} \qquad \rho = 0.0049 \qquad A_s = 0.29\ \text{in.}^2$$

Use no. 4 bars spaced at 8 in. ($A_s = 0.29\,\text{in.}^2$)

6. If a uniform load is assumed to be acting on the flight of stairs, similar results will be obtained. For example, ultimate node load was calculated to be 290 lb acting over a 10-in. run width. Load per foot is $\frac{290}{10} \times 12 = 348$ lb/ft. Maximum moment is at midspan, section *B*:

$$M_u = \frac{0.348}{8}(13.33)^2 = 92.8\ \text{K·in.}$$

Moments at other sections can be easily calculated, and the design can be arranged in a table form, as explained in step 4.

SUMMARY

Sections 18.1–18.2

The different types of stairs are single and multiple flights, cantilever and precast concrete flights, free-standing and helical staircases, and run-riser stairs.

Section 18.3

Design examples are presented in this section.

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2. A. C. Scordelis, "Internal Forces in Uniformly Loaded Helicoidal Girders". *Journal of the American Concrete Institute* (April 1960).
3. A. R. Cusens and Jing-gwo Kuang. "A Simplified Method of Analyzing Free Standing Stairs". *Concrete & Constructional Engineering* (May 1965): 167–72.
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5. Franz Sauter. "Free Standing Stairs". *Journal of the American Concrete Institute* (July 1961): 847–69.
6. Cusens & Santathadapon. "Design Charts for Helical Stairs with Fixed Supports." London: Concrete Publications Ltd.
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PROBLEMS

18.1 Design a typical flight of the staircase shown in Fig. 18.22, which is a part of a multistory building. The height between the concrete floors is 10 ft (3.0 m). The stairs are supported at the ends of the landings and carry a live load equal to 120 psf (5.75 kN/m^2); $f'_c = 3$ ksi (20 MPa) and $f_y = 60$ ksi (400 MPa)

18.2 Repeat Problem 18.1 if the stairs are supported by four transverse beams at A, B, C, and D and the live load is increased to 150 psf (7.2 kN/m^2).

18.3 The stairs shown in Fig. 18.23 are to be designed for a live load equal to 100 psf (4.8 kN/m). The stairs are supported by beams, as shown. Design the stairs and the supporting beams for $f'_c = 3$ ksi (20 MPa) and $f_y = 60$ ksi (400 MPa).

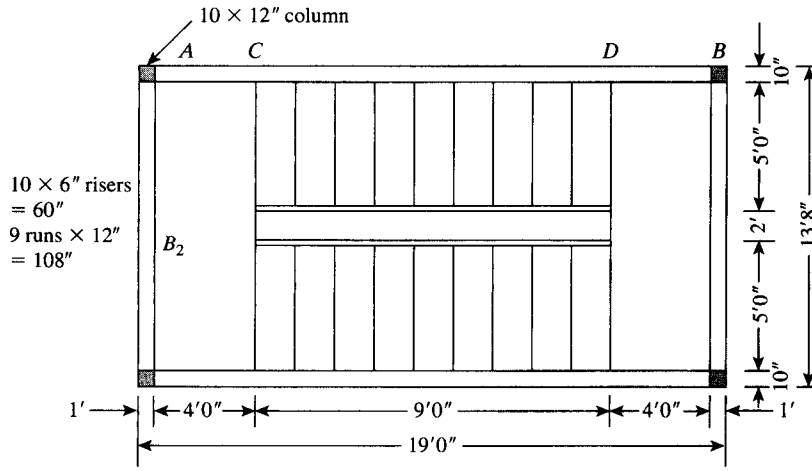


Figure 18.22 Problem 18.1.

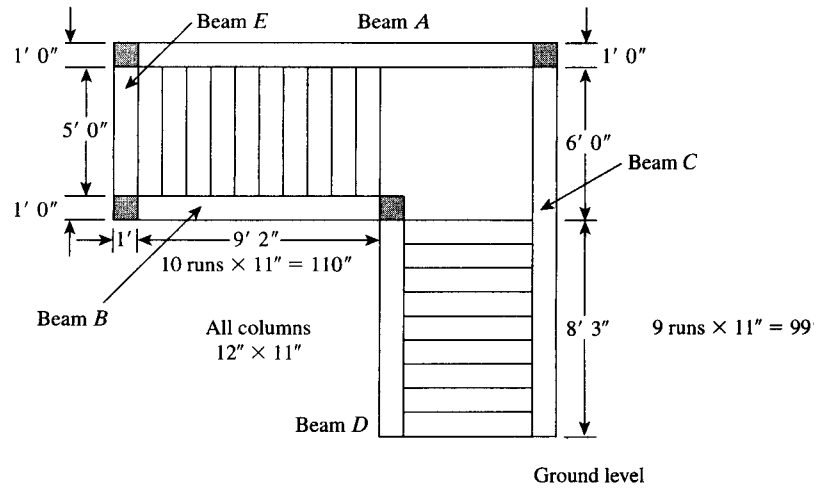


Figure 18.23 Problem 18.3.

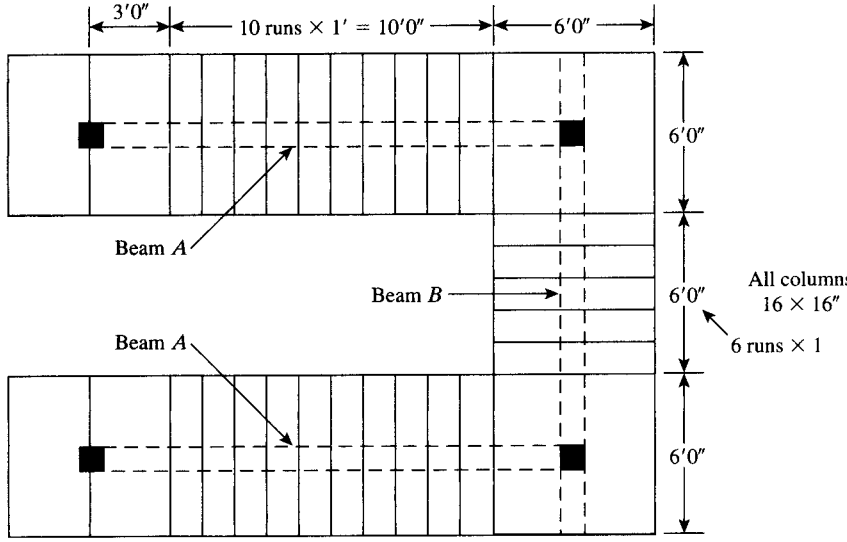


Figure 18.24 Problem 18.4.

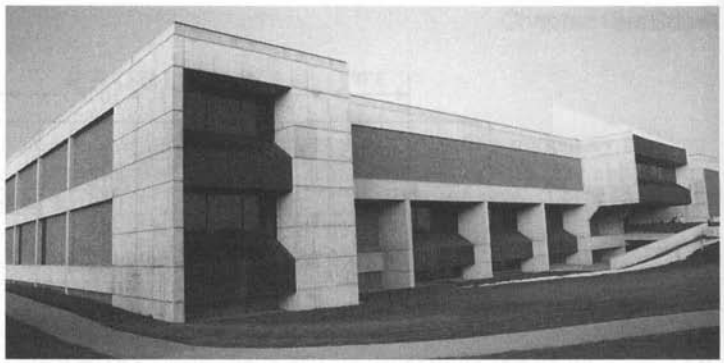
18.4 Design a typical flight of stairs in a public building for the staircase arrangement shown in Fig. 18.24. The stairs are supported by central beams, A and B. Design only one flight and the supporting beams A and B. The runs are 1.0 ft (300 m) deep and the rises are 6.5 in. high. Use $f'_c = 3$ ksi (20 MPa), $f_y = 60$ ksi (400 MPa), and a live load equal to 80 psf (3.85 kN/m^2).
Note : Design the beams for bending moments and shear, and neglect torsional moments caused by loading one-half of the steps.

18.5 Repeat Example 18.3 if the run is 12 in. (300 mm) and the rise is 6 in. (150 mm).

18.6 Repeat Example 18.3 if the landing is 5 ft (6 × 10″), runs are 8.33 ft (10 × 10″), risers at 5.5 ft (11 × 6″), and the live load is 120 psf.

CHAPTER 19

INTRODUCTION TO PRESTRESSED CONCRETE



Library building, South Dakota State University, Brookings, South Dakota.

19.1 PRESTRESSED CONCRETE

19.1.1 Principles of Prestressing

To prestress a structural member is to induce internal, permanent stresses that counteract the tensile stresses in the concrete resulting from external loads; this extends the range of stress that the member can safely withstand. Prestressing force may be applied either before or at the same time as the application of the external loads. Stresses in the structural member must remain, everywhere and for all states of loading, within the limits of stress that the material can sustain indefinitely. The induced stresses, primarily compressive, are usually created by means of hightensile steel tendons, which are tensioned and anchored to the concrete member. Stresses are transferred to the concrete either by the bond along the surface of the tendon or by anchorages at the ends of the tendon.

To explain this discussion, consider a beam made of plain concrete, which has to resist the external gravity load shown in Fig. 19.1*a*. The beam section is chosen with the tensile flexural stress as the critical criterion for design; therefore, an uneconomical section results. This is because concrete is considerably stronger in compression than in tension. The maximum flexural tensile strength of concrete, the modulus of rupture, f_r, is equal to $7.5\lambda\sqrt{f'_c}$ (Fig. 19.1*a*).

In normal reinforced concrete design, the tensile strength of concrete is ignored and steel bars are placed in the tension zone of the beam to resist the tensile stresses, whereas the concrete resists the compressive stresses (Fig. 19.1*b*).

In prestressed concrete design, an initial compressive stress is introduced to the beam to offset or counteract the tensile stresses produced by the external loads (Fig. 19.1*c*). If the induced compressive stress is equal to the tensile stress at the bottom fibers, then both stresses cancel themselves, whereas the compressive stress in the top fibers is doubled; in this case, the whole section is in compression. If the induced compressive stress is less than the tensile stress at the bottom fibers, these fibers will be in tension, whereas the top fibers are in compression.

In practice, a concrete member may be prestressed in one of the following methods.

1. *Posttensioning:* In posttensioning, the steel tendons are tensioned after the concrete has been cast and hardened. Posttensioning is performed by two main operations: tensioning the steel wires or strands by hydraulic jacks that stretch the strands while bearing against the ends of the member and then replacing the jacks by permanent anchorages that bear on

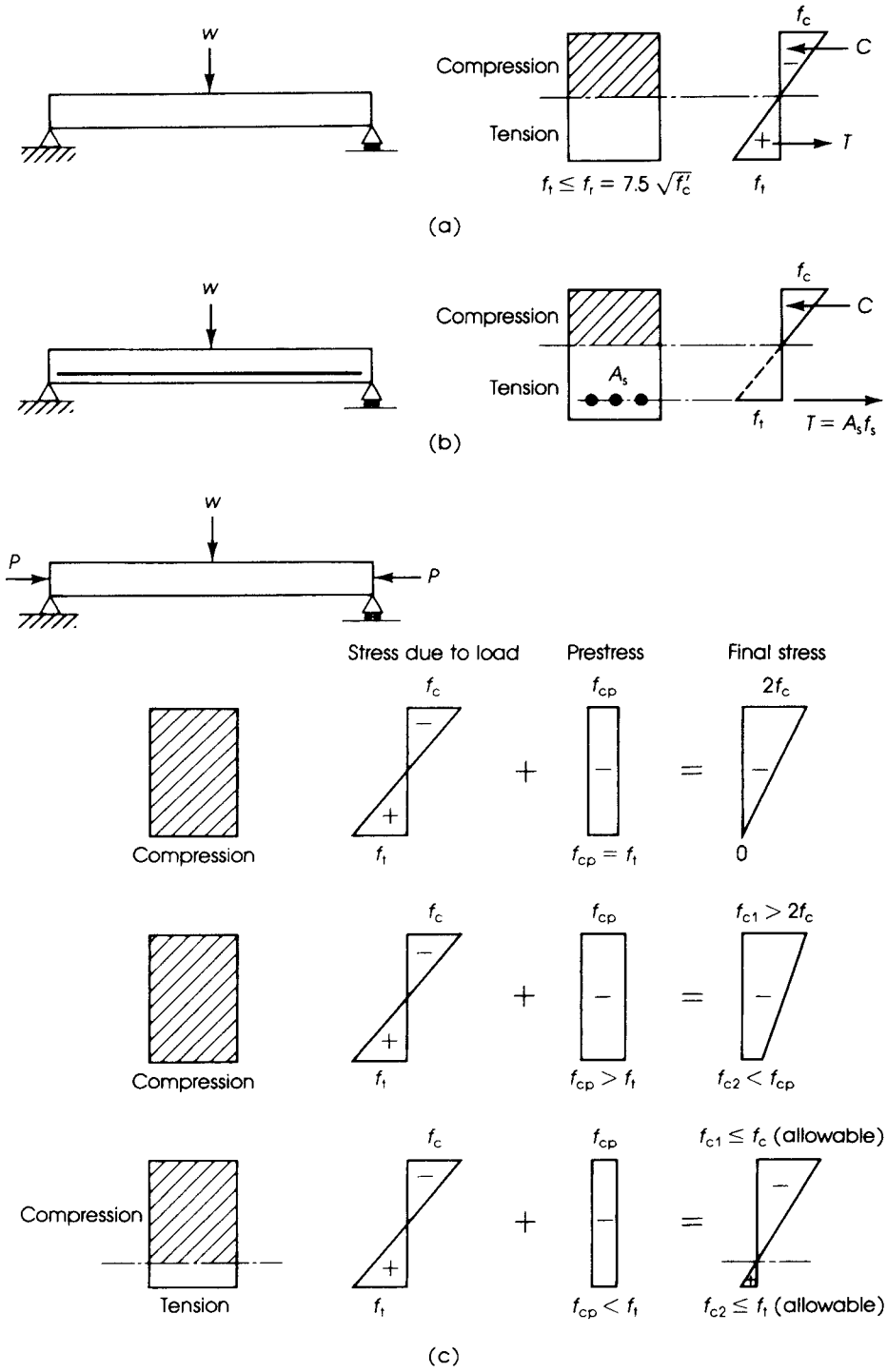


Figure 19.1 Effect of prestressing: (*a*) plain concrete, (*b*) reinforced concrete, and (*c*) prestressed concrete.

the member and maintain the steel strands in tension. A tendon is generally made of wires, strands, or bars. Wires and strands can be tensioned in groups, whereas bars are tensioned one at a time. In the posttensioning process, the steel tendons are placed in the formwork before the concrete is cast and the tendons are prevented from bonding to the concrete by waterproof paper wrapping or a metal duct (sheath). Tendons bonded to the concrete are called bonded tendons. Unbonded tendons, left without grout or coated with grease, have no bond throughout the length of the tendon.

2. *Pretensioning:* In pretensioning, the steel tendons are tensioned before the concrete is cast. The tendons are temporarily anchored against some abutments and then cut or released after the concrete has been placed and hardened. The prestressing force is transferred to the concrete by the bond along the length of the tendon. Pretensioning is generally done in precasting plants in permanent beds, which are used to produce pretensioned precast concrete elements for the building industry.
3. *External prestressing:* In external prestressing, the prestressing force is applied by flat jacks placed between the concrete member ends and permanent rigid abutments. The member does not contain prestressing tendons, as in the previous two methods (also called internal prestressing). External prestressing is not easy in practice because shrinkage and creep in concrete tend to reduce the induced compressive stresses unless the prestressing force can be adjusted.

The profile of the tendons may be straight, curved (bent), or circular, depending on the design of the structural member. Straight tendons are generally used in solid and hollow-cored slabs, whereas bent tendons are used in beams and most structural members. Circular tendons are used in circular structures such as tanks, silos, and pipes. The prestressing force may be applied in one or more stages, either to avoid overstressing concrete or in cases when the loads are applied in stages. In this case, part of the tendons are fully prestressed at each stage.

A considerable number of prestressing systems have been devised, among them Freyssinet, Magnel Blaton, B.B.R.V., Dywidag, CCL, Morandi, VSL, Western Concrete, Prescon, and INRYCO. The choice of the prestressing system for a particular job can sometimes be a problem. The engineer should consider three main factors that govern the choice of the system:

1. The magnitude of the prestressing force required
2. The geometry of the section and the space available for the tendons
3. Cost of the prestressing system (materials and labor)

The following example illustrates some of the features of prestressed concrete.

Example 19.1

For the simply supported beam shown in Fig. 19.2, determine the maximum stresses at midspan section due to its own weight and the following cases of loading and prestressing:

1. A uniform live load of 900 lb/ft
2. A uniform live load of 900 lb/ft and an axial centroidal longitudinal compressive force of $P = 259.2$ K
3. A uniform live load of 2100 lb/ft and an eccentric longitudinal compressive force $P = 259.2$ K acting at an eccentricity $e = 4$ in.
4. A uniform live load of 2733 lb/ft and an eccentric longitudinal compressive force $P = 259.2$ K acting at the maximum practical eccentricity for this section ($e = 6$ in.)

5. The maximum live load when $P = 259.2\,\text{K}$ acting at $e = 6\,\text{in.}$
Use $b = 12\,\text{in.}$, $h = 24\,\text{in.}$, normal-weight concrete with $f'_c = 4500$ psi, and an allowable $f'_c = 2050$ psi.

Solution

1. Stresses due to dead and live loads only are Self-weight of beam $= (1 \times 2) \times 150 = 300\,\text{lb/ft}$

$$\text{Dead-load moment } M_{\text{D.L.}} = \frac{wL^2}{8} = \frac{0.300(24)^2}{8} = 21.6\ \text{K·ft}$$

Stresses at the extreme fibers are

$$\sigma = \frac{Mc}{I} = \frac{M(h/2)}{bh^3/12} = \frac{6M}{bh^2}$$

$$\sigma_D = \frac{6 \times 21.6 \times 12{,}000}{12(24)^2} = \pm 225\ \text{psi}$$

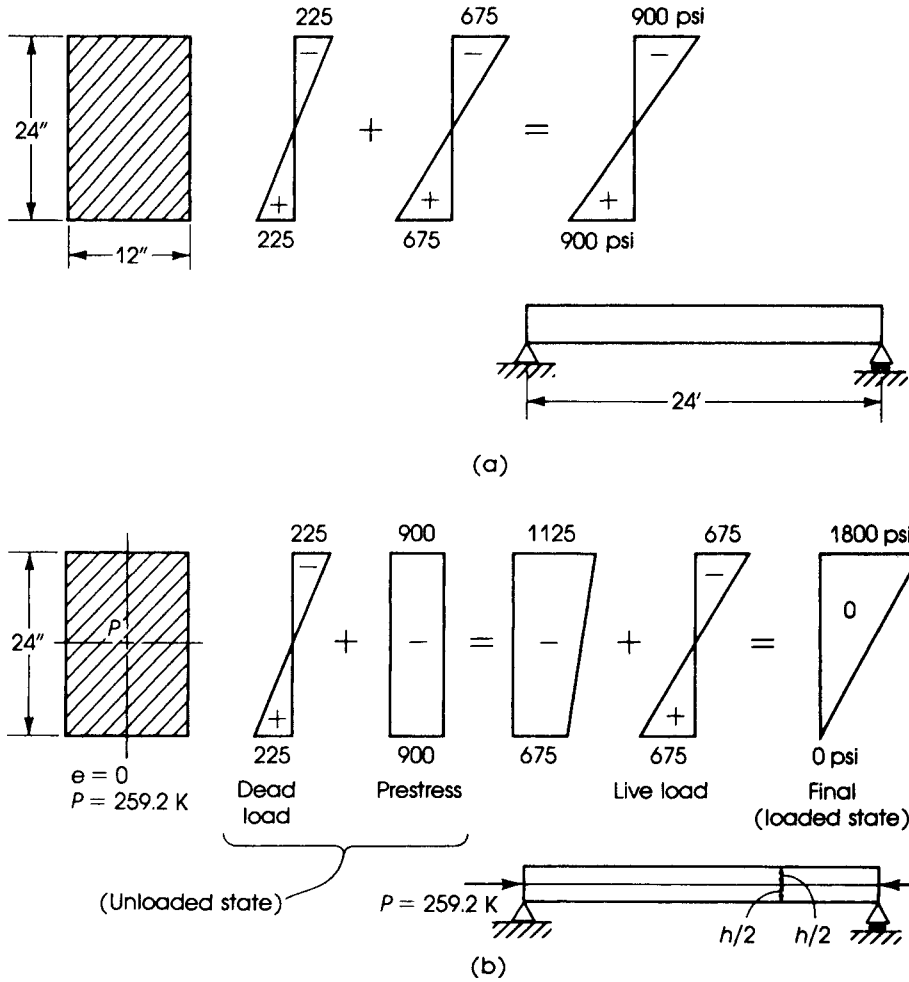
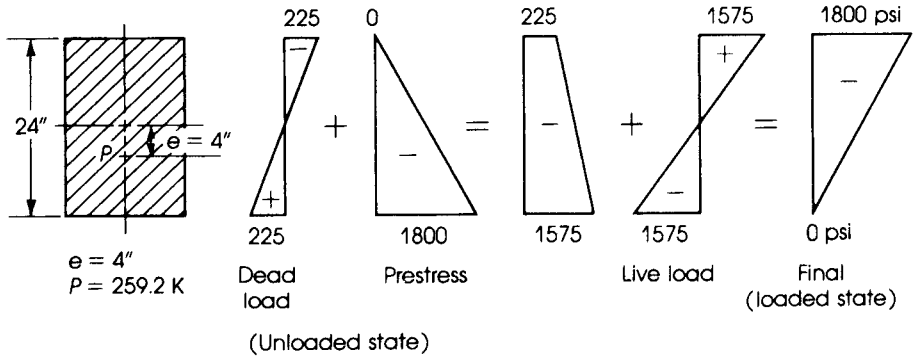
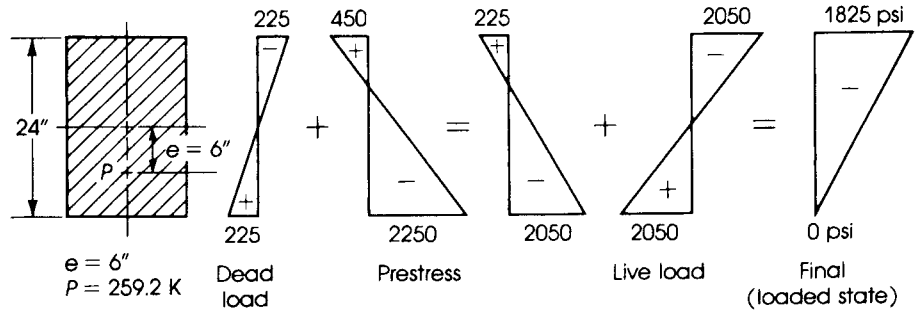


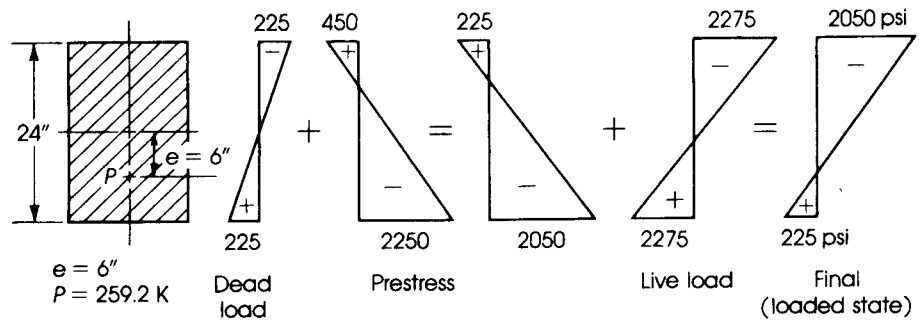
Figure 19.2 Example 19.1.



(c)



(d)



(e)

Figure 19.2 (*continued*)

Stresses due to the live load $L_1 = 900$ lb/ft are

$$M_{\text{L.L.}} = \frac{0.9(24)^2}{8} = 64.8 \text{ K·ft}$$

$$\sigma_{L_1} = \frac{6M}{bh^2} = \frac{6 \times 64.8 \times 12{,}000}{12(24)^2} = \pm 675 \text{ psi}$$

Adding stresses due to the dead and live loads (Fig. 19.2*a*) gives

$$\text{Top stress} = -225 - 675 = -900 \text{ psi} \quad \text{(compression)}$$

$$\text{Bottom stress} = +225 + 675 = +900 \text{ psi} \quad \text{(tension)}$$

The tensile stress is higher than the modulus of rupture of concrete, $f_r = 7.5\lambda\sqrt{f'_c} = 503$ psi; hence, the beam will collapse.

2. In the case of stresses due to uniform prestress, if a compressive force $P = 259.2$ K is applied at the centroid of the section, then a uniform stress is induced on any section along the beam.

$$\sigma_P = \frac{P}{\text{area}} = \frac{259.2 \times 1000}{12 \times 24} = -900 \text{ psi} \quad \text{(compression)}$$

Final stresses due to live and dead loads plus prestress load at the top and bottom fibers are 1800 psi and 0, respectively (Fig. 19.2*b*). In this case, the prestressing force has doubled the compressive stress at the top fibers and reduced the tensile stress at the bottom fibers to 0. The maximum compressive stress of 1800 psi is less than the allowable stress of 2050 psi.

3. For stresses due to an eccentric prestress ($e = 4$ in.), if the prestressing force $P = 259.2$ K is placed at an eccentricity of $e = 4$ in. below the centroid of the section, the stresses at the top and bottom fibers are calculated as follows. Moment due to eccentric prestress is Pe:

$$\begin{aligned}\sigma_P &= -\frac{P}{A} \pm \frac{(Pe)c}{I} = -\frac{P}{A} \pm \frac{6(Pe)}{bh^2} \\ &= -\frac{259.2 \times 1000}{12 \times 24} \pm \frac{6(259.2 \times 1000 \times 4)}{12(24)^2} \\ &= -900 \pm 900 \\ &= -1800 \text{ psi}\end{aligned}$$

at the bottom fibers and $\sigma_P = 0$ at the top fibers. Consider now an increase in the live load of $L_2 = 2100$ lb/ft:

$$M_{\text{L.L.}} = \frac{2.1 \times (24)^2}{8} = 151.2 \text{ K·ft}$$

$$\sigma_{L_2} = \frac{6(151.2 \times 12{,}000)}{12(24)^2} = \pm 1575 \text{ psi}$$

Final stresses due to the dead, live, and prestressing loads at the top and bottom fibers are 1800 psi and 0, respectively (Fig. 19.2*c*). Note that the final stresses are exactly the same as those of the previous case when the live load was 900 lb/ft; by applying the same prestressing force but at an eccentricity of 4 in., the same beam can now support a greater live load (by 1200 lb/ft).

4. For stresses due to eccentric prestress with maximum eccentricity, assume that the maximum practical eccentricity for this section is at $e = 6$ in., leaving a 2-in. concrete cover; then the bending moment induced is $Pe = 259.2 \times 6 = 1555.2$ K·in. $= 129.6$ K·ft. Stresses due to the prestressing force are

$$\begin{aligned}\sigma_P &= -\frac{259.2 \times 1000}{12 \times 24} \pm \frac{6 \times (129.6 \times 12{,}000)}{12(24)^2} \\ &= -900 \pm 1350 \text{ psi} \\ &= -2250 \text{ psi} \quad \text{and} \quad +450 \text{ psi}\end{aligned}$$

Increase the live load now to $L_3 = 2733$ lb/ft. The stresses due to the live load, L_3, are

$$M_{\text{L.L.}} = \frac{2.733 \times (24)^2}{8} = 196.8 \text{ K·ft}$$

$$\sigma_{L_3} = \frac{6(196.8 \times 12{,}000)}{12(24)^2} = \pm 2050 \text{ psi}$$

The final stresses at the top and bottom fibers due to the dead load, live load (L_3), and the prestressing force are 1825 psi and 0, respectively (Fig. 19.2*d*). Note that the final stresses are about the same as those in the previous cases, yet the live load has been increased to 2733 lb/ft. A tensile stress of 225 psi is developed when the prestressing force is applied on the beam. This stress is less than the modulus of rupture of concrete, $f_r = 503$ psi; hence, cracks will not develop in the beam.

5. The maximum live load when the eccentric force P acts at $e = 6$ in. is determined as follows. In the previous case, the final compressive stress is equal to 1825 psi, which is less than the allowable stress of 2050 psi. Therefore, the live load may be increased to $L_4 = 3033$ lb/ft.

$$M_{\text{L.L.}} = \frac{3.033 \times (24)^2}{8}$$

$$\sigma_{L_4} = \frac{6(218.4 \times 12{,}000)}{12(24)^2} = \pm 2275 \text{ psi}$$

Final stresses due to the dead load, live load (L_4), and the prestressing force are -2050 psi and ± 225 psi (Fig. 19.2*e*). The compressive stress is equal to the allowable stress of 2050 psi, and the tensile stress is less than the modulus of rupture of concrete of 503 psi. In this case, the uniform live load of 3033 lb/ft has been calculated as follows: Add the maximum allowable compressive stress of 2050 psi to the initial tensile stress at the top fibers of 225 psi to get 2275 psi. The moment that will produce a stress at the top fibers of 2275 psi is equal to

$$M = \sigma\left(\frac{bh^2}{6}\right)$$

$$= \frac{2.275}{6}(12)(24)^2 = 2620.8 \text{ K·in.} = 218.4 \text{ K·ft}$$

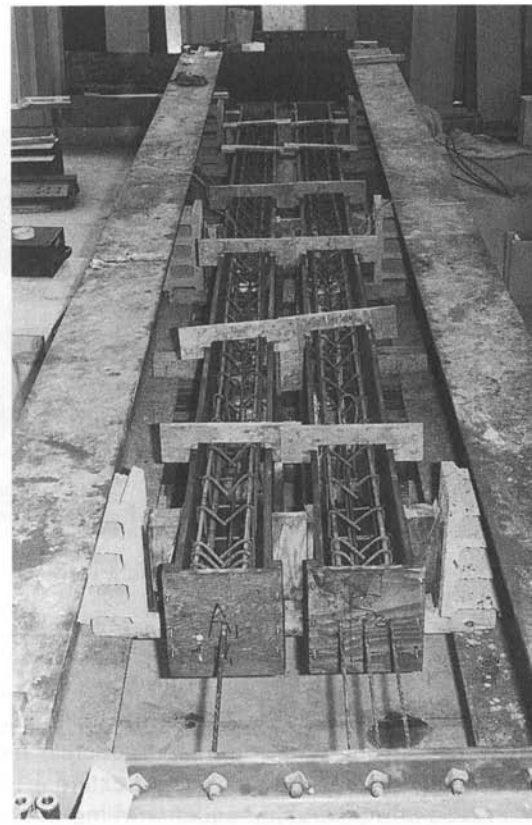
$$M = \frac{W_L L^2}{8} \quad \text{and} \quad W_L = \frac{8 \times 218.4}{(24)^2} = 3.033 \text{ K/ft}$$

Notes:

1. The entire concrete section is active in resisting the external loads.
2. The final tensile stress in the section is less than the modulus of rupture of concrete, which indicates that a crackless concrete section can be achieved under full load.
3. The allowable load on the beam has been increased appreciably due to the application of the prestressing force.
4. An increase in the eccentricity of the prestressing force will increase the allowable applied load, provided that the allowable stresses on the section are not exceeded.

19.1.2 Partial Prestressing

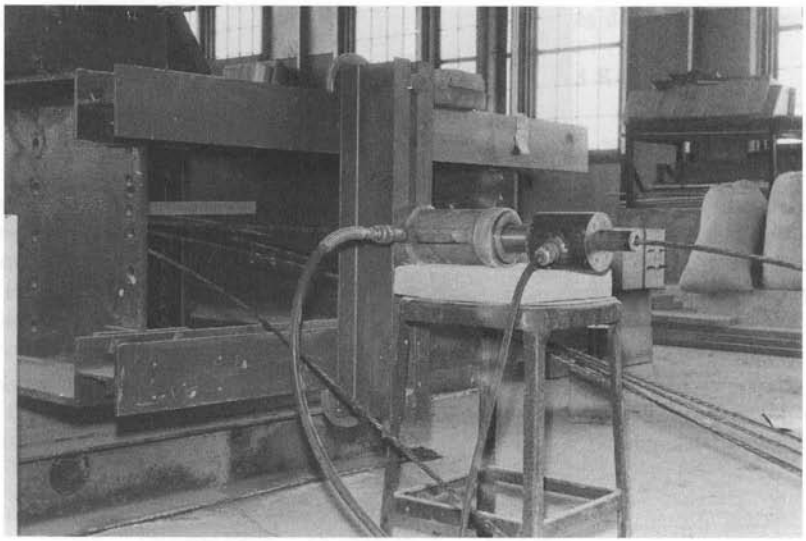
A partially prestressed concrete member can be defined as one in which (1) there have been introduced internal stresses to counteract part of the stresses resulting from external loadings, (2) tensile stresses are developed in the concrete under working loads, and (3) nonprestressed reinforcement may be added to increase the moment capacity of the member. That definition implies that there are two cases that could be considered as partially prestressed concrete:



Partially prestressed concrete beams.

1. A combination of prestressed and nonprestressed steel is used in the same section. The prestressed cables induce internal stresses designed to take only part of the ultimate capacity of the concrete section. The rest of the capacity is taken by nonprestressed steel placed along the same direction as the prestressed cables. The steel used as nonprestressed steel could be any common grade of carbon steel or high-tensile-strength steel of the same kind as the prestressing cables with ultimate strength of 250 ksi (1725 N/mm^2). The choice depends on two main factors: allowable deflection and allowable crack width. As for deflection, the ACI Code specifies a maximum ratio of span to depth of reinforced concrete members. With the smaller depth expected in partially prestressed concrete, and because a smaller steel percentage is used, excessive deflection under working loads must not be allowed. Cracks develop on the tension side of the concrete section or at the steel level because tensile stresses are allowed to occur under working loads. The maximum crack width that may be allowed is 0.016 in. (0.41 mm) for interior members and 0.013 in. (0.33 mm) for exterior members.
2. Internal stresses act on the member from prestressed steel only, but tensioned to a lower limit. In this case cracking develops earlier than in a fully prestressed member under similar loadings.

Partially prestressed concrete can be considered an intermediate form between reinforced and fully prestressed concrete. In reinforced concrete members, cracks develop under working loads; therefore, reinforcement is placed in the tension zone. In prestressed concrete members,



Prestressing jack with a load cell.

cracks do not usually develop under working loads. The compressive stresses due to prestressing may equal or exceed the tensile stresses due to external loadings. Therefore, a partially prestressed concrete member may be considered a reinforced concrete member in which internal stresses are introduced to counteract part of the stress from external loadings so that tensile stresses in the concrete do not exceed a limited value under working load. It reduces to reinforced concrete when no internal stresses act on the member. Full prestressing is an upper extreme of partial prestressing in which nonprestressed reinforcing steel reduces to 0.

Between a reinforced cracked member and a fully prestressed uncracked member, there exists a wide range of design in partial prestressing (Fig. 19.3). A proper choice of the degree of prestressing will produce a safe and economical structure.

Figure 19.3 shows the load deflection curves of concrete beams containing different amounts and types of reinforcement. Curve *a* represents a reinforced concrete beam, which normally cracks at a small load W_{cr}. The cracking moment M_{cr} can be determined as follows:

$$M_{cr} = \frac{f_r I}{c}$$

where

f_r = the modulus of rupture of concrete = $7.5\lambda\sqrt{f'_c}$
I = moment of inertia of the gross concrete section
c = distance from the neutral axis to the tensile extreme fibers

The cracking load can be determined from the cracking moment when the span and the type of loading are specified. For a simply supported beam subjected to a concentrated load at midspan, $W_{cr} = (4M_{cr})/L$.

Curves *e* and *f* represent underreinforced and overreinforced fully prestressed concrete beams, respectively. The overreinforced concrete beam fails by crushing of the concrete before the steel reaches its yield strength or proof stress. The beam has small deflection and undergoes brittle failure. The under-reinforced beam fails by the steel reaching its yield or ultimate strength.

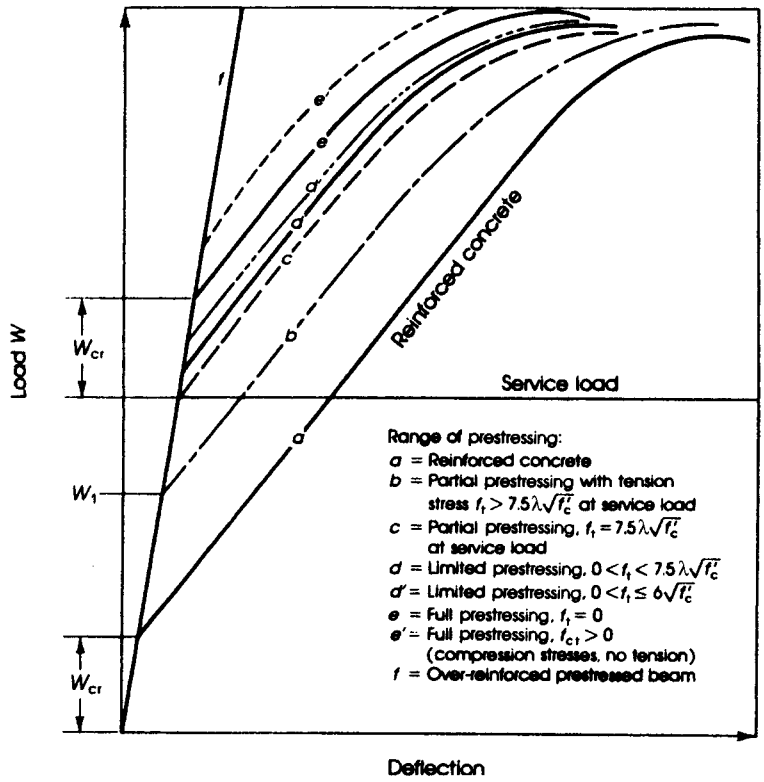


Figure 19.3 Load–deflection curves of concrete beams with different prestressing. The cracking load is W_{cr}.

It shows appreciable deflection and cracking due to elongation of the steel before the gradual crushing of the concrete and the collapse of the beam.

Between curves *a* and *e* is a wide range of concrete beams with varying amounts of reinforcement and subjected to varying amounts of prestress. The beam with little prestressing is closer to curve *a*, while the beam with a large prestress is closer to curve *e*. Depending upon the allowable concrete stress, deflection, and maximum crack width, a suitable combination of prestressed and nonprestressed reinforcement may be chosen for the required design.

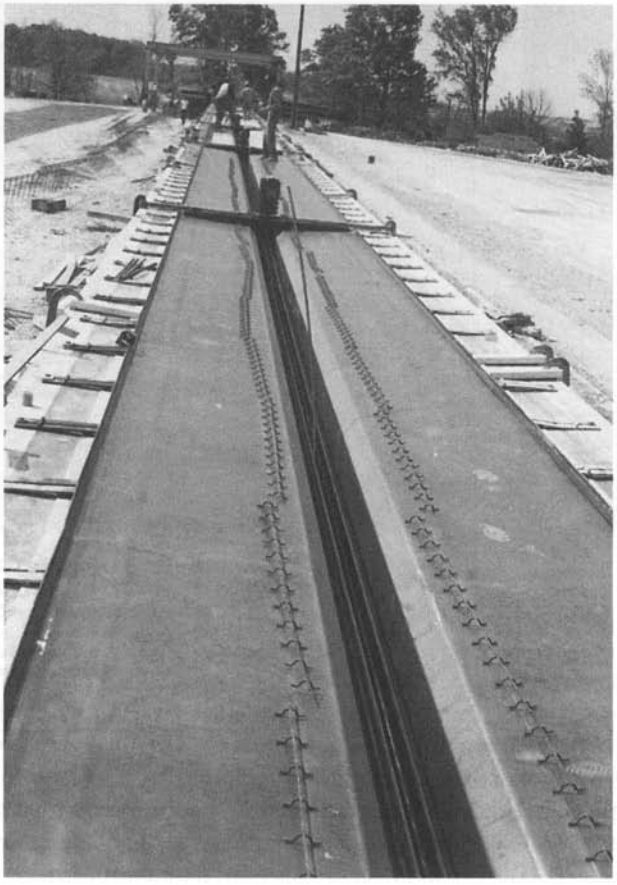
Curve *b* represents a beam that will crack under full working load. If only part of the live load L_1 occurs frequently on the structure, then W_1 represents the total dead load and that part of the live load L_1.

Curve *c* represents a beam that starts cracking at working load. The maximum tensile stress in the concrete $= 7.5\sqrt{f'_c}$.

Curve *d* represents a beam with limited prestress. The critical section of the beam will not crack under full working load, but it will have a maximum tensile stress of $0 < f_t < 7.5\sqrt{f'_c}$. The maximum tensile stress in concrete allowed by the current ACI Code is $6\sqrt{f'_c}$.

Curves *e* and *e'* represent fully prestressed concrete beams with no tensile stress under working loads. (See Fig. 19.4.)

The most important advantage of partial prestressing is the possibility of controlling camber. By reducing the prestressing force, the camber will be reduced and a saving in the amount of the prestressing steel, the amount of work in tensioning, and the number of end anchorages is realized.



Prestressing bed for T-beam sections.

Depending on the magnitude of the prestressing force, earlier cracking may occur in partially prestressed rather than in fully prestressed concrete members under service loads. Once cracks develop, the effective moment of inertia of the critical section is reduced and a greater deflection is expected. However, partial prestressing has been used with satisfactory results, and its practical application is increasing.

19.1.3 Classification of Prestressed Concrete Flexural Members

The ACI Code, Section 18.3.3, divided prestressed concrete members into three classes based on the computed extreme tensile fiber stress, f_t, in the tension zone at service load as follows:

1. Class U (uncracked section), with $f_t \leq 7.5\sqrt{f'_c}$. In this uncracked concrete section, the gross section properties are used to check deflection at service load. No cracks will develop in this section and no skin reinforcement is needed.
2. Class T (section in the transition zone), with $7.5\sqrt{f'_c} < f_t \leq 12\sqrt{f'_c}$. This type of sections has a tensile stress in concrete higher than the modulus of rupture of concrete, $f_r = 7.5\sqrt{f'_c}$ producing a case between uncracked and cracked sections. In this case, the gross section

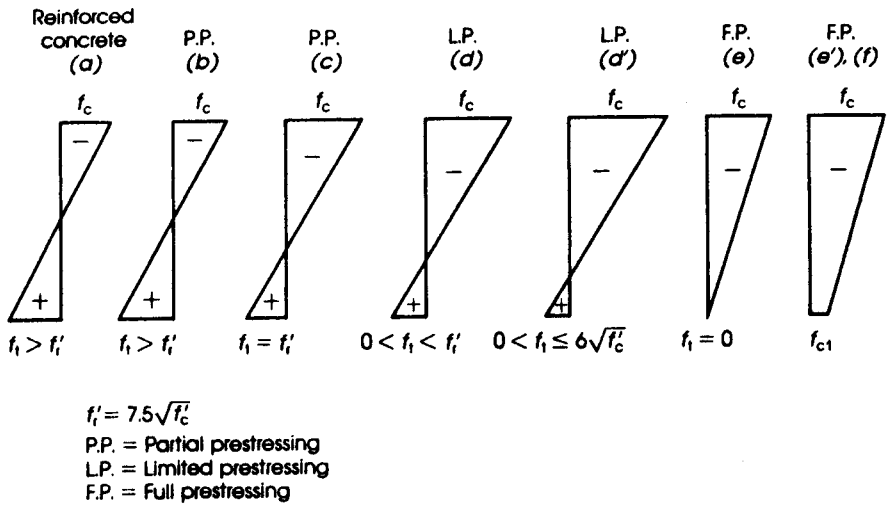


Figure 19.4 Distribution of stresses in beams with varying amounts of prestressed and nonprestressed reinforcement.

properties are used to check stresses, while the cracked section bilinear section is used to calculate deflection. No skin reinforcement is needed in the tension zone.

3. Class C (Cracked section), with $f_t > 12\sqrt{f_c'}$. The tensile stress in the section exceeds 1.6 times the modulus of rupture. Therefore, cracks will develop as in the case of partially prestressed concrete members. In this case a cracked section properties should be used to check stresses, cracking, and deflection. Crack control provisions and skin reinforcement should be used as explained in Section 6.7 for reinforced concrete members with the effective depth of $d > 36$ in.

19.2 MATERIALS AND SERVICEABILITY REQUIREMENTS

19.2.1 Concrete

The physical properties of concrete were discussed in Chapter 2. Although reinforced concrete members are frequently made of concrete with a compressive strength of 3 to 5 ksi (21 to 35 MPa), prestressed concrete members are made of higher strength material, usually from 4 to 8 ksi (28 to 56 MPa). High-strength concrete may be adopted for precast, prestressed concrete members where components are prepared under optimum control of mixing concrete, placing, vibrating, and curing.

The allowable stresses in concrete according to the ACI Code, Section 18.4, are as follows.

1. Stresses after prestress transfer and before prestress losses:
   - **a.** Maximum compressive stress of $0.6f_{ci}'$
   - **b.** Maximum compressive stress at ends of simply supported $0.7f_{ci}'$
   - **c.** Maximum tensile stress (experts as permitted below in d) of $3\sqrt{f_{ci}'}$
   - **d.** Maximum tensile stress at the ends of simply supported members of $6\sqrt{f_{ci}'}$ where f_{ci}' is the strength of concrete at transfer

If the maximum tensile stresses are exceeded in c or d, then reinforcement must be provided in the tensile zone to resist the total tensile force in concrete (based on uncracked gross section).

2. Stresses at service loads after all losses (for class U and class T) are as follows: Maximum compressive stress of $0.45f'_c$ due to prestresses plus sustained loads and of $0.6\sqrt{f'_c}$ due to prestress plus total load.
3. These stresses may be exceeded if it is shown by tests or analysis that performance is satisfactory.

19.2.2 Prestressing Steel

The most common type of steel tendons used in prestressed concrete are strands (or cables) made with several wires, usually seven or 19. Wires and bars are also used. The strands and wires are manufactured according to ASTM Standard A421 for uncoated stress-relieved wires and A416 for uncoated seven-wire stress-relieved strands. Properties of prestressing steel are given in Table 19.1.

Table 19.1 Properties of Prestressing Steel, Nominal Diameters, Areas, and Weights

Type	Diameter (in.)	Area (in.^{2})	Weight (lb/ft)	Diameter (mm)	Area (mm^2)	Mass (kg/m)
Seven-wire strand (grade 250)	$\frac{1}{4}$ (0.250)	0.036	0.12	6.350	23.2	0.179
	$\frac{5}{16}$ (0.313)	0.058	0.20	7.950	37.4	0.298
	$\frac{3}{8}$ (0.375)	0.080	0.27	9.525	51.6	0.402
	$\frac{7}{16}$ (0.438)	0.108	0.37	11.125	69.7	0.551
	$\frac{1}{2}$ (0.500)	0.144	0.49	12.700	92.9	0.729
	(0.600)	0.216	0.74	15.240	139.4	1.101
Seven-wire strand (grade 270)	$\frac{3}{8}$ (0.375)	0.085	0.29	9.525	54.8	0.432
	$\frac{7}{16}$ (0.438)	0.115	0.40	11.125	74.2	0.595
	$\frac{1}{2}$ (0.500)	0.153	0.53	12.700	98.7	0.789
	(0.600)	0.215	0.74	15.250	138.7	1.101
Prestressing wire grades (250)	0.192	0.029	0.10	4.877	18.7	0.146
(250)	0.196	0.030	0.10	4.978	19.4	0.149
(240)	0.250	0.049	0.17	6.350	31.6	0.253
(235)	0.276	0.060	0.20	7.010	38.7	0.298
Prestressing bars (smooth)	$\frac{3}{4}$ (0.750)	0.44	1.50	19.050	283.9	2.232
(grade 145 or 160)	$\frac{7}{8}$ (0.875)	0.60	2.04	22.225	387.1	3.036
	1 (1.000)	0.78	2.67	25.400	503.2	3.973
	$1\frac{1}{8}$ (1.125)	0.99	3.38	28.575	638.7	5.030
	$1\frac{1}{4}$ (1.250)	1.23	4.17	31.750	793.5	6.206
	$1\frac{3}{8}$ (1.385)	1.48	5.05	34.925	954.8	7.515
Prestressing bars (deformed)	$\frac{5}{8}$ (0.625)	0.28	0.98	15.875	180.6	1.458
(grade 150–160)	$\frac{3}{4}$ (0.750)	0.42	1.49	19.050	271.0	2.218
	1 (1.000)	0.85	3.01	25.400	548.4	4.480
	$1\frac{1}{4}$ (1.250)	1.25	4.39	31.750	806.5	6.535
	$1\frac{3}{8}$ (1.385)	1.58	5.56	34.925	1006	8.274



Seven-wires prestressing strands (shipped in coils as shown).

Prestressing steel used in prestressed concrete must be of high-strength quality, usually of ultimate strength, f_{pu}, of 250 ksi to 270 ksi (1730–1860 MPa). High-strength steel is necessary to permit high elongation and to maintain a permanent sufficient prestress in the concrete after the inelastic shortening of the concrete.

The allowable stresses in prestressing steel according to the ACI Code, Section 18.5, are as follows:

1. Maximum stress due to tendon jacking force must not exceed the smaller of $0.8f_{pu}$ or $0.94f_{py}$. (The smaller value must not exceed that stress recommended by the manufacturer of tendons or anchorages.)
2. Maximum stress in pretensioned tendons immediately after transfer must not exceed the smaller of $0.74f_{pu}$ or $0.82f_{py}$.
3. Maximum stress in posttensioning tendons after tendon is anchored is $0.70f_{pu}$.

19.2.3 Reinforcing Steel

Nonprestressed reinforcing steel is commonly used in prestressed concrete structural members, mainly in the prestressed, precast concrete construction. The reinforcing steel is used as shear reinforcement, as supplementary reinforcement for transporting and handling the precast elements, and in combination with the prestressing steel in partially prestressed concrete members. The types and allowable stresses of reinforcing bars were discussed in Chapters 2 and 5.

19.3 LOSS OF PRESTRESS

19.3.1 Lump-Sum Losses

Following the transfer of the prestressing force from the jack to the concrete member, a continuous loss in the prestressing force occurs; the total loss of prestress is the reduction in the prestressing force during the lifespan of the structure. The amount of loss in tendon stress varies between 15% and 30% of the initial stress, because it depends on many factors. For most normal-weight concrete structures constructed by standard methods, the tendon stress loss due to elastic shortening, shrinkage, creep, and relaxation of steel is about 35 ksi (241 MPa) for

pretensioned members and 25 ksi (172 MPa) for posttensioned members. Friction and anchorage slip are not included.

Two current recommendations for estimating the total loss in prestressed concrete members are presented by AASHTO and the Posttensioning Institute (PTI). AASHTO [23] recommends a total loss (excluding friction loss) of 45 ksi (310 MPa) for pretensioned strands and 33 ksi (228 MPa) for posttensioned strands and wires when a concrete strength of $f'_c = 5$ ksi is used. The PTI [24] recommends a total lump-sum prestress loss for posttensioned members of 35 ksi (241 MPa) for beams and 30 ksi (207 MPa) for slabs (excluding friction loss). These values can be used unless a better estimate of the prestress loss by each individual source is made, as is explained shortly.

In general, the sources of prestress loss are

- Elastic shortening of concrete
- Shrinkage of concrete
- Creep of concrete
- Relaxation of steel tendons
- Friction
- Anchorage set

19.3.2 Loss Due to Elastic Shortening of Concrete

In pretensioned members, estimating loss proceeds as follows. Consider a pretensioned concrete member of constant section and stressed uniformly along its centroidal axis by a force F_o. After the transfer of the prestressing force, the concrete beam and the prestressing tendon shorten by an equal amount, because of the bond between the two materials. Consequently, the starting prestressing force F_o drops to F_i and the loss in the prestressing force is $F_o - F_i$. Also, the strain in the concrete, ε_c, must be equal to the change in the tendon strain, $\Delta\varepsilon_s$. Therefore, $\varepsilon_c = \Delta\varepsilon_s$, or $(f_c/E_c) = (\Delta f_s/E_s)$, and the stress loss due to the elastic shortening is

$$\Delta f_s = \frac{E_s}{E_c} \times f_c = nf_c = \frac{nF_i}{A_c} \approx \frac{nF_o}{A_c} \tag{19.1}$$

where

A_c = the area of the concrete section
$n = E_s/E_c$ = modular ratio
f_c = the stress in the concrete at the centroid of the prestressing steel

Multiply the stress by the area of the prestressing steel, A_{sp}, to get the total force; then the elastic loss is

$$ES = F_o - F_i = \Delta f_s A_{sp} = (nf_c)A_{sp} \approx \left(\frac{nF_o}{A_c}\right) A_{sp} \tag{19.2}$$

$$F_i = F_o - (nf_c)A_{sp} \tag{19.3}$$

For practical design, the loss in the prestressing force, Δf_s per unit A_{sp}, may be taken to be approximately nF_o/A_c. If the force F_o acts at an eccentricity e, then the elastic loss due to the presence of F_o and the applied dead load at transfer is

$$ES = -(nf_c)A_{sp} \text{ (due to prestress)} + (nf_c)A_{sp} \quad \text{(dead load)}$$

$$ES = F_o - F_i = -\left(\frac{F_i}{A} + \frac{F_i e^2}{I}\right) nA_{sp} + \left(\frac{M_D e}{I}\right) nA_{sp} \tag{19.4}$$

An approximate value of $F_i = (0.63 f_{pu}) A_{sp}$ may be used in the above equation.

$$F_o + f_c(\text{D.L.}) n A_{sp} = F_i \left[1 + n A_{sp} \left(\frac{1}{A} + \frac{e^2}{I} \right) \right] \tag{19.5}$$

$$F_i = \frac{F_o + (n A_{sp}) f_c(\text{D.L.})}{1 + (n A_{sp}) \left(\frac{1}{A} + \frac{e^2}{I} \right)}$$

For posttensioned members where the tendons or individual strands are not stressed simultaneously, the loss of the prestress can be taken as half the value *ES* for pretensioned members.

Also, it is practical to consider the elastic shortening loss in slabs equal to one-quarter of the equivalent pretensioned value, because stretching of one tendon will have little effect on the stressing of the other tendons.

19.3.3 Loss Due to Shrinkage

The loss of prestress due to shrinkage is time dependent. It may be estimated as follows:

$$SH = \Delta f_s \text{ (shrinkage)} = \varepsilon_{sh} E_s \tag{19.6}$$

where $E_s = 29 \times 10^6$ psi and ε_{sh} = shrinkage strain in concrete.

The average strain due to shrinkage may be assumed to have the following values: for pretensioned members, $\varepsilon_{sh_1} = 0.0003$; for posttensioned members, $\varepsilon_{sh_2} = 0.0002$. If posttensioning is carried out within 5 to 7 days after concreting, the shrinkage strain can be taken to be $0.8\varepsilon_{sh_1}$. If posttensioning is carried out between 1 and 2 weeks, $\varepsilon_{sh} = 0.7\varepsilon_{sh_1}$ can be used, and if it occurs more than 2 weeks later, $\varepsilon_{sh} = \varepsilon_{sh2}$ can be adopted. Shrinkage loss, *SH*, can also be estimated as follows [28]:

$$SH = 8.2 \times 10^{-6} K_{sh} E_s \left(1 - \frac{0.06V}{S} \right) (100 - RH)$$

where V/S = volume-to-surface ratio and RH = average relative humidity. K_{sh} is 1.0 for pretensioned members and is 0.8, 0.73, 0.64, and 0.58 for posttensioned members if posttensioning is carried out after 5, 10, 20, and 30 days, respectively.

19.3.4 Loss Due to Creep of Concrete

Creep is a time-dependent deformation that occurs in concrete under sustained loads. The developed deformation causes a loss of prestress from 5% to 7% of the applied force.

The creep strain varies with the magnitude of the initial stress in the concrete, the relative humidity, and time. The loss in stress due to creep can be expressed as follows:

$$CR = \Delta f_s \text{ (creep)} = C_c(n f_c) = C_c(\varepsilon_{cr} E_s) \tag{19.7}$$

where

$$C_c = \text{creep coefficient} = \frac{\text{creep strain, } \varepsilon_{cp}}{\text{initial elastic strain, } \varepsilon_i}$$

The value of C_c may be taken as follows 22.

Concrete strength	$f'_c \le 4$ ksi		$f'_c > 4$ ksi	
Relative humidity	%	50%	100%	50%
C_c	1–2	2–4	0.7–1.5	1.5–3

Linear interpolation can be made between these values. Considering that half the creep takes place in the first 134 months of the first 6 months after transfer and under normal humidity conditions, the creep strain can be assumed for practical design as follows:

1. For pretensioned members, $\varepsilon_{cr} = 48 \times 10^{-5}$ stress in concrete (ksi).
2. For posttensioned members, $\varepsilon_{cr} = 36 \times 10^{-5} \times$ stress in concrete (ksi). This value is used when posttensioning is made within 2 to 3 weeks. For earlier posttensioning, an intermediate value may be used.

These values apply when the strength of concrete at transfer is $f'_{ci} \geq 4$ ksi. When $f'_{ci} < 4$ ksi, the creep strain should increase in the ratio of (4/actual strength).

$$\text{Total loss of prestress due to creep} = \varepsilon_{cr} E_s \tag{19.8}$$

19.3.5 Loss Due to Relaxation of Steel

Relaxation of steel causes a time-dependent loss in the initial prestressing force, similar to creep in concrete. The loss due to relaxation varies for different types of steel; its magnitude is usually furnished by the steel manufacturers. The loss is generally assumed to be 3% of the initial steel stress for posttensioned members and 2% to 3% for pretensioned members. If test information is not available, the loss percentages for relaxation at 1000 h can be assumed as follows:

1. In low-relaxation strands, when the initial prestress is $0.7f_{pu}$ and $0.8f_{pu}$, relaxation (RE) is 2.5% and 3.5%, respectively.
2. In stress-relieved strands or wire, when the initial prestress is $0.7f_{pu}$ or $0.8f_{pu}$, relaxation (RE) is 8% and 12%, respectively.

19.3.6 Loss Due to Friction

With pretensioned steel, friction loss occurs when wires or strands are deflected through a diaphragm. This loss is usually small and can be neglected. When the strands are deflected to follow a concordant profile, the friction loss may be considerable. In such cases, accurate load measuring devices are commonly used to determine the force in the tendon.

With posttensioned steel, the effect of friction is considerable because of two main factors: the curvature of the tendon and the lack of alignment (wobble) of the duct. The curvature effect may be visualized if a belt around a fixed cylinder is tensioned on one end with a force P_2; then the force, P_1, at the other end to initiate slippage in the direction of P_1 is

$$P_1 = P_2 e^{\mu\alpha_{px}} \tag{19.9}$$

where μ = the coefficient of static angular friction and α_{px} = the angle between P_1 and P_2. It is a general practice to treat the wobbling effect similarly:

$$\begin{aligned} P_x &= P_s e^{-(\mu\alpha + Kl_x)} \\ P_{px} &= P_{px} e^{+(Kl_{px} + \mu_p \alpha_{px})} \\ P_{px} &= P_{pj} e^{-(Kl_{px} + \mu_p \alpha_{px})} \end{aligned} \tag{19.10}$$

where

P_{pj} = the prestressing tendon force at any point x
P_{px} = the prestressing tendon force at the jacking end
μ_p = curvature friction coefficient

α_{px} = total angular change of prestressing tendon profile, in radians, from tendon jacking end to any point x

$$= \frac{\text{length of curve}}{\text{radius of curvature}}$$

K = wobble friction coefficient per foot of the prestressing tendon

As an approximation, the ACI Code gives the following expression:

$$P_{px} = P_{pj}(1 + Kl_{px} + \mu_p\alpha_{px})^{-1} \quad \text{(ACI Code, Eq. 18.2)} \tag{19.11}$$

provided that $(\mu_p\alpha_{px} + Kl_x) \leq 0.30$.

The frictional coefficients α and K depend on the type of prestressing strands or wires, type of duct, and the surface conditions. Some approximate values for μ and K are given in the ACI Code, Section R.18.6.2, and in Table 19.2.

Friction loss in the jack is variable and depends on many factors, including the length of travel of the arm over a given load range. The use of accurate load cells to measure directly the force in the tendon is recommended. The use of pressure gauges may lead to inaccuracies unless they are calibrated against a known force in the tendon.

The friction loss in the anchorage is dependent mainly upon the type of anchorage and the amount of deviation of the tendon as it passes through the anchorage. This loss is usually small and may be neglected. Guidance in particular cases should be obtained from the manufacturers.

19.3.7 Loss Due to Anchor Set

When the force in a tendon is transferred from the jack to the anchorage unit, a small inward movement of the tendon takes place due to the seating of the gripping device or wedges. The slippage causes a shortening of the tendon, which results in a loss in the prestressing force. The magnitude of slippage varies between 0.1 and 0.25 in. (2.5 and 6 mm) and is usually specified by the manufacturer. The loss due to the anchor set may be calculated as follows:

$$\Delta f_s = \Delta\varepsilon E_s = \frac{\Delta L}{L} \times E_s \tag{19.12}$$

where

$\Delta\varepsilon$ = magnitude of the anchor slippage

$E_s = 29 \times 10^6$ psi

L = length of the tendon

Table 19.2 Friction Coefficients for Posttensioned Tendons

Type of Tendon	Wobble Coefficient K Per Foot ($\times 10^{-3}$)	Curvature Coefficient μ
Tendon in flexible metal sheathing (grouted)		
Wire tendons	1.0–1.5	0.15–0.25
Seven-wire strand	0.5–2.0	0.15–0.25
High-strength bars	0.1–0.6	0.08–0.30
Pregreased unbonded tendon		
Wire tendons and seven-wire strand	0.3–2.0	0.05–0.15
Mastic-coated unbonded tendons		
Wire tendons and seven-wire strand	1.0–2.0	0.05–0.15

Because the loss in stress is inversely proportional to the length of the tendon (or approximately half the length of the tendon if it is stressed from both ends simultaneously), the percentage loss in steel stress decreases as the length of the tendon increases. If the tendon is elongated by $\Delta\varepsilon$ at transfer, the loss in prestress due to slippage is neglected.

Example 19.2

A 36-ft-span pretensioned simply supported beam has a rectangular cross-section with $b = 18$ in. and $h = 32$ in. Calculate the elastic loss and all time-dependent losses. Given: prestressing force at transfer is $F_i = 435$ K, area of prestressing steel is $A_{PS} = 3.0\,\text{in.}^2$, $f'_c = 5$ ksi, $E_c = 5000$ ksi, $E_s = 29{,}000$ ksi, profile of tendon is parabolic, eccentricity at midspan = 6.0 in., and eccentricity at ends = 0.

Solution

1. Elastic shortening: Stress due to the prestrepssing force at transfer is

$$\frac{F_i}{A_{ps}} = \frac{435}{3} = 145 \text{ ksi}$$

$$\text{Strain in prestressing steel} = \frac{f_s}{E_s} = \frac{145}{29{,}000} = 0.005$$

Using Eq. 19.1,

$$n = \frac{E_s}{E_c} = \frac{29{,}000}{5000} = 5.8 \quad \text{or} \quad 6$$

$$\Delta f_s = \frac{nF_i}{A_c} = \frac{6 \times 435}{32 \times 18} = 4.5 \text{ ksi}$$

Considering the variation in the eccentricity along the beam,

$$\text{Strain at end of section} = \frac{F_i}{A_c E_c} = \frac{435}{(18 \times 32) \times 5000} = 0.151 \times 10^{-3}$$

$$\text{Strain at midspan} = \frac{F_i}{A_c E_c} + \frac{F_i e^2}{I E_c}$$

$$I = \frac{bh^3}{12} = \frac{18(32)^3}{12} = 49{,}152 \text{ in.}^4$$

$$\text{Strain} = 0.151 \times 10^{-3} + \frac{435(6)^2}{49{,}152(5000)} = 0.215 \times 10^{-3}$$

$$\text{Average strain} = \frac{1}{2}(0.151 + 0.215) \times 10^{-3} = 0.183 \times 10^{-3}$$

$$\text{Prestress loss} = \text{strain} \times E_s = 0.183 \times 10^{-3} \times 29{,}000 = 5.3 \text{ ksi}$$

$$\text{Percent loss} = \frac{5.3}{145} = 3.66\%$$

2. Loss due to shrinkage:

$$\text{Shrinkage strain} = 0.0003$$

$$\Delta f_s = \varepsilon_{sh} E_s = 0.0003 \times 29{,}000 = 8.7 \text{ ksi}$$

$$\text{Percent loss} = \frac{8.7}{145} = 6\%$$

3. Loss due to creep of concrete: Assuming $C_c = 2.0$, then $\Delta f_s = C_c(\varepsilon_{cr} E_s)$

$$\text{Elastic strain} = \frac{F_i}{A_c E_c} = 0.151 \times 10^{-3}$$

$$\Delta f_s = 2(0.151 \times 10^{-3} \times 29{,}000) = 8.8 \text{ ksi}$$

$$\text{Percent loss} = \frac{8.8}{145} = 6.1\%$$

Or, approximately, $\varepsilon_{cr} = 48 \times 10^{-5} \times$ stress in the concrete (ksi):

$$\varepsilon_{cr} = 48 \times 10^{-5} \left(\frac{435}{32 \times 18} \right) = 36 \times 10^{-5}$$

$$\Delta f_s = \varepsilon_{cr} E_s = 36 \times 10^{-5} \times 29{,}000 = 10.4 \text{ ksi}$$

$$\text{Percent loss} = \frac{10.4}{145} = 7.2\%$$

This is a conservative value, and the same ratio is obtained if $C_c = 2.38$ is adopted in the preceding calculations.

4. Loss due to relaxation of steel: For low-relaxation strands, the loss is assumed to be 2.5%.

$$\Delta f_s = 0.025 \times 145 = 3.6 \text{ ksi}$$

5. Assume the losses due to bending, friction of cable spacers, and the end block of the pretensioning system are 2%.

$$\Delta f_s = 0.02 \times 145 = 2.9 \text{ ksi}$$

6. Loss due to friction in tendon is 0.
7. Total losses are as follows.

Elastic shortening loss	5.3 ksi	3.6%
Shrinkage loss	8.7 ksi	6.0%
Creep of concrete loss	8.8 ksi	6.1%
Relaxation of steel loss	3.6 ksi	2.5%
Other losses	2.9 ksi	2.0%
Total losses	29.3 ksi	20.2%

$$\text{Effective prestress} = 145 - 24 = 121 \text{ ksi}$$

$$\text{Effective prestressing force } F = 121 \times 3\text{in.}^2 = 363 \text{ ksi}$$

$$F = (1 - 0.166)F_i = 0.834F_i$$

For $F = \eta F_i$, $\eta = 0.834$.

Example 19.3

Calculate all losses of a 120-ft-span posttensioned beam that has an I-section with the following details. Area of concrete section (A_c) = 760 in.^{2}; moment of inertia (I_g) = 1.64×10^5 in.^{4}; prestressing force at transfer (F_i) = 1110 K; area of prestressing steel (A_{ps}) = 7.5 in.^{2}; f'_c = 5 ksi, E_c = 5000 ksi, and E_s = 29,000 ksi; profile of tendon is parabolic; eccentricity at midspan = 20 in; and eccentricity at ends = 0.

Solution

1. Loss due to elastic shortening:

$$\text{Steel stress at transfer} = \frac{F_i}{A_{ps}} = \frac{1110}{7.5} = 148 \text{ ksi}$$

$$\text{Stress in concrete at end section} = \frac{1110}{760} = 1.46 \text{ ksi}$$

$$\text{Stress in concrete at midspan} = \frac{F_i}{A_c} + \frac{F_i e^2}{I} - \frac{M_D e}{I}$$

$$\text{Weight of beam} = \frac{760}{144} \times 150 = 790 \text{ lb/ft}$$

$$M_D = 0.79\frac{(120)^2}{8} = 1422 \text{ K·ft}$$

$$\text{Stress at midspan} = \frac{1110}{760} + \frac{1110(20)^2}{164{,}000} - \frac{(1422 \times 12)(20)}{164{,}000}$$

$$= 1.46 + 2.71 - 2.08 = 2.09 \text{ ksi}$$

$$\text{Average stress} = \frac{1.46 + 2.09}{2} = 1.78 \text{ ksi}$$

$$\text{Average strain} = \frac{1.78}{E_c} = \frac{1.78}{5000} = 0.356 \times 10^{-3}$$

Elastic loss is $\Delta f_s = \varepsilon_c E_s = 0.356 \times 103 \times 29{,}000 = 10.3$ ksi, assuming that the tendons are tensioned two at a time. The first pair will have the greatest loss, whereas the last pair will have 0 loss. Therefore, average $\Delta f_s = 10.3/2 = 5.15$ ksi.

$$\text{Percent loss} = \frac{5.15}{148} = 3.5\%$$

2. Loss due to shrinkage of concrete:

$$\Delta f_s \text{ (shrinkage)} = 0.0002E_s = 0.0002 \times 29{,}000 = 5.8 \text{ ksi}$$

$$\text{Percent loss} = \frac{5.8}{148} = 3.9\%$$

3. Loss due to creep of concrete: Assume $C_c = 1.5$.

$$\text{Elastic strain} = \frac{F_i}{A_c E_c} = \frac{1110}{760 \times 5000} = 0.92 \times 10^{-3}$$

$$\Delta f_s \text{ (creep)} = C_c(\varepsilon_{cr} E_s)$$

$$= 1.5(0.292 \times 10^{-3} \times 29{,}000) = 12.7 \text{ ksi}$$

$$\text{Percent loss} = \frac{12.7}{148} = 8.6\%$$

4. Loss due to relaxation of steel: For low-relaxation strands, the loss is 2.5%.

$$\Delta f_s = 0.025 \times 148 = 3.7 \text{ ksi}$$

5. Slip in anchorage: For tensioning from one end only, assume a slippage of 0.15 in. The length of the cable is $120 \times 12 = 1440$ in.

$$\Delta f_s = \frac{\Delta L}{L} \times E_s = \frac{0.15}{1440} \times 29{,}000 = 3 \text{ ksi} \tag{19.12}$$

To allow for anchorage slip, set the tensioned force to 148 + 3 = 151 ksi on the pressure gauge to leave a net stress of 148 ksi in the tendons.

6. Loss due to friction: The equation of parabolic profile is

$$e_x = \frac{4e}{L^2}(Lx - x^2)$$

where e_x = the eccentricity at a distance x measured from the support and e = eccentricity at midspan.

$$\frac{d(e_x)}{dx} = \frac{4e}{L^2}(L - 2x)$$

is the slope of the tendon at any point. At the support, $x = 0$ and the slope

$$\frac{d(e_x)}{dx} = \frac{4e}{L} = \frac{4 \times 20}{120 \times 12} = 0.056$$

The slope at midspan is 0; therefore, $\alpha_{px} = 0.056$. Using flexible metallic sheath, $\mu_p = 0.5$ and $K = 0.001$. At midspan, $x = 60$ ft. Check if $(\mu_p \alpha_{px} + Kl_x) \leq 0.30$:

$$\begin{aligned} \mu_p \alpha_{px} + Kl_x &= 0.5 \times 0.056 + 0.001 \times 60 = 0.0088 < 0.3 \\ P_{px} &= P_{pj}(1 + Kl_{px} + \mu_p \alpha_{px}) \\ &= P_x(1 + 0.088) = 1.088 P_x \\ &= 1.088 \times 148 = 161 \text{ K} \quad \text{(force at jacking end)} \qquad (19.11) \\ \Delta f_s &= 161 - 148 = 13 \text{ ksi} \\ \text{Percent loss} &= \frac{13}{148} = 8.8\% \end{aligned}$$

7. Total losses:

Elastic shortening loss	5.2 ksi	3.5%
Shrinkage loss	5.8 ksi	3.9%
Creep of concrete loss	12.7 ksi	8.6%
Relaxation of steel loss	3.7 ksi	2.5%
Friction losses	13.0 ksi	8.8%
Total losses	40.4 ksi	27.3%

$$\text{Effective prestress} = 148 - 35.2 = 112.8 \text{ ksi}$$

$$\text{Effective prestressing force}(F) = (1 - 0.238)F_i = 0.762F_i$$

$$F = 0.762 \times 1110 = 846 \text{ K}$$

For $F = \eta F_i$, $\eta = 0.762$.

19.4 ANALYSIS OF FLEXURAL MEMBERS

19.4.1 Stresses Due to Loaded and Unloaded Conditions

In the analysis of prestressed concrete beams, two extreme loadings are generally critical. The first occurs at transfer, when the beam is subjected to the prestressing force, F_i, and the weight of the beam or the applied dead load at the time of transfer of the prestressing force. No live

load or additional dead loads are considered. In this unloaded condition, the stresses at the top and bottom fibers of the critical section must not exceed the allowable stresses at transfers, f_{ci} and f_{ti}, for the compressive and tensile stresses in concrete, respectively.

The second case of loading occurs when the beam is subjected to the prestressing force after all losses F and all dead and live loads. In this loaded condition, the stresses at the top and bottom fibers of the critical section must not exceed the allowable stresses, f_c and f_t, for the compressive and tensile stresses in concrete, respectively.

These conditions can be expressed mathematically as follows.

1. For the unloaded condition (at transfer):
   - At top fibers,

$$\alpha_{ti} = -\frac{F_i}{A} + \frac{(F_i e)y_t}{I} - \frac{M_D y_t}{I} \leq f_{ti} \tag{19.14}$$

   - At bottom fibers,

$$\alpha_{bi} = -\frac{F_i}{A} - \frac{(F_i e)y_b}{I} + \frac{M_D y_b}{I} \geq -f_{ci} \tag{19.15}$$

2. For the loaded condition (all loads are applied after all losses):
   - At top fibers,

$$\sigma_t = -\frac{F}{A} + \frac{(Fe)y_t}{I} - \frac{M_D y_t}{I} - \frac{M_L y_t}{I} \geq -f_c \tag{19.16}$$

   - At bottom fibers,

$$\sigma_b = -\frac{F}{A} - \frac{(Fe)y_b}{I} + \frac{M_d y_b}{I} + \frac{M_L y_b}{I} \leq f_t \tag{19.16}$$

   where

F_i and F = the prestressing force at transfer and after all losses
f_{ti} and f_t = allowable tensile stress in concrete at transfer and after all losses
f_{ci} and f_c = allowable compressive stress in concrete at transfer and after all losses
M_D and M_L = moments due to dead load and live load
y_t and y_b = distances from the neutral axis to the top and bottom fibers

In this analysis, it is assumed that the materials behave elastically within the working range of stresses applied.

19.4.2 Kern Limits

If the prestressing force is applied at the centroid of the cross-section, uniform stresses will develop. If the prestressing force is applied at an eccentricity, e below the centroid such that the stress at the top fibers is equal to 0, that prestressing force is considered acting at the lower Kern point (Fig. 19.5). In this case e is denoted by K_b, and the stress distribution is triangular, with maximum compressive stress at the extreme bottom fibers. The stress at the top fibers is

$$\begin{aligned} \sigma_t &= -\frac{F_i}{A} + \frac{(F_i e)y_t}{I} = 0 \\ e &= K_b = \text{lower Kern} = \frac{I}{Ay_t} \end{aligned} \tag{19.17}$$

Similarly, if the prestressing force is applied at an eccentricity e' above the centroid such that the stress at the bottom fibers is equal to 0, that prestressing force is considered acting at the upper Kern point (Fig. 19.5). In this case the eccentricity it e' is denoted by K_t, and the stress distribution is triangular, with maximum compressive stress at the extreme top fibers. The stress at the bottom fibers is

$$\begin{aligned} \sigma_b &= -\frac{F_i}{A} + \frac{(F_i e')y_b}{I} = 0 \\ e' &= K_t = \text{upper Kern} = \frac{I}{Ay_b} \end{aligned} \tag{19.18}$$

The Kern limits of a rectangular section are shown in Fig. 19.5.

19.4.3 Limiting Values of Eccentricity

The four stress equations, Eqs. 19.13 through 19.16,can be written as a function of the eccentricity e for the various loading conditions. For example, Eq. 19.13 can be rewritten as follows:

$$\sigma_{ti} = -\frac{F_i}{A} + \frac{(F_i e)y_t}{I} - \frac{M_D y_t}{I} \le f_{ti}$$

$$\frac{(f_i e)y_t}{I} \le f_{ti} + \frac{F_i}{A} + \frac{M_D y_t}{I}$$

$$e \le \frac{I}{F_i y_t}\left(\frac{F_i}{A} + \frac{M_D y_t}{I} + f_{ti}\right) \tag{19.19}$$

If the lower Kern limit $K_b = I/Ay_t$ is used, then

$$e \le K_b + \frac{M_D}{F_i} + \frac{f_{ti} A K_b}{F_i} \tag{19.20}$$

This value of e represents the maximum eccentricity based on the top fibers, unloaded condition.

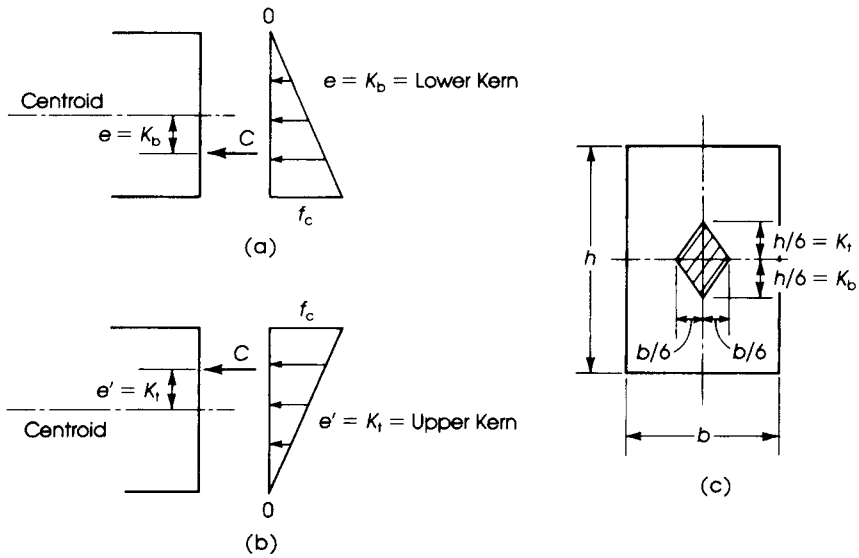


Figure 19.5 Kern points: (*a*) lower, (*b*) upper, and (*c*) central.

Similarly, from Eq. 19.14,

$$e \le \frac{I}{F_i y_b}\left(-\frac{F_i}{A} + \frac{M_D y_b}{I} + f_{ci}\right) \tag{19.21}$$

$$e \le -K_t + \frac{M_D}{F_i} + \frac{f_{ci} A K_t}{F_i} \tag{19.22}$$

This value of e represents the maximum eccentricity based on the bottom fibers, unloaded condition. The two maximum values of e should be calculated from the preceding equations and the smaller value used.

From Eq. 19.15,

$$e \ge \frac{I}{F y_t}\left(\frac{F}{A} + \frac{M_T y_t}{I} - f_c\right) \tag{19.23}$$

$$e \ge K_b + \frac{M_T}{F} - \frac{f_c A K_b}{F} \tag{19.24}$$

where M_T = moment due to dead and live loads = $(M_D + M_L)$. This value of e represents the minimum eccentricity based on the top fibers, loaded condition. From Eq. 19.17,

$$e \ge \frac{I}{F y_b}\left(-\frac{F}{A} + \frac{M_T y_b}{I} - f_t\right) \tag{19.25}$$

$$e \ge K_t + \frac{M_T}{F} - \frac{f_t A K_t}{F} \tag{19.26}$$

This value of e represents the minimum eccentricity based on the bottom fibers, loaded condition. The two minimum values of e should be calculated from the preceding equations and the larger of the two minimum eccentricities used.

19.4.4 Limiting Values of the Prestressing Force at Transfer F_i

Considering that $F = \eta F_i$, where η represents the ratio of the net prestressing force after all losses, and for the different cases of loading, Eqs. 19.20, 19.22, 19.24, and 19.26 can be rewritten as follows:

$$(e - K_b)F_i \le M_D + f_{ti} A K_b \tag{19.27}$$

$$(e + K_t)F_i \le M_D + f_{ci} A K_t \tag{19.28}$$

$$(e - K_b)F_i \ge \frac{M_D}{\eta} + \frac{M_L}{\eta} - \frac{1}{\eta}(f_c A K_t) \tag{19.29}$$

$$(e + K_t)F_i \ge \frac{M_D}{\eta} + \frac{M_L}{\eta} - \frac{1}{\eta}(f_t A K_t) \tag{19.30}$$

Subtract Eq. 19.29 from Eq. 19.32 to get

$$F_i(K_b + K_t) \ge M_D\left(\frac{1}{\eta} - 1\right) + \frac{M_L}{\eta} - \frac{f_t A K_t}{\eta} - f_{ti} A K_t$$

or

$$F_i \ge \frac{1}{(K_b + K_t)}\left[\left(\frac{1}{\eta} - 1\right) M_D + \frac{M_L}{\eta} - \left(\frac{f_t A K_t}{\eta}\right) - (f_{ti} A K_b)\right] \tag{19.31}$$

This value of F_i represents the *minimum* value of the prestressing force at transfer without exceeding the allowable stresses under the loaded and unloaded conditions. Subtract Eq. 19.29 from Eq. 19.28 toget

$$F_i \leq \frac{1}{(K_b + K_t)} \left[\left(1 - \frac{1}{\eta}\right) M_D - \frac{M_L}{\eta} + \left(\frac{f_c A K_b}{\eta}\right) + (f_{ci} A K_t) \right] \tag{19.32}$$

This value of F_i represents the *maximum* value of the prestressing force at transfer without exceeding the allowable stresses under the loaded and unloaded conditions. Subtracting Eq. 19.31 from Eq. 19.32,

$$\left(1 - \frac{1}{\eta}\right) 2M_D - \frac{2M_L}{\eta} + \left(f_{ti} + \frac{f_c}{\eta}\right) A K_b + \left(f_{ci} + \frac{f_t}{\eta}\right) A K_t \geq 0 \tag{19.33}$$

This equation indicates that (maximum F_i) − (minimum F_i) ≥ 0. If this equation is checked for any given section and proved to be satisfactory, then the section is adequate.

Example 19.4

A pretensioned simply supported beam of the section shown in Fig. 19.6 is to be used on a span of 48 ft. The beam made with normal-weight concrete must carry a dead load of 900 lb/ft (excluding its own weight), which will be applied at a later stage, and a live load of 1100 lb/ft. Assuming that prestressing steel is made of 20 tendons that are 716 in. in diameter, with $E_s = 29 \times 10^6$ psi, $F_o = 175$ ksi, and ultimate strength $f_{pu} = 250$ ksi, it is required to do the following:

1. Determine the location of the upper and lower limits of the tendon profile (centroid of the prestressing steel) for the section at midspan and for three other sections between the midspan section and the beam end.
2. Locate the tendon to satisfy these limits by harping some of the tendons at one-third points of the span. Check the limiting values of the prestressing force at transfer.
3. Revise the prestress losses, taking into consideration the chosen profile of the tendons and the variation of the eccentricity, e.
   Use f_{ci} (at transfer) = 4 ksi, $f'_c = 5$ ksi, $E_c = 4000$ ksi, and $E_{ci} = 3600$ ksi.

Solution

1. Determine the properties of the section:

$$\text{Area} = 18 \times 6 + 24 \times 6 + 12 \times 10 = 372 \text{ in.}^2$$

Determine the centroid of the section by taking moments about the base line.

$$y_b = \frac{1}{372}(120 \times 5 + 144 \times 22 + 108 \times 37) = 20.8 \text{ in.}$$

$$y_t = 40 - 20.8 = 19.2 \text{ in.}$$

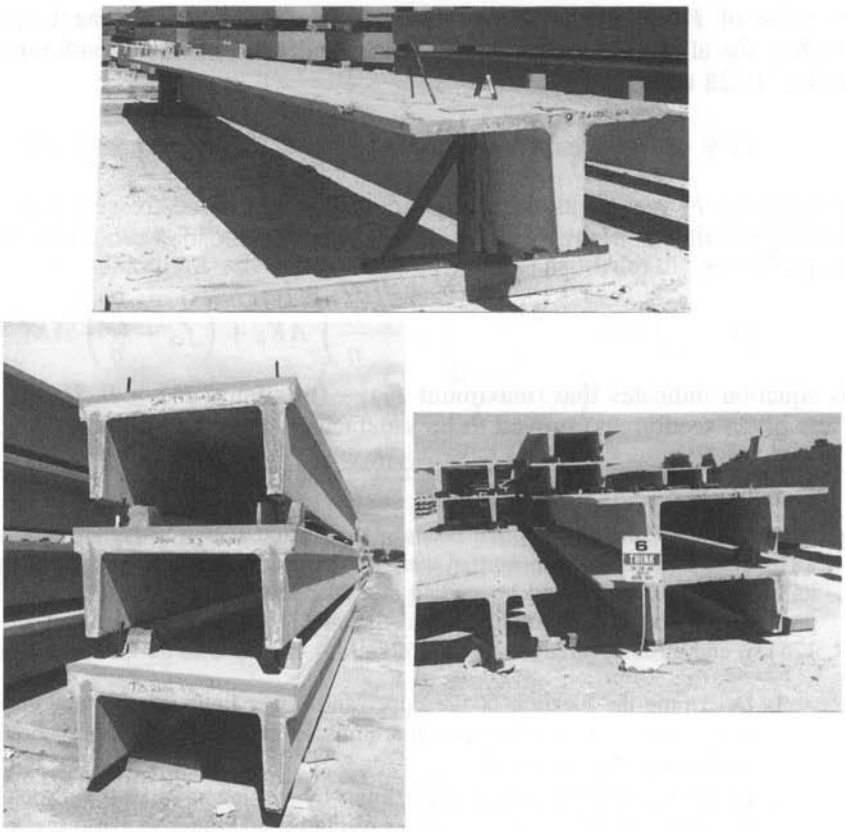
Calculate the gross moment of inertia, I_g:

$$I_g = \left[\frac{18(6)^3}{12} + 108(16.2)^2\right] + \left[\frac{6(24)^3}{12} + 144(1.2)^2\right] + \left[\frac{12(10)^3}{12} + 120(15.8)^2\right]$$

$$= 66{,}862 \text{ in.}^4$$

$$K_b = \frac{I}{Ay_t} = \frac{66{,}862}{372 \times 19.2} = 9.4 \text{ in.}$$

$$K_t = \frac{I}{Ay_b} = \frac{66{,}862}{372 \times 20} = 8.6 \text{ in.}$$



Precast, prestressed concrete sections: (*a*) single T-, (*b*) double T- and (*c*) U-sections.

2. Estimate prestress losses, given $F_o = 175$ ksi.
   - **a.** Assume elastic loss is 4%, or $0.04 \times 175 = 7$ ksi.
   - **b.** Loss due to shrinkage is $0.0003E_s = 0.0003 \times 29{,}000 = 8.7$ ksi.
   - **c.** Loss due to creep of concrete: A good first estimate of creep loss is 1.67 times the elastic loss.

$$1.67 \times 7 = 11.7 \text{ ksi}$$

   - **d.** Loss due to relaxation of steel is 4%:

$$0.04 \times 175 = 7 \text{ ksi}$$

   Time-dependent losses are $8.7 + 11.7 + 7 = 27.4$ ksi.

$$\text{Percentage} = \frac{27.4}{175} = 15.7\%$$

   - **e.** The total loss is 27.4 + 7 (elastic loss) = 34.4 ksi. The percentage of total loss is

$$34.4/175 = 19.7\%$$

f. Prestress stresses are

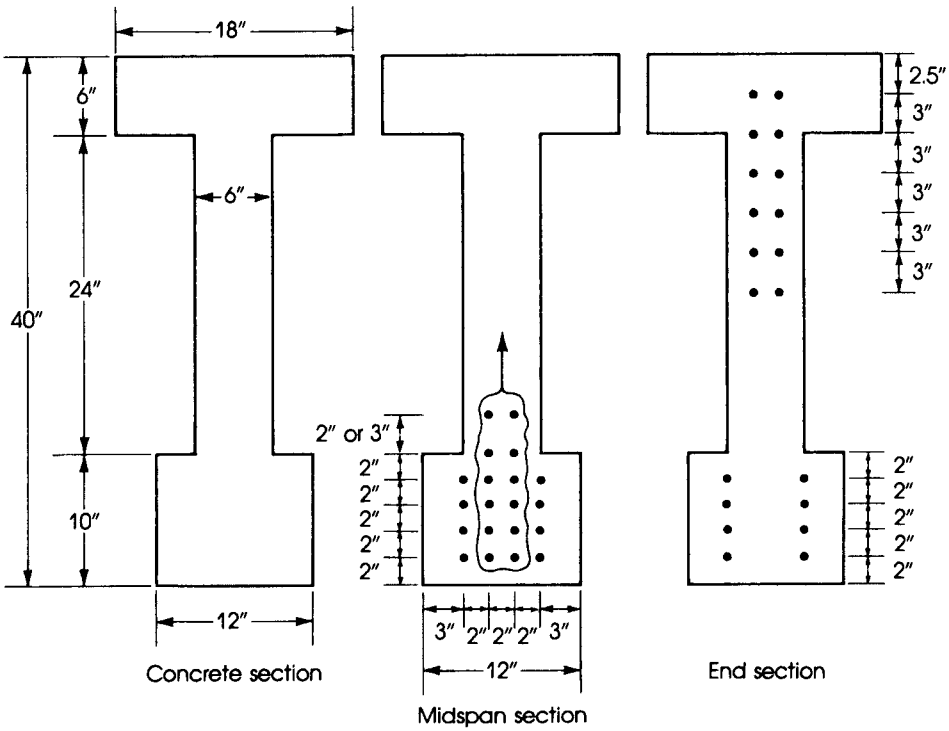
$$F_i = 175 - 7 = 168 \text{ ksi} \qquad \text{(at transfer)}$$

$$F = 175 - 34.4 = 140.6 \text{ ksi}$$

$$F = \eta F_i$$

$$\eta = 1 - \text{time} - \text{dependent losses ratio}$$

$$= \frac{140.6}{168} = 0.837$$



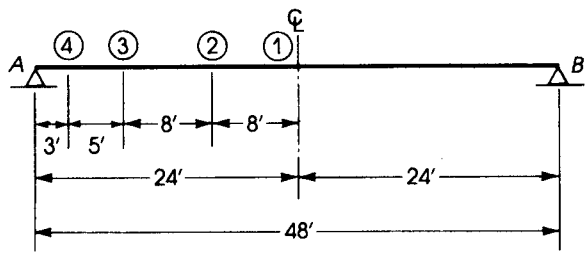


Figure 19.6 (a) Example 19.4.

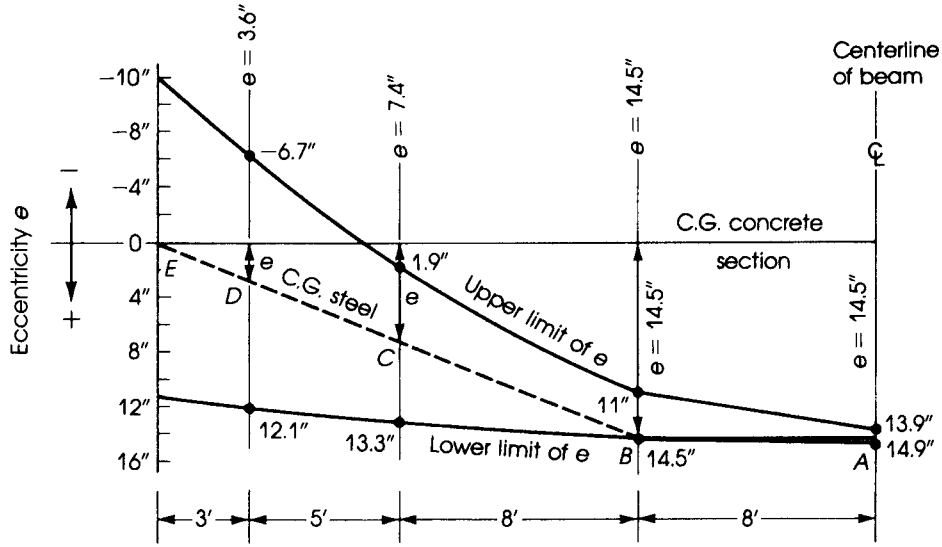


Figure 19.6 (b) Example 19.4: tendon profile.

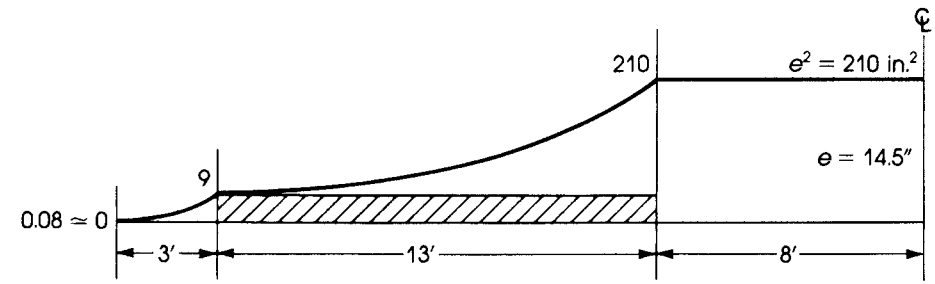


Figure 19.6 (c) Example 19.4: average e^2.

3. Limits of the eccentricity, e, at midspan section: Calculate the allowable stresses and moments. At transfer, $f'_{ci} = 4000$ psi, $f_{ci} = 0.6 \times 4000 = 2400$ psi, and $f_{ti} = 3\sqrt{f'_c} = 190$ psi. At service load, $f'_c = 5000$ psi, $f_c = 0.45 f'_c = 2250$ psi, and $f_t = 6\sqrt{f'_c} = 424$ psi.

$$\text{Self-weight of beam} = \frac{372}{144} \times 150 = 388 \text{ lb/ft}$$

$$M_D \text{ (self-weight)} = \frac{0.388}{8}(48)^2 \times 12 = 1341 \text{ K·in.}$$

$$M_a \text{ (additional load and live load)} = \frac{w_a L^2}{8}$$

$$= \frac{(0.9 + 1.1)}{8}(48)^2 \times 12 = 6912 \text{ K·in.}$$

$$\text{Total moment } (M_T) = M_D + M_a = 8253 \text{ K·in.}$$

$$F_i = \text{stress at transfer} \times \text{area of prestressing steel}$$

The area of 20 tendons, $\frac{7}{16}$ in. in diameter, is $20 \times 0.1089 = 2.178$ in.^{2}

$$F_i = 2.178 \times 168 = 365.9 \text{ K}$$

$$F = 2.178 \times 140.6 = 306.2 \text{ K}$$

a. Consider the section at midspan.
Top fibers, *unloaded* condition:

$$e \le K_b + \frac{M_D}{F_i} + \frac{f_{ti} A K_b}{F_i}$$

$$\le 9.4 + \frac{1341}{365.9} + \frac{0.190(372)(9.4)}{365.9} \le 14.9 \text{ in.} \tag{19.20}$$

Bottom fibers, *unloaded* condition:

$$e \le -K_t + \frac{M_D}{F_i} + \frac{f_{ci} A K_t}{F_i}$$

$$\le -8.6 + \frac{1341}{365.9} + \frac{2.4(372)(8.6)}{365.9} \le 16.1 \text{ in.} \tag{19.22}$$

Maximum $e = 14.9$ in. controls.

Top fibers, *loaded* condition:

$$e \ge K_b + \frac{M_T}{F} - \frac{f_c A K_b}{F}$$

$$\ge 9.4 + \frac{8253}{306.2} - \frac{0.424(372)(8.6)}{306.2} \ge 10.7 \text{ in.} \tag{19.24}$$

Bottom fibers, *loaded* condition:

$$e \ge -K_t + \frac{M_T}{F} - \frac{f_t A K_t}{F}$$

$$\ge -8.6 + \frac{8253}{306.2} - \frac{0.424(372)(8.6)}{306.2} \ge 13.9 \text{ in.} \tag{19.26}$$

Minimum $e = 13.9$ in. controls.

b. Consider a section 8 ft from the midspan (section 2, Fig. 19.6a):

$$M_D \text{ (self-weight)} = R_A(16) - \frac{w_D}{2} \times (16)^2$$

$$= 0.388(24)(16) - \frac{0.388}{2}(16)^2 = 99.3 \text{ K·ft} = 1192 \text{ K·in.}$$

$$M_a = 2(24)(16) - \frac{2}{2}(16)^2 = 512 \text{ K·ft} = 6144 \text{ K·in.}$$

$$M_T = 6144 + 1192 = 7336 \text{ K·in.}$$

Top fibers, *unloaded* condition:

$$e \le 9.4 + \frac{1192}{365.9} + \frac{0.190(372)(9.4)}{365.9} \le 14.5 \text{ in.}$$

Bottom fibers, *unloaded* condition:

$$e \le -8.6 + \frac{1192}{365.9} + \frac{2.4(372)(8.6)}{365.9} \le 15.6 \text{ in.}$$

Maximum $e = 14.5$ in. controls.

Top fibers, *loaded* condition:

$$e \geq 9.4 + \frac{7336}{306.2} - \frac{2.25(372)(9.4)}{306.2} \geq 7.7 \text{ in.}$$

Bottom fibers, *loaded* condition:

$$e \geq -8.6 + \frac{7336}{306.2} - \frac{0.424(372)(8.6)}{306.2} \geq 11.0 \text{ in.}$$

Minimum $e = 11.0$ in. controls.

c. Consider a section 16 ft from midspan (section 3, Fig. 19.6a): M_D (self-weight) = 745 K·in., $M_a = 3840$ K·in., and $M_T = 4585$ K·in.

- Top fibers, unloaded condition, $e \leq 13.3$ in. (max) controls.
- Bottom fibers, unloaded condition, $e \leq 14.4$ in.
- Top fibers, loaded condition, $e \geq -1.3$ in.
- Bottom fibers, loaded condition, $e \geq 1.9$ in. (min) controls.

d. Consider a section 3 ft from the end (anchorage length): $M_D = 314$ K·in., $M_a = 1620$ K·in., and $M_T = 1934$ K·in.

- Top fibers, unloaded condition, $e \leq 12.1$ in. (max) controls.
- Bottom fibers, unloaded condition, $e \leq 13.3$ in.
- Top fibers, loaded condition, $e \geq -10$ in.
- Bottom fibers, loaded condition, $e \geq -6.7$ in. (min) controls.

4. The tendon profile is shown in Fig. 19.6b. The eccentricity chosen at midspan is $e = 14.5$ in., which is adequate for section B at 8 ft from midspan. The centroid of the prestressing steel is horizontal between A and B and then harped linearly between B and the end section at E. The eccentricities at sections C and D are calculated by establishing the slope of line BE, which is 14.5/16 = 0.91 in/ft. The eccentricity at C is 7.25 in. and at D it is 2.72 in. The tendon profile chosen satisfies the upper and lower limits of the eccentricity at all sections.

Harping of tendons is performed as follows:

a. Place the 20 tendons ($\frac{7}{16}$ diameter) within the middle third of the beam at spacings of 2 in., as shown in Fig. 19.6a. To calculate the actual eccentricity at midspan section, take moments for the number of tendons about the base line of the section:

$$\text{Distance from base} = \tfrac{1}{20}(16 \times 5 + 4 \times 11) = 6.2 \text{ in.}$$

$$e \text{ (midspan)} = y_b - 6.2 \text{ in.}$$

$$= 20.8 - 6.2 = 14.6 \text{ in.}$$

which is close to the 14.5 in. assumed. If the top two tendons are placed at 3 in. from the row below them, then the distance from the base becomes $\frac{1}{20}(16 \times 5 + 2 \times 10 + 2 \times 13) = 6.3$ in. The eccentricity becomes $20.8 - 6.3 = 14.5$ in., which is equal to the assumed eccentricity. Practically, all tendons may be left at 2 in. spacing by neglecting the difference of 0.1 in.

b. Harp the central 12 tendons only. The distribution of tendons at the end section is shown in Fig. 19.6a. To check the eccentricity of tendons, take moments about the centroid of the concrete section for the 12 tendons at top and the eight tendons left at bottom:

$$e = \tfrac{1}{20}(8 \times 14.5 - 12 \times 9.2) = 0.28 \text{ in.}$$

This value of e is small and adequate. The actual eccentricity at 3 ft from the end section is

$$e = \tfrac{3}{16}(14.5 - 0.28) + 0.28 = 2.95 \text{ in.} \qquad (3 \text{ in.})$$

The actual eccentricity at 8 ft from the end section is

$$e = \tfrac{1}{2}(14.5 - 0.28) + 0.28 = 7.4 \text{ in.}$$

5. Limited values of F_i: The value of F_i used in the preceding calculations is $F_i = 365.9$ K. Check minimum F_i by Eq. 19.31:

$$\text{Min. } F_i = \frac{1}{(K_b + K_t)}\left[\left(\frac{1}{\eta} - 1\right)M_D + \frac{M_L}{\eta} - \frac{(f_t A K_t)}{\eta}\right) - (f_{ti} A K_b)$$

$$= \frac{1}{(9.4 + 8.6)}\left[\left(\frac{1}{0.8423} - 1\right)1341 + \frac{6912}{0.843} - \frac{(0.424 \times 372 \times 8.6)}{0.843}(0.19 \times 372 \times 9.4)\right] = 343.1 \text{ K}$$

which is less than the F_i used. Check maximum F_i using Eq. 19.32:

$$\text{Max. } F_i = \frac{1}{(K_b + K_t)}\left[\left(1 - \frac{1}{\eta}\right)M_D - \frac{M_L}{\eta} + \frac{(f_c A K_b)}{\eta} + (f_{ci} A K_t)\right]$$

$$= \frac{1}{18}\left[\left(1 - \frac{1}{0.843}\right)1341 - \frac{6912}{0.843} + \frac{(2.25 \times 3.72 \times 9.4)}{0.843} + (2.4 \times 3.72 \times 8.6)\right]$$

$$= 475.7 \text{ K}$$

which is greater than the F_i used. Therefore, the critical section at midspan is adequate.

6. Check prestress losses, recalling that $F_o = 175$ ksi and $A_{ps} = 2.178$ in.^{2}

$$\text{Total } F_o = 2.178 \times 175 = 381 \text{ K}$$

$$E_c = 4000 \text{ ksi}$$

$$n = \frac{E_s}{E_c} = \frac{29}{4.0} = 7.25$$

n can be assumed to be 7.

$$M_D \text{ at midspan} = 1341 \text{ K·in.}$$

$$F_i = \frac{F_o + nA_{ps}f_c(\text{D.L.}) \times \frac{2}{3}}{1 + (nA_{ps})\left(\frac{1}{A} + \frac{e^2}{I}\right)} \tag{19.5}$$

The value of f_c due to the distributed dead load is multiplied by $\frac{2}{3}$ to reflect the parabolic variation of the dead load stress along the span, giving a better approximation of F_i.

a. Determine the average value of e^2, as adopted in the beam. The curve representing e^2 is shown in Fig. 19.6c:

$$\text{Average } e^2 = \tfrac{1}{24}\left[\left(\tfrac{1}{3} \times 3 \times 9\right) + (9 \times 13) + \left(\tfrac{1}{3} \times 13 \times 201\right) + (210 \times 8)\right]$$

$$= 111.5 \text{ in.}^2$$

$$e = 10.56 \text{ in.}$$

The area of a parabola is one-third the area of its rectangle.

b. Stress due to dead load at the level of the tendons is

$$f_c(\text{D.L.}) = \frac{1341 \times 10.56}{66{,}862} = 0.212 \text{ ksi}$$

Therefore,

$$F_i = \frac{381 + 7(2.178) \times 0.212 \times 2/3}{1 + (7 \times 2.178)\left(\frac{1}{372} + \frac{111.5}{66{,}862}\right)} = 358 \text{ K}$$

Elastic loss is 381 − 358 = 23 K = 6.1%. This value is greater than the assumed elastic loss of 4%.

$$\text{Elastic loss per unit steel area} = \frac{23}{2.178} = 10.6 \text{ ksi}$$

$$F_i \text{ per unit steel area} = \frac{358}{2.178} = 164.4 \text{ ksi}$$

c. Time-dependent losses:

Loss due to shrinkage = 8.7 ksi (as before)

Loss due to creep:

$$\text{Elastic strain} = \frac{F_i}{A_c E_c} = \frac{358}{372 \times 4000} = 0.240 \times 10^{-3}$$

$$\Delta f_s = C_c(\varepsilon_{cr} E_s)$$

Let $C_c = 1.5$. Then

$$\Delta f_s = 1.5(0.24 \times 10^{-3} \times 29{,}000) = 10.4 \text{ ksi}$$

$$\text{Percent loss} = \frac{10.4}{164.4} = 6.3\%$$

Loss due to relaxation of steel is 7 ksi (as before). Time-dependent losses equal 8.7 + 10.4 + 7 = 26.1 ksi, for a percentage loss of 26.1/164.4 = 15.8%, which is very close to the previously estimated value of 15.7%.

$$F = \eta F_i = (1 - 0.158)F_i = 0.842F_i$$

$$\eta = 0.842$$

19.5 DESIGN OF FLEXURAL MEMBERS

19.5.1 General

The previous section emphasized that the stresses at the top and bottom fibers of the critical sections of a prestressed concrete member must not exceed the allowable stresses for all cases or stages of loading. In addition to these conditions, a prestressed concrete member must be designed with an adequate factor of safety against failure. The ACI Code requires that the moment due to the factored service loads, M_u, must not exceed ϕM_n, the flexural strength of the designed cross-section.

For the case of a tension-controlled, prestressed concrete beam, failure begins when the steel stress exceeds the yield strength of steel used in the concrete section. The high-tensile prestressing steel will not exhibit a definite yield point, such as that of the ordinary mild steel bars used in reinforced concrete. But under additional increments of load, the strain in the steel increases at an accelerated rate, and failure occurs when the maximum compressive strain in the concrete reaches a value of 0.003 (Fig. 19.9).

The limits for reinforcement of prestressed concrete flexural members according to the ACI Code, Section 18.8, is based on the net tensile strain for tension-controlled, transition, or compression-controlled sections in accordance with the ACI Code, Section 10.3, as was explained here in this textbook, Section 3.5. The strength reduction factor, ϕ, was given earlier in Section 3.7 of this textbook based on the ACI Code, Section 9.3.

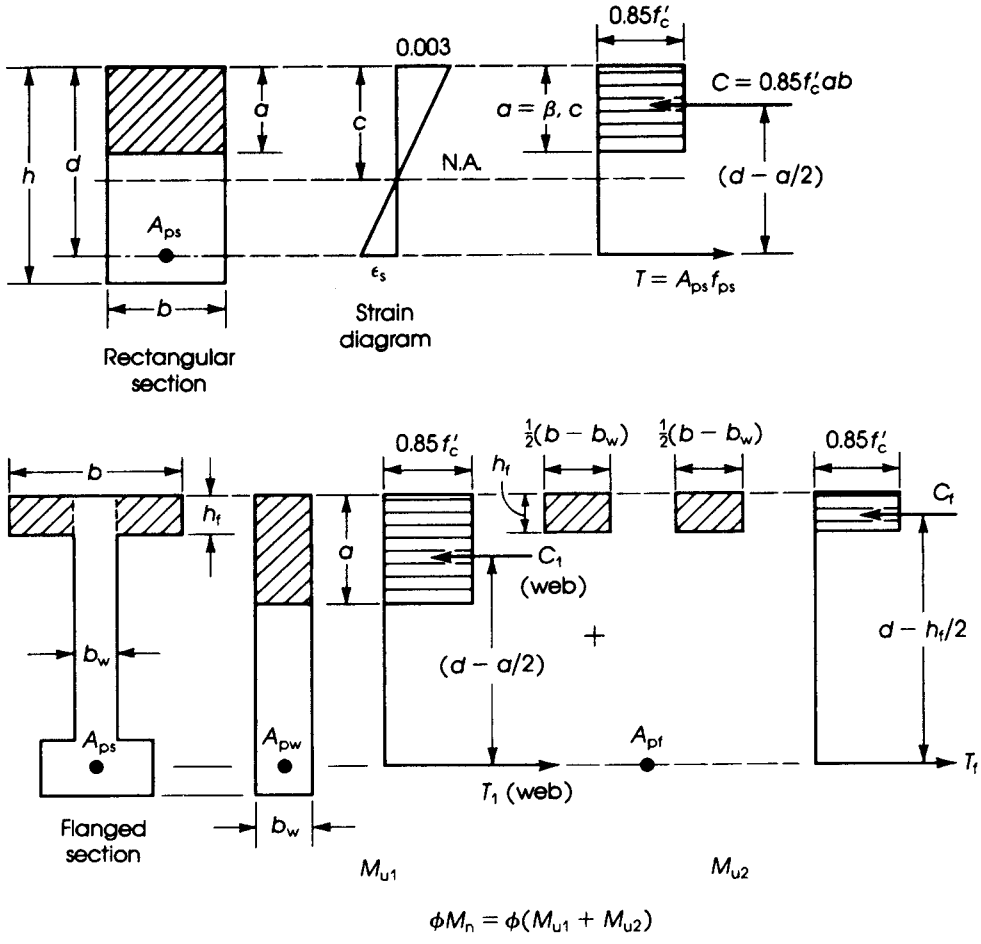


Figure 19.7 Factored moment capacity of prestressed concrete beams.

19.5.2 Rectangular Sections

The nominal moment capacity of a rectangular section may be determined as follows (refer to Fig. 19.7):

$$M_n = C\left(d - \frac{a}{2}\right) = T\left(d - \frac{a}{2}\right) \tag{19.34}$$

where $T = A_{ps} f_s$ and $C = 0.85\ f'_c ab$. For $C = T$,

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{\rho_p\, f_{ps}}{0.85 f'_c} d \tag{19.35}$$

where the prestressing steel ratio is $\rho_p = A_{ps}/bd$, and A_{ps} and f_{ps} refer to the area and tensile stress of the prestressing steel. Let

$$\omega_p = \rho_p \left(\frac{f_{ps}}{f'_c}\right) \leq 0.32\,\beta_1$$

Then

$$a = \frac{\omega_p}{0.85} d \tag{19.36}$$

The quantity ω_p is a direct measure of the force in the tendon. To ensure a tension-controlled behavior, the ACI Code, Section 18.8.1, specifies that ω_p must not exceed $0.32\beta_1$, which corresponds to a net tensile strain, ε_t, of 0.005. Note that the value of $\beta_1 = 0.85$ for $f'_c \leq 4$ ksi and reduces by 0.05 for each 1 ksi greater than 4 ksi (ACI Code, Section 10.2.7.3). M_n can also be written as follows:

$$M_n = A_{ps} f_{ps} \left(d - \frac{a}{2}\right)$$

$$M_n = A_{ps} f_{ps} d \left(1 - \frac{\rho_p f_{ps}}{1.7 f'_c}\right) \tag{19.37}$$

$$M_n = A_{ps} f_{ps} d \left(1 - \frac{\omega_p}{1.7}\right) \tag{19.38}$$

and $M_u = \phi M_n$.

In the preceding equations, f_{ps} indicates the stress in the prestressing steel at failure. The actual value of f_{ps} may not be easily determined. Therefore, the ACI Code, Section 18.7.2, permits f_{ps} to be evaluated as follows (all stresses are in psi). For *bonded* tendons,

$$f_{ps} = f_{pu} \left[1 - \frac{\gamma_p}{\beta_1} \left(\rho_p \times \frac{f_{pu}}{f'_c}\right)\right] \tag{19.39}$$

For *unbonded* tendons in members with a span-to-depth ratio less than or equal to 35,

$$f_{ps} = \left(f_{se} + 10{,}000 + \frac{f'_c}{100\rho_p}\right) \leq f_{py} \tag{19.40}$$

provided that $f_{se} \geq 0.5\ f_{pu}$ and that f_{ps} for unbonded tendons does not exceed either f_{py} or $f_{se} + 60{,}000$ psi. For *unbonded* tendons in members with a span-to-depth ratio greater than 35,

$$f_{ps} = \left(f_{se} + 10{,}000 + \frac{f'_c}{300\rho_p}\right) \tag{19.41}$$

but not greater than f_{py} or $f_{se} + 30{,}000$ psi, where

γ_p = factor for the type of prestressing tendon
= 0.55 for f_{py}/f_{pu} not less than 0.8
= 0.4 for f_{py}/f_{pu} not less than 0.85
= 0.28 for f_{py}/f_{pu} not less than 0.9

f_{pu} = specified tensile strength of prestressing steel

f_{se} = effective stress in prestressing steel after all losses

f_{py} = specified yield strength of prestressing steel

In the event that $\omega_p > 0.32\beta_1$, a compression-controlled, prestressed concrete beam may develop. To ensure a ductile failure, ω_p is limited to a maximum value of $0.32\beta_1$. For

$\omega_p = 0.32\beta_1$, $a = 0.377\beta_1 d$ (from Eq. 19.36). Substituting this value of a in Eq. 19.38,

$$\begin{aligned} M_n &= A_{ps} f_{ps} d \left(1 - \frac{0.32\beta_1}{1.7}\right) \\ &= (\rho_p bd) f_{ps} d(1 - 0.188\beta_1) \\ &= \omega_p f_c'(1 - 0.188\beta_1) bd^2 \\ &= (0.32\beta_1 - 0.06\beta_1^2) f_c' bd^2 \end{aligned} \tag{19.42}$$

for $f_c' = 5$ ksi, $\beta_1 = 0.8$. Then

$$M_n = 0.22 f_c' bd^2 = 1.09 bd^2$$

Similarly, for $f_c' = 4$ ksi, $M_n = 0.915 bd^2$, and for $f_c' = 6$ ksi, $M_n = 1.238 bd^2$.

19.5.3 Flanged Sections

For flanged sections (T- or I-sections), if the stress block depth a lies within the flange, it will be treated as a rectangular section. If a lies within the web, then the web may be treated as a rectangular section using the web width, b_w, and the excess flange width $(b - b_w)$ will be treated similarly to that of reinforced concrete T-sections discussed in Chapters 3 and 4. The design moment strength of a flanged section can be calculated as follows (see Fig. 19.7)

$M_n = M_{n_1}$ (moment strength of the web) + M_{n_2} (moment strength of excess flange)

$$M_n = A_{pw} f_{ps} \left(d_p - \frac{a}{2}\right) + A_{pf} f_{ps} \left(d_p - \frac{h_f}{2}\right) \tag{19.43}$$

$$M_u = \phi M_n \quad \text{and} \quad a = \frac{A_{pw} f_{ps}}{0.85 f_c' b_w}$$

where

$A_{pw} = A_{ps} - A_{pf}$
$A_{pf} = [0.85 f_c'(b - b_w)hf]/f_{ps}$
h_f = thickness of the flange

Note that the total prestressed steel, A_{ps}, is divided into two parts, A_{pw} and A_{pf}, developing the web and flange moment capacity. For flanged sections, the reinforcement index, ω_{pw}, must not exceed $0.32\beta_1$ for tension-controlled sections, where

$$\omega_{pw} = \left(\frac{A_{pw}}{b_w d}\right)\left(\frac{f_{ps}}{f_c'}\right) = \text{prestressed web steel ratio} \times \left(\frac{f_{ps}}{f_c'}\right)$$

19.5.4 Nonprestressed Reinforcement

In some cases, nonprestressed reinforcing bars (A_s) are placed in the tension zone of a prestressed concrete flexural member together with the prestressing steel (A_{ps}) to increase the moment strength of the beam. In this case, the total steel (A_{ps} and A_s) is considered in the moment analysis. For rectangular sections containing prestressed and nonprestressed steel, the design moment strength, ϕM_n, may be computed as follows:

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2}\right) + A_s f_y \left(d - \frac{a}{2}\right) \tag{19.44}$$

where

$$a = \frac{A_{ps} f_{ps} + A_s f_y}{0.85 f'_c b}$$

Also, d_p and d are the distances from extreme compression fibers to the centroid of the prestressed and nonprestressed steels, respectively. For flanged sections,

$$M_n = A_{pw} f_{ps} \left(d_p - \frac{a}{2}\right) + A_s f_y \left(d - \frac{a}{2}\right) + A_{pf} f_{ps} \left(d_p - \frac{h_f}{2}\right) \tag{19.45}$$

where

$$A_{pw} = A_{ps} - A_{pf}$$

$$a = \frac{A_{ps} f_{ps} + A_s f_y}{0.85 f'_c b_w}$$

For rectangular sections with compression reinforcement, and taking moments about the force C,

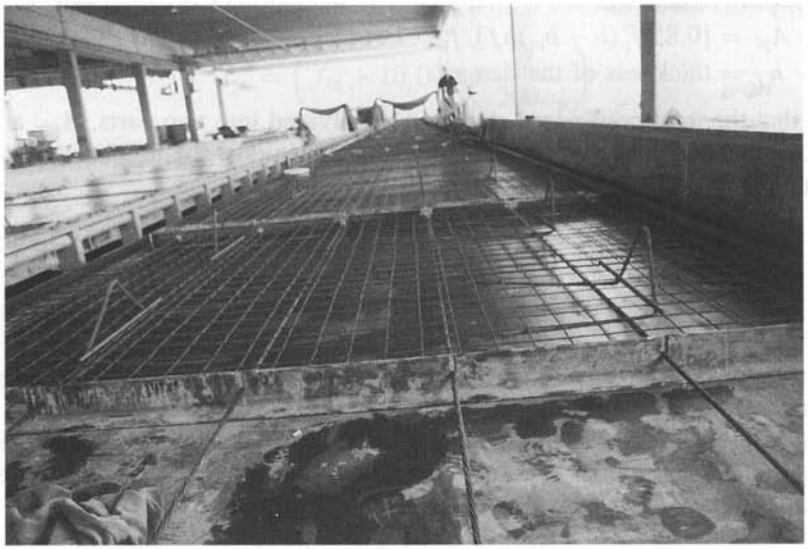
$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2}\right) + A_s f_y \left(d - \frac{a}{2}\right) + A'_s f_y \left(\frac{a}{2} - d'\right) \tag{19.46}$$

where

$$a = \frac{A_{ps} f_{ps} + A_s f_y - A'_s f_y}{0.85 f'_c b}$$

This equation is valid only if compression steel yields. The condition for compression steel to yield is

$$\left(\frac{A_{ps} f_{ps} + A_s f_y - A'_s f_y}{bd}\right) \geq 0.85 \beta_1 \frac{f'_c d'}{d} \left(\frac{87}{87 - f_y}\right)$$



Prestressed concrete beds for slabs and wall panels.

If this condition is not met, then compression steel does not yield. In this case, A_s' may be neglected (let $A_s' = 0$), or alternatively, the stress in A_s' may be determined by general analysis, as explained in Chapter 3.

When prestressed and nonprestressed reinforcement are used in the same section, Eq. 19.39 should read as follows:

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')\right)\right] \tag{19.47}$$

(ACI Code, Eq. 18.3). If any compression reinforcement is taken into account when calculating f_{ps}, the term

$$\rho_p \frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')$$

must be greater than or equal to 0.17 and d' must be less than or equal to $0.15d_p$, where d, d', and d_p are the distances from the extreme compression fibers to the centroid of the nonprestressed tension steel, compression steel, and prestressed reinforcement, respectively,

γ_p = factor for type of prestressing tendon

$= 0.55$ for $\frac{f_{py}}{f_{pu}}$ not less than 0.8

$= 0.40$ for $\frac{f_{py}}{f_{pu}}$ not less than 0.85

$= 0.28$ for $\frac{f_{py}}{f_{pu}}$ not less than 0.90

$\beta_1 = 0.85$ for $f_c' \leq 4$ ksi less 0.05 for each 1 ksi increase in f_c', but $\beta_1 \geq 0.65$.

1. For rectangular sections, the ACI Code, Section 18.8, limits the reinforcement ratio as follows ($\varepsilon_t \geq 0.005$ for tension-controlled sections):

$$\omega_p + \frac{d}{d_p}\omega \leq 0.32\beta_1$$

where

$$\omega_p = \rho_p\left(\frac{f_{ps}}{f_c'}\right) \quad \text{and} \quad \rho_p = \frac{A_{ps}}{bd} \quad \text{(prestressed steel)}$$

$$\omega = \rho\left(\frac{f_y}{f_c'}\right) \quad \text{and} \quad \rho = \frac{A_s}{bd} \quad \text{(nonprestressed steel)}$$

2. If ordinary reinforcing bars A_s' are used in the compression zone, then the condition becomes

$$\omega_p + \frac{d}{d_p}(\omega - \omega') \leq 0.32\beta_1$$

where $\omega' = \rho'(f_y/f_c')$ and $\rho' = A_s'/bd$. This reinforcement limitation is necessary to ensure a plastic failure of underreinforced concrete beams.

3. For flanged sections, the steel area required to develop the strength of the web (A_{pw}) is used to check the reinforcement index.

$$\omega_{pw}(\text{web}) = \rho_{pw}\left(\frac{f_{ps}}{f'_c}\right) \leq 0.32\beta_1$$

where

$$\rho_{pw} = \frac{A_{pw}}{b_w d_d}$$

If nonprestressed reinforcement is used, then the reinforcement limitations are

$$\omega_{pw} + \frac{d}{d_{pw}}(\omega_w - \omega'_w) \leq 0.32\beta_1$$

where

$$\omega_w \quad \text{and} \quad \omega'_w = \frac{A_s}{b_w d}\left(\frac{f_y}{f'_c}\right) \quad \text{and} \quad \frac{A'_s}{b_w d}\left(\frac{f_y}{f'_c}\right)$$

respectively. When compression steel A'_s is not used, then $\omega'_w = 0$. The preceding reinforcement conditions must be met in the analysis and design of partially prestressed concrete members.

For class C of prestressed concrete flexural members, where $f_t > 12\sqrt{f'_c}$ (cracked section), crack control provisions should be used as explained in Section 6.7 of this textbook. When using Eq. 6.18 for the maximum spacing s, the ACI Code, Section 18.4.4, specifies the following:

a. For tendons, use $\frac{2}{3}$ of the spacing s.

b. For a combination of nonprestressed reinforcement and tendons, use $\frac{5}{6}$ of the spacing s.

c. For tendons, use Δf_{ps} in place of f_s, where Δf_{ps} is the difference between the stress computed in the prestressing tendons at service load based on a cracked section and the decompression stress, f_{dc}, in the prestressing tendons, which may be taken conservatively, to be equal to the effective prestress, f_{se}. Note that Δf_{ps} should not exceed 36 ksi. If it is less than or equal to 20 ksi, the spacing requirement will not apply.

Equation 8.18 can be written as follows:

$$s = \left(\frac{2}{3}\right)\left[15\left(\frac{40}{\Delta f_{ps}}\right) - 2.5C_c\right]$$

19.6 CRACKING MOMENT

Cracks may develop in a prestressed concrete beam when the tensile stress at the extreme fibers of the critical section equals or exceeds the modulus of rupture of concrete, f_r. The value of f_r for normal-weight concrete may be assumed to be equal to $7.5\lambda\sqrt{f'_c}$ where $\lambda = 1.0$. The stress at the bottom fibers of a simply supported beam produced by the prestressing force and the cracking moment is

$$\sigma_b = -\frac{F}{A} - \frac{(Fe)y_b}{I} + \frac{M_{cr}y_b}{I}$$

When $\sigma_b = f_r = 7.5\sqrt{f'_c}$, then the cracking moment is

$$M_{cr} = \frac{I}{y_b}\left(7.5\lambda\sqrt{f'_c} + \frac{F}{A} + \frac{(Fe)y_b}{I}\right) \tag{19.48}$$

The maximum tensile stress after all losses is $7.5\lambda\sqrt{f'_c}$, which represents f_r. In this case, prestressed concrete beams may remain uncracked at service loads. To ensure adequate strength against cracking, the ACI Code, Section 18.8.2, requires that the factored moment of the member ϕM_n be at least 1.2 times the cracking moment, M_{cr}.

Example 19.5

For the beam of Example 19.4, check the design strength and cracking moment against the ACI Code requirements.

Solution

1. Check if the stress block depth a lies within the flange.

$$a = \frac{A_{ps}\, f_{ps}}{0.85 f'_c b} \tag{19.35}$$

$$A_{ps}\left(\text{of 20 tendons } \frac{7}{16} \text{ in. in diameter}\right) = 2.178 \text{ in.}^2$$

Let $f_{py}/f_{pu} = 0.85$, $\rho_p = 0.4$, and $\gamma_p/\beta_1 = 0.4/0.8 = 0.5$. For bonded tensions,

$$f_{ps} = f_{pu}\left(1 - \frac{\gamma_p}{\beta_1}\rho_p \times \frac{f_{pu}}{f'_c}\right) \tag{19.39}$$

$$d = 40 - 6.3 = 33.7 \text{ in.}$$

$$\rho_p = \frac{A_{ps}}{bd} = \frac{2.178}{18 \times 33.7} = 0.00359$$

Given $f_{pu} = 250$ ksi,

$$f_{ps} = 250\left[1 - 0.5(0.00359) \times \frac{250}{5}\right] = 228 \text{ ksi}$$

$$a = \frac{2.178 \times 228}{0.85 \times 5 \times 18} = 6.5 \text{ in.}$$

which is greater than 6 in. Therefore, the section acts as a flanged section.

2. For flanged sections,

$$M_n = A_{pw} f_{ps}\left(d - \frac{a}{2}\right) + A_{pf} f_{ps}\left(d - \frac{h_f}{2}\right)$$

where

$$A_{pw}\text{ (web)} = A_{ps} - A_{pf}\text{ (flange)}$$

$$A_{pf} = \frac{1}{f_{ps}}[0.85 f'_c(b - b_w)h_f]$$

$$= \frac{1}{228}[0.85 \times 5(18 - 6)6] = 1.342 \text{ in.}^2$$

$$A_{pw} = 2.178 - 1.342 = 0.836 \text{ in.}^2$$

$$a = \frac{A_{pw} f_{ps}}{0.85 f'_c b_w} = \frac{0.836(228)}{0.85 \times 5 \times 6} = 7.5 \text{ in.}$$

$$M_n = 0.836(228)\left(33.7 - \frac{7.5}{2}\right) + 1.342 \times 228\left(33.7 - \frac{6}{2}\right)$$

$$= 15{,}102 \text{ K·in.} = 1258.5 \text{ K·ft}$$

$$\phi M_n = 0.9(1258.5) = 1132.7 \text{ K·ft}$$

Check the reinforcement index for the flanged section:

$$\rho_{pw} \text{ (web)} = \frac{A_{pw}}{b_w d} = \frac{0.836}{6 \times 33.7} = 0.00413$$

$$\omega_{pw} \text{ (web)} = \rho_{pw} \frac{f_{ps}}{f'_c} \leq 0.32\beta_1 = 0.32 \times 0.8 = 0.256$$

($\beta_1 = 0.8$ for $f'_c = 5$ ksi.)

$$\omega_{pw} = 0.00413 \frac{(228)}{5} = 0.188 < 0.256 \qquad (\phi = 0.9)$$

3. Calculate the external factored moment due to dead and live loads.

$$\text{Dead load} = \text{self-weight} + \text{additional dead load}$$

$$= 0.388 + 0.9 = 1.29 \text{ K/ft}$$

$$\text{Live load} = 1.1 \text{ K/ft}$$

$$U = 1.2D + 1.6L$$

$$M_u = \frac{(48)^2}{8}[1.2(1.29) + 1.6(1.1)] = 952.7 \text{ K·ft}$$

This external moment is less than the factored moment capacity of the section of 1132.7 K·ft; therefore, the section is adequate.

4. The cracking moment (Eq. 19.48) is

$$M_{cr} = \frac{I}{y_b}\left(7.5\lambda\sqrt{f'_c} + \frac{F}{A} + (Fe)\frac{y_b}{I}\right)$$

From Example 19.4, $F = 306.2$ K, $A = 372$ in.^{2}, $e = 14.5$ in., $y_b = 20.8$ in., $I = 66{,}862$ in.^{4}, $f'_c = 5$ ksi, and $7.5\lambda\sqrt{f'_c} = 7.55000 = 530$ psi.

$$M_{cr} = \frac{66{,}862}{20.8}\left[0.53 + \frac{306.2}{372} + \frac{(306.2)(14.5)(20.8)}{66{,}862}\right]$$

$$= 8790 \text{ K·in.} = 732.5 \text{ K·ft}$$

Check that $1.2M_{cr} \leq \phi M_n$.

$$1.2\,M_{cr} = 1.2\,(732.5) = 879 \text{ K·ft}$$

This value is less than $\phi M_n = 1132.7$ K·ft. Thus, the beam is adequate against cracking.

19.7 DEFLECTION

Deflection of a point in a beam is the total movement of the point, either downward or upward, due to the application of load on that beam. In a simply supported prestressed concrete beam, the prestressing force is usually applied below the centroid of the section, causing an upward deflection called *camber*. The self-weight of the beam and any external gravity loads acting on

the beam will cause a downward deflection. The net deflection will be the algebraic sum of both deflections.

In computing deflections, it is important to consider both the short-term, or immediate, deflection and the long-term deflection. To ensure that the structure remains serviceable, the maximum short- and long-term deflections at all critical stages of loading must not exceed the limiting values specified by the ACI Code (see Section 6.3 in this text).

The deflection of a prestressed concrete member may be calculated by standard deflection equations or by the conventional methods given in books on structural analysis. For example, the midspan deflection of a simply supported beam subjected to a uniform gravity load w is equal to $(5wL^4/384EI)$. The modulus of elasticity of concrete is $E_c = 33\omega^{1.5}\sqrt{f'_c} = 57{,}000\sqrt{f'_c}$ for normal-weight concrete.

The moment of inertia of the concrete section I is calculated based on the properties of the gross section for an uncracked beam. This case is appropriate when the maximum tensile stress in the concrete extreme fibers does not exceed the modulus of rupture of concrete, $f_r = 7.5\sqrt{f'_c}$ (class U beams). When the maximum tensile stress based on the properties of the gross section exceeds $7.5\sqrt{f'_c}$, the effective moment of inertia, I_e, based on the cracked and uncracked sections must be used as explained in Chapter 6 (class T and C beams). Typical midspan deflections for simply supported beams due to gravity loads and prestressing forces are shown in Table 19.3.

Example 19.6

For the beam of Example 19.4, calculate the camber at transfer and then calculate the final anticipated immediate deflection at service load.

Solution

1. Deflection at transfer:
   a. Calculate the downward deflection due to dead load at transfer, self-weight in this case. For a simply supported beam subjected to a uniform load,

$$\Delta_D \text{ (dead load)} = \frac{5wL_4}{384EI}$$

   From Example 19.4, $w_D = 388$ lb/ft, $L = 48$ ft, $E_{ci} = 3600$ ksi, and $I = 66{,}862$ in.^{4}

$$\Delta_D = \frac{5(0.388/12)(48 \times 12)^4}{384(3600)(66{,}862)} = 0.192 \text{ in.} \qquad \text{(downward)}$$

   b. Calculate the camber due to the prestressing force. For a simply supported beam harped at one-third points with the eccentricity $e_1 = 14.5$ in. at the middle third and $e_2 = 0$ at the ends,

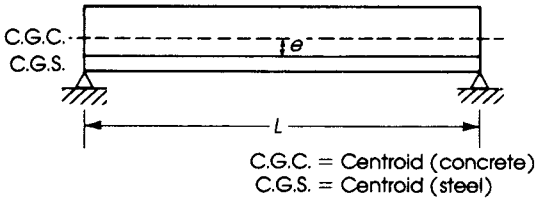
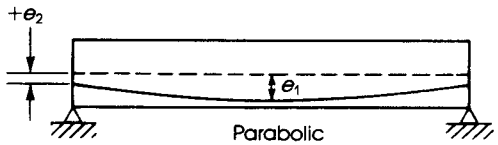
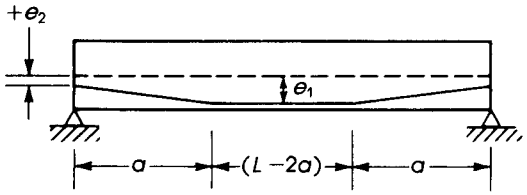
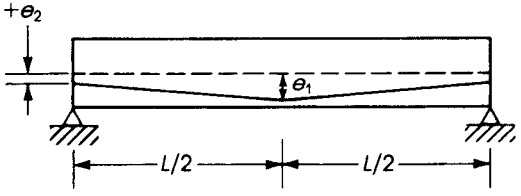
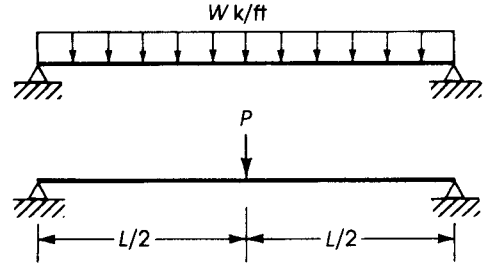
$$\Delta_p = \frac{23(F_i e_1)L^2}{216E_{ci}I} \qquad \text{(Table 19.3)}$$

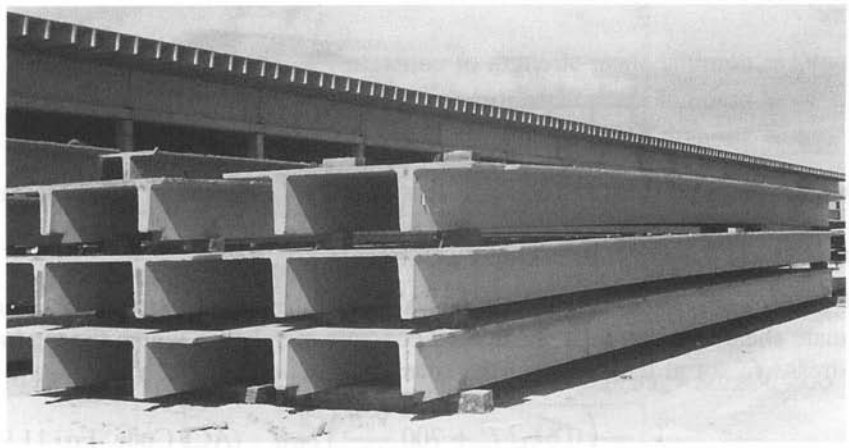
$$= \frac{23(365.9 \times 14.5)(48 \times 12)^2}{216(3600)(66,862)} = -0.779 \text{ in.} \qquad \text{(upward)}$$

   c. Final camber at transfer is $-0.779 + 0.192 = -0.587$ in. (upward).
2. Deflection at service load: The total uniform service load is $W_T = 0.388 + 0.9 + 1.1 = 2.388$ K/ft, and $E_c = 4000$ ksi. The downward deflection due to W_T is

$$\Delta_w = \frac{5W_T L^4}{384E_c I} = \frac{5(2.388/12)(48 \times 12)^4}{384(4000)(66,862)} = +1.067 \text{ in.} \qquad \text{(downward)}$$

Table 19.3 Midspan Deflections of Simply Supported Beams

Schematic	Deflection Equations
Camber due to prestressing force	
	$\Delta = \frac{Fe)L^2}{8\,EI}$ (1) (Horizontal tendons)
	$\Delta = \frac{FL^2}{8\,EI}\left[\frac{5}{6}e_1 + \frac{1}{6}e_2\right]$ (2) When $e_2 = 0$: $\Delta = \frac{5(Fe_1)L^2}{48\,EI}$ (3)
	$\Delta = \frac{FL^2}{8\,EI}\left[e_1 + \frac{4}{3}\left(\frac{a}{L}\right)^2 (e_2 - e_1)\right]$ (4) When $a = \frac{L}{3}$: $\Delta = \frac{FL^2}{8\,EI}\left[e_1 + \frac{4}{27}(e_2 - e_1)\right]$ (5) When $a = \frac{L}{3}$ and $e_2 = 0$: $\Delta = \frac{23(Fe_1)L^2}{216\,EI}$ (6)
	$\Delta = \frac{FL^2}{24\,EI}[2e_1 + e_2]$ (7) When $e_2 = 0$: $\Delta = \frac{(Fe_1)L^2}{12\,EI}$ (8)
Deflection due to gravity loads	
	$\Delta = \frac{5\,wL^4}{384\,EI}$ (9) $\Delta = \frac{PL^3}{48\,EI}$ (10)



Upward deflection (camber) in double-T prestressed concrete beams.

The camber due to prestressing force $F = 306.2\,\text{K}$ and $E_c = 4000\,\text{ksi}$ is

$$\Delta_p = \frac{23(306.2 \times 14.5)(48 \times 12)^2}{216(4000)(66{,}862)} = -0.587 \text{ in.} \qquad \text{(upward)}$$

The final immediate deflection at service load is

$$\Delta = \Delta_w - \Delta_p = 1.067 - 0.587 = +0.48 \text{ in.} \qquad \text{(downward)}$$

19.8 DESIGN FOR SHEAR

The design approach to determine the shear reinforcement in a prestressed concrete beam is almost identical to that used for reinforced concrete beams. Shear cracks are assumed to develop at 45° measured from the axis of the beam. In general, two types of shear-related cracks form. One type is due to a combined effect of flexure and shear: The cracks start as flexural cracks and then deviate and propagate at an inclined direction due to the effect of diagonal tension. The second type, web-shear cracking, occurs in beams with narrow webs when the magnitude of principal tensile stress is high in comparison to flexural stress. Stirrups must be used to resist the principal tensile stresses in both cases. The ACI design criteria for shear will be adopted here.

19.8.1 Basic Approach

The ACI design approach is based on ultimate strength requirements using the load factors mentioned in Chapter 3. When the factored shear force, V_u, exceeds half the nominal shear strength ($\phi V_c/2$), shear reinforcement must be provided. The required design shear force, V_u, at each section must not exceed the nominal design strength, ϕV_n, of the cross-section based on the combined nominal shear capacity of concrete and web reinforcement:

$$V_u \leq \phi V_u \leq \phi(V_c + V_s) \tag{19.49}$$

where

V_c = nominal shear strength of concrete
V_s = nominal shear capacity of reinforcement
ϕ = strength reduction factor = 0.75

When the factored shear force, V_u, is less than $\frac{1}{2}\phi V_c$, minimum shear reinforcement is required.

19.8.2 Shear Strength Provided by Concrete

The ACI Code, Section 11.3, presents a simple empirical expression to estimate the nominal ultimate shear capacity of a prestressed concrete member in which the tendons have an effective prestress, f_{se}, of at least 40% of the specified tensile strength, f_{pu}:

$$V_c = \left(0.6\lambda\sqrt{f'_c} + 700\frac{V_u d}{M_u}\right) b_w d \quad \text{(ACI Code, Eq. 11.9)} \tag{19.50}$$

where

V_u and M_u = factored shear and moment at the section under consideration
b_w = width of web
d (in the term $\frac{V_u d}{M_u}$) = the distance from the compression fibers to the centroid of the prestressing steel
d(in V_{ci} or V_{cw} equations) = the larger of the above d or $0.8h$ (ACI Code, Section 11.3.1)

The use of Eq. 19.50 is limited to the following conditions:

1. The quantity $V_u d/M_u \leq 1.0$ (to account for small values of V_u and M_u)
2. $V_c \geq (2\lambda\sqrt{f'_c})b_w d$(minimum V_c)
3. $V_c \leq (5\lambda\sqrt{f'_c})b_w d$(maximum V_c)

The variation of the concrete shear capacity for a simply supported prestressed concrete beam subjected to a uniform load is shown in Fig. 19.8. Note that the maximum shear reinforcement may be required near the supports and near one-fourth of the span where ϕV_s reaches maximum values. In contrast, similar reinforced concrete beams require maximum shear reinforcement (or minimum spacings) only near the support where maximum ϕV_s develops.

The values of V_c calculated by Eq. 19.50 may be conservative sometimes; therefore, the ACI Code, Section 11.3.3, gives an alternative approach for calculating V_c that takes into consideration the additional strength of concrete in the section. In this approach, V_c is taken as the smaller of two calculated values of the concrete shear strength V_{ci} and V_{cw} (Fig. 19.8). Both are explained next.

The shear strength, V_{ci}, is based on the assumption that flexural-shear cracking occurs near the interior extremity of a flexural crack at an approximate distance of $d/2$ from the load point in the direction of decreasing moment. The ACI Code, Section 11.3.3.1 specifies that V_{ci} be computed as follows:

$$V_{ci} = (0.6\lambda\sqrt{f'_c})b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \tag{19.51}$$

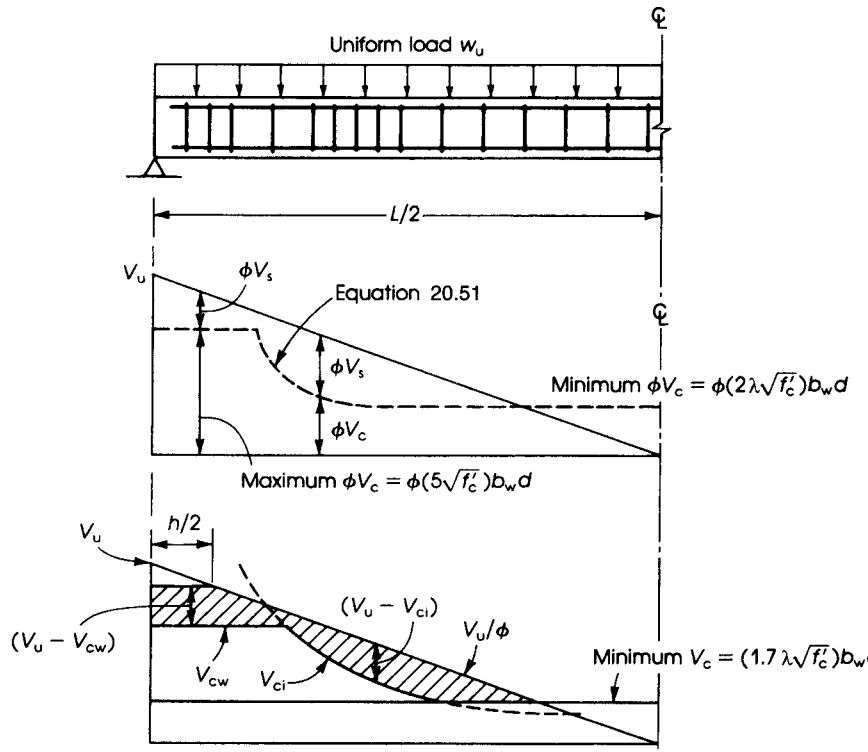


Figure 19.8 Distribution of shear forces along span. The middle diagram shows shear capacity of a simply supported prestressed concrete beam. The bottom diagram shows ACI analysis. (Stirrups are required for shaded areas).

but it is not less than $(1.7\lambda\sqrt{f'_c})b_w d$, where

V_d = shear force at section due to unfactored dead load
V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}
M_{max} = maximum factored moment at the section due to externally applied loads
M_{cr} = cracking moment

The cracking moment can be determined from the following expression:

$$M_{cr} = \frac{I}{y_t}(6\lambda\sqrt{f'_c} + f_{pe} - f_d) \quad \text{(ACI Code, Eq. 11.11)} \tag{19.52}$$

where

I = moment of inertia of the section resisting external factored loads
y_t = distance from the centroidal axis of the gross section neglecting reinforcement to the extreme fiber in tension
f_{pe} = compressive strength at the extreme fibers of the concrete section due to the effective prestress force after all losses

f_d = stress due to the unfactored dead load at the extreme fiber, where tensile stress is caused by external loads

λ = modification factor for concrete

The web-shear strength, V_{cw}, is based on shear cracking in a beam that has not cracked by flexure. Such cracks develop near the supports of beams with narrow webs. The ACI Code, Section 11.3.3.2, specifies that V_{cw} be computed as follows:

$$V_{cw} = (3.5\lambda\sqrt{f'_c} + 0.3f_{pc})b_w d + V_p \tag{19.53}$$

where

V_p = vertical component of the effective prestress force at the section considered

f_{pc} = compressive stress (psi) in the concrete (after allowance for prestress losses) at the centroid of the section resisting the applied loads or at the junction of the web and flange when the centroid lies within the flange

Alternatively, V_{cw} may be determined as the shear force that produces a principal tensile stress of $4\lambda\sqrt{f'_c}$ at the centroidal axis of the member or at the intersection of the flange and web when the centroid lies within the flange. The equation for the principal stresses may be expressed as follows:

$$f_t = 4\lambda\sqrt{f'_c} = \sqrt{v_{cw}^2 + \left(\frac{f_{pc}}{2}\right)^2} - \frac{f_{pc}}{2}$$

or

$$V_{cw} = f_t\left(\sqrt{\frac{1 + f_{pc}}{f_t}}\right)b_w d \tag{19.54}$$

where $f_t = 4\lambda\sqrt{f'_c}$. When applying Eqs. 19.51 and 19.53 or 19.54, the value of d is taken as the distance between the compression fibers and the centroid of the prestressing tendons but is not less than $0.8h$.

The critical section for maximum shear is to be taken at $h/2$ from the face of the support. The same shear reinforcement must be used at sections between the support and the section at $h/2$.

19.8.3 Shear Reinforcement

The value of V_s must be calculated to determine the required area of shear reinforcement.

$$V_u = \phi(V_c + V_s) \tag{19.49}$$

$$V_s = \frac{1}{\phi}(V_u - \phi V_c) \tag{19.55}$$

For vertical stirrups,

$$V_s = \frac{A_v f_y d}{s} \tag{19.56}$$

and

$$A_v = \frac{V_s s}{f_y d} \quad \text{or} \quad s = \frac{A_v f_y d}{V_s} \tag{19.57}$$

where A_v = area of vertical stirrups and s = spacing of stirrups. Equations for inclined stirrups are the same as those discussed in Chapter 8.

19.8.4 Limitations

1. Maximum spacing, s_{max}, of the stirrups must not exceed $3h/4$ or 24 in. If V_s exceeds $4\sqrt{f'_c}b_w d$, the maximum spacing must be reduced to half the preceding values (ACI Code, Section 11.4.4).
2. Maximum shear, V_s, must not exceed $8\sqrt{f'_c}b_w d$; otherwise, increase the dimensions of the section (ACI Code, Section 11.4.7.9).
3. The minimum shear reinforcement, A_v, required by the ACI Code is

$$A_{v\,\min} = \frac{50 b_w s}{f_y} \leq 0.75\sqrt{f'_c}\left(\frac{b_w s}{f_y}\right) \tag{19.58}$$

When the effective prestress, f_{pe}, is greater than or equal to 0.4 f_{pe}, the minimum A_v is

$$A_v = \frac{A_{ps}}{80} \times \frac{f_{pu}}{f_y} \times \frac{s}{d} \times \frac{d}{b_w} \tag{19.59}$$

The effective depth, d, need not be taken less than $0.8h$. Generally, Eq. 19.59 requires greater minimum shear reinforcement than Eq. 19.58.

Example 19.7

For the beam of Example 19.4, determine the nominal shear strength and the necessary shear reinforcement. Check the sections at $h/2$ and 10 ft from the end of the beam. Use f_y = 60 ksi for the shear reinforcement, and a live load = 1.33 K/ft. using normal-weight concrete.

Solution

1. For the section at $h/2$:

$$\frac{h}{2} = \frac{40}{2} = 20 \text{ in.} = 1.67 \text{ ft from the end}$$

2. The factored uniform load on beam is

$$W_u = 1.2(0.388 + 0.9) + 1.6 \times 1.33 = 3.68 \text{ K/ft}$$

$$V_u \text{ at a distance } \frac{h}{2} = 3.68(24 - 1.67) = 82.2 \text{ K}$$

Using the simplified ACI method (Eq. 19.50), determine M_u at section $h/2$.

$$M_u = (3.68 \times 21) \times 1.67 - 3.68\frac{(1.67)^2}{2} = 142.4 \text{ K·ft} = 1708 \text{ K·in.}$$

The value of d at section $h/2$ from the end (Fig. 19.6*b*) is

$$d = 33.7 \text{ (at midspan)} - \frac{(16 - 1.67)}{16} \times 14.5 = 20.7 \text{ in.}$$

$$\frac{V_u d}{M_u} = \frac{82.2 \times 20.7}{1708} = 0.966 \leq 1.0$$

as required by the ACI Code.

$$V_c = \left(0.6\lambda\sqrt{f'_c} + 700\frac{V_u d}{M_u}\right) b_w d$$

$$= (0.6 \times 1 \times \sqrt{5000} + 700 \times 0.996)6 \times 20.7 = 91{,}800 \text{ lb} = 91.8 \text{ K}$$

$$\text{Minimum } V_c = 2\lambda\sqrt{f'_c} b_w d = 2 \times 1 \times \sqrt{5000} \times 6 \times 20.7 = 17.6 \text{ K}$$

$$\text{Maximum } V_c = 5\lambda\sqrt{f'_c} b_w d = 43.9 \text{ K}$$

The maximum V_c of 43.9 K controls.

3. The alternative approach presented by the ACI Code is that V_c may be taken as the smaller value of V_{ci} and V_{cw}.

a. Based on the flexural-shear cracking strength,

$$V_{ci} = (0.6\lambda\sqrt{f'_c})b_w d + \left(V_d + \frac{V_j M_{cr}}{M_{max}}\right) \tag{19.51}$$

Calculate each item separately:

$$(0.6\lambda\sqrt{f'_c})b_w d = 0.6 \times 1 \times \sqrt{5000} \times 6 \times 20.7 = 5.3 \text{ K}$$

V_d = unfactored dead load shear = $1.288(24 - 1.67) = 28.8$ K

M_{max} = maximum factored moment at section (except for weight of beam)

Factored load = $1.2 \times 0.9 + 1.6 \times 1.3 = 3.13$ K/ft

$$M_{max} = 3.13\left[24 \times 1.67 - \frac{(1.67)^2}{2}\right] = 121 \text{ K/ft} = 1453 \text{ K·in.}$$

$$V_i = 3.13(24 - 1.67) = 69.9 \text{ K}$$

$$M_{cr} = \frac{I}{y_t}(6\lambda\sqrt{f'_c} + f_{pe} - f_d)$$

$I = 66{,}862 \text{ in.}^4 \qquad y_b = 20.8 \text{ in.}$

f_e = compressive stress due to prestressing force

$$= \frac{F}{A} + \frac{Fey_b}{I}$$

$$= \frac{306.2}{372} + \frac{306.2(1.5)(20.8)}{66{,}862} = 0.966 \text{ ksi}$$

$$f_d = \text{dead load stress} = \frac{M_D y_b}{I}$$

$$M_D = (1.288)\left[24 \times 1.67 - \frac{(1.67)^2}{2}\right] = 49.8 \text{ K·ft} = 598 \text{ K·in.}$$

$$f_d = \frac{598 \times 20.8}{66{,}862} = 0.186 \text{ ksi}$$

$$M_{cr} = \frac{66{,}862}{20.8}(6\sqrt{5000} + 966 + 186) = 3871 \text{ K·in.}$$

Therefore,

$$V_{ci} = (5.3) + (28.8) + 69.9\left(\frac{3871}{1453}\right) = 220.3 \text{ K}$$

V_{ci} must not be less than $(1.7\lambda\sqrt{f'_c})b_w d = (1.7 \times 1 \times \sqrt{5000}) \times 6 \times 20.7 = 14.9$ K.

b. Shear strength based on web-shear cracking is

$$V_{cw} = (3.5\lambda\sqrt{f'_c} + 0.3f_{pc})b_w d + V_p$$
$$f_{pc} = \frac{306.2}{372} = 0.823 \text{ ksi} \tag{19.53}$$
$$d = 20.7 \text{ in.} \quad \text{or} \quad 0.8h = 0.8 \times 40 = 32 \text{ in.}$$

Use $d = 32$ in.

$$V_p = 306.2 \times \frac{1}{13.2} = 23.2 \text{ K}$$

where 1/13.2 = slope of tendon profile = 14.5 in./(16 × 12).

$$3.5\lambda\sqrt{f'_c} = 3.5 \times 1 \times \sqrt{5000} = 248 \text{ psi}$$

Therefore,

$$V_{cw} = (0.248 + 0.3 + 0.823) \times 6 \times 32 + 23.2 = 118.2 \text{ K}$$

c. Because $V_{cw} < V_{ci}$ the value $V_{cw} = 118.2$ K represents the nominal shear strength at section $h/2$ from the end of the beam. In most cases, V_{cw} controls at $h/2$ from the support.

4. Web reinforcement:

$$V_u = 82.3\text{K} \qquad \phi V_{cw} = 0.75 \times 118.2 = 88.65\text{K}$$

Because $V_u < \phi V_{cw}$, $V_s = 0$; therefore, use minimum stirrups. Use no. 3 stirrups. $A_v = 2 \times 0.11 = 0.22$ in.^{2} Maximum spacing is the least of

$$s_1 = \frac{3}{4}h = \frac{3}{4} \times 40 = 30 \text{ in.} \qquad s_2 = 24 \text{ in.}$$

Calculate s_3 from the equation of minimum web reinforcement:

$$\text{Min. } A_v = \frac{A_{ps}}{80} \times \frac{f_{pu}}{f_y} \times \frac{s}{d} \times \sqrt{\frac{d}{b_w}}$$
$$0.22 = \frac{2.178}{80} \times \frac{250}{60} \times \frac{s_3}{20.7}\sqrt{\frac{20.7}{6}} \tag{19.59}$$
$$s_3 = 21.6 \text{ in.} \qquad (20 \text{ in.})$$

Also,

$$\text{Min. } A_v = \frac{50b_w s}{f_y} \le 0.75\sqrt{f'_c}\left(\frac{b_w s}{f_y}\right), \qquad 0.75\sqrt{f'_c} = 53$$
$$s_4 = \frac{A_v f_y}{53b_w} = \frac{0.22 \times 60{,}000}{53 \times 6} = 41.5 \text{ in.}$$

$s_{max} = s_3 = 20$ in. controls. Thus, use no. 3 stirrups spaced at 20 in.

5. For the section at 10 ft from the end, the calculation procedure is similar to that for the section at $h/2$. Using the ACI simplified method,

$$V_u = 3.68(24 - 10) = 51.5 \text{ K}$$
$$M_u = 3.68\left[24 \times 10 - \frac{(10)^2}{2}\right] = 699.2 \text{ K·ft} = 8390 \text{ K·in.}$$
$$d = 33.7 \text{ (at midspan)} - \frac{(16 \times 10)}{16} \times 14.5 = 28.3 \text{ in.}$$

$$\frac{V_u d}{M_u} = \frac{515 \times 28.3}{8390} = 0.174 < 1.0$$

$$V_c = (0.6 \times 1 \times \sqrt{5000} + 0.174 \times 700)6 \times 28.3 = 27{,}886 \text{ lb} = 27.9 \text{ K} \quad \text{(controls)}$$

Minimum $V_c = 17.6$ K and maximum $V_c = 43.9$ K.

6. Using the ACI Code equations to compute V_{ci} and V_{cw}, calculate V_{ci} first (which controls at this section):

$$0.6\lambda\sqrt{f'_c}b_w d = 0.6 \times 1 \times \sqrt{5000} \times 6 \times 28.3 = 7.2 \text{ K}$$

$$V_d = 1.288(24 - 10) = 18 \text{ K}$$

$$M_{\max} = 3.13\left[24 \times 10 - \frac{(10)^2}{2}\right] = 594.7 \text{ K·ft} = 7136 \text{ K·in.}$$

$$V_i = 3.13(24 - 10) = 43.8 \text{ K}$$

$$f_{pe} = \frac{306.2}{372} + \frac{306.2(9.1)(20.8)}{66{,}862} = 1.69 \text{ ksi}$$

$$M_D = 1.288\left[24 \times 10 - \frac{(10)^2}{2}\right] = 244.7 \text{ K·ft} = 2937 \text{ K·in.}$$

$$f_d = \frac{2937 \times 20.8}{66{,}862} = 0.914 \text{ ksi} \qquad M_{cr} = 3858 \text{ K·in.}$$

Therefore,

$$V_{ci} = 7.2 + 18 + \frac{43.8(3858)}{7136} = 48.9 \text{ K}$$

$$V_{ci\ min} = (1.7 \times 1 \times \sqrt{5000})6 \times 28.3 = 20.4 \text{ K}$$

Thus the minimum is met. Then calculate V_{cw}:

$$f_{pc} = 0.893 \text{ ksi} \qquad V_p = 23.2 \text{ K} \qquad \text{(as before)}$$

$$d = 28.3 \text{ in.} \quad \text{or} \quad 0.8h = 32 \text{ in.}$$

Use $d = 32$ in.

$$V_{cw} = (3.5\lambda\sqrt{f'_c} + 0.3f_{pc})b_w d + V_p$$

$$= (0.248 + 0.3 \times 0.823)6 \times 32 + 23.2 = 118.2 \text{ K}$$

This value of V_{cw} is not critical. At about span/4, the critical shear value is V_{ci} (Fig. 19.8).

7. To calculate web reinforcement,

$$V_u = 51.5 \text{ K} \quad \phi V_{ci} = 0.75 \times 48.9 = 36.7 \text{ K}$$

$$V_u = \phi(V_c + V_s)$$

$$V_s = \frac{1}{0.75}(51.5 - 36.7) = 19.7 \text{ K}$$

Use no. 3 stirrups; $A_v = 0.22 \text{ in}^2$. Check maximum spacing: $s_{\max} = 18$ in. (as before).

$$\text{Required } A_v = \frac{V_s s}{f_y d} = \frac{19.7 \times 18}{60 \times 28.3} = 0.209 \text{ in.}^2$$

A_v used is $0.22 \text{ in.}^2 > 0.209 \text{ in}^2$. Therefore, use no. 3 stirrups spaced at 14 in.

19.9 PRELIMINARY DESIGN OF PRESTRESSED CONCRETE FLEXURAL MEMBERS

19.9.1 Shapes and Dimensions

The detailed design of prestressed concrete members often involves a considerable amount of computation. A good guess at the dimensions of the section can result in a savings of time and effort. Hence it is important to ensure, by preliminary design, that the dimensions are reasonable before starting the detailed design.

At the preliminary design stage, some data are usually available to help choose proper dimensions. For example, the bending moments due to the applied external loads, the permissible stresses, and the data for assessing the losses are already known or calculated.

The shape of the cross-section of a prestressed concrete member may be a rectangular, T-, I-, or box section. The total depth of the section, h, may be limited by headroom considerations or may not be specified. Given the freedom of selection, an empirical practical choice of dimensions for a preliminary design is as follows (Fig. 19.9):

1. Total depth of section is $h = \frac{1}{20}$ to $\frac{1}{30}$ of the span L; for heavy loading $h = L/20$ and for light loading $h = L/30$ or $h = 2\sqrt{M_D + M_L}$, where M is in K·l ft.
2. The depth of top flange is $h_f = h/8$ to $h/6$.
3. The width of top flange is $b \geq 2h/5$.
4. The thickness of the web is $b_w \geq 4$ in. Usually b_w is taken as $h/30 + 4$ in.
5. b_w and t are chosen to accommodate and uniformly distribute the prestressing tendons, keeping appropriate concrete cover protection.
6. The approximate area of the concrete section required is

$$A_c(\text{ft}^2) = \frac{M_D + M_t}{30h}$$

where $M_D + M_L$ are in K·ft and h is in ft. In SI units,

$$A_c(\text{m}^2) = \frac{M_D + M_L}{1450h} \quad (M_D + M_L \text{in kN} \cdot \text{m and} \, h \text{ in m})$$

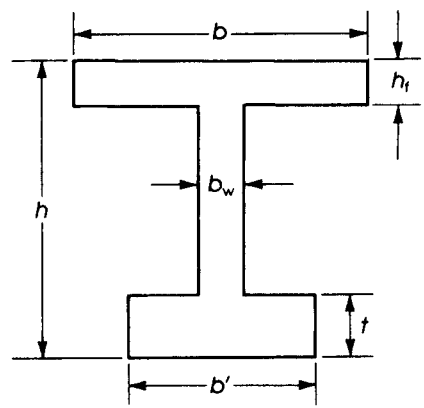


Figure 19.9 Proportioning prestressed concrete sections.

For practical and economical design of prestressed concrete beams and floor slabs, the precast concrete industry has introduced a large number of standardized shapes and dimensions from which the designer can choose an adequate member. Tables of standard sections are available in the PCI Design Handbook [3]. AASHTO [23] has also presented standard girders to be used in bridge construction (Table 19.4).

19.9.2 Prestressing Force and Steel Area

Once the shape, depth, and other dimensions of the cross-section have been selected, approximate values of the prestressing force and the area of the prestressing steel, A_{ps}, can be determined.

From the internal couple concept, the total moment, M_T, due to the service dead and live loads is equal to the tension force, T, times the moment arm, jd.

$$M_T = T(jd) = C(jd)$$

$$M_T = A_{ps} f_{se}(jd) \quad A_{ps} = \frac{M_T}{f_{se}(jd)}$$

where A_{ps} is the area of the prestressing steel and f_{se} is the effective prestressing stress after all losses. The value of the moment arm, jd, varies from $0.4h$ to $0.8h$, with a practical range of $0.6h$ to $0.7h$. An average value of 0.65 may be used. Therefore,

$$A_{ps} = \frac{M_T}{(0.65h) f_{se}} \tag{19.60}$$

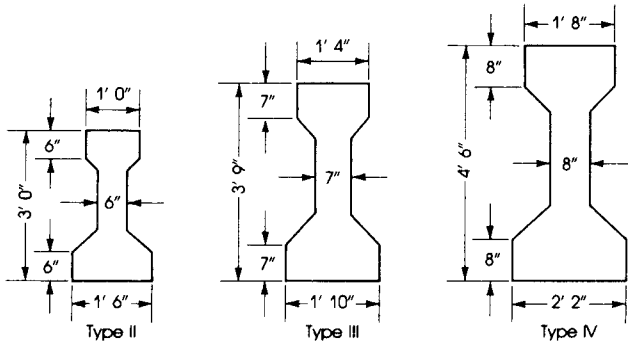
and the prestressing force is

$$F = T = A_{ps} f_{se} = \frac{M_T}{0.65h} \tag{19.61}$$

The prestressing force at transfer is $F_i = F/\eta$, where η is the factor of time-dependent losses.

Table 19.4 AASHTO Girders, Normal-Weight Concrete 25

Designation	A (in.^{2})	I (in.^{4})	y_b (in.)	Z_b (in.^{3})	Z_t (in.^{3})	Weight (lb/ft)
Type II	369	50,979	15.83	3220	2527	384
Type III	560	125,390	20.27	6186	5070	593
Type IV	789	260,741	24.73	10,544	8908	822



The compressive force, C, on the section is equal to the tension force, T:

$$C = T = A_{ps} f_{se}$$

In terms of stresses,

$$\frac{C}{A_c} = \frac{A_{ps} f_{se}}{A_c} = f_{c_1}$$

where f_{c_1} is an assumed uniform stress on the section.

For preliminary design, a triangular stress distribution is assumed with maximum allowable compressive stress, f_{ca}, on one extreme fiber; therefore, the average stress is $0.5 f_{ca} = f_{c_1}$. The allowable compressive stress in concrete is $f_{ca} = 0.45 f'_c$. Thus, the required concrete area, A_c, can be estimated from the force, T, as follows:

$$A_c = \frac{T}{f_{c_1}} = \frac{A_{ps} f_{se}}{f_{c_1}} = \frac{A_{ps} f_{se}}{0.5 f_{ca}} = \frac{A_{ps} f_{se}}{0.225 f'_c} \tag{19.62}$$

$$A_c = \frac{T}{0.5 f_{ca}} = \frac{M_T}{(0.65h)(0.5 f_{ca})} = \frac{M_T}{0.33 f_{ca}} = \frac{M_T}{0.15 f'_c} \tag{19.63}$$

This analysis is based on the design for service loads and not for the factored loads. The eccentricity, e, is measured from the centroid of the section to the centroid of the prestressing steel and can be estimated approximately as follows:

$$e = K_b + \frac{M_D}{F_i} \tag{19.64}$$

where K_b is the lower Kern limit and M_D is the moment due to the service dead load.

19.10 END-BLOCK STRESSES

19.10.1 Pretensioned Members

Much as a specific development length is required in every bar of a reinforced concrete beam, the prestressing force in a prestressed concrete beam must be transferred to the concrete by embedment or end anchorage or a combination thereof. In pretensioned members, the distance over which the effective prestressing force is transferred to the concrete is called the transfer length, l_t. After transfer, the stress in the tendons at the extreme end of the member is equal to 0, whereas the stress at a distance l_t from the end is equal to the effective prestress, f_{pe}. The transfer length, l_t, depends on the size and type of the tendon, surface condition, concrete strength, f'_c, stress, and method of force transfer. A practical estimation of l_t ranges between 50 and 100 times the tendon diameter. For strands, a practical value of l_t is equal to 50 tendon diameters, whereas for single wires, l_t is equal to 100 wire diameters.

In order that the tension in the prestressing steel develops full ultimate flexural strength, a bond length is required. The purpose is to prevent general slip before the failure of the beam at its full design strength. The development length, l_d, is equal to the bond length plus the transfer length, l_t. Based on established tests, the ACI Code, Section 12.9.1, gives the following expression for computing the development length of three- or seven-wire pretensioning strands:

$$l_d(\text{in.}) = \left(f_{ps} - \frac{2}{3} f_{se} \right) d_b \tag{19.65}$$

where

f_{ps} = stresses in prestressed reinforcement at nominal strength (ksi)
f_{se} = effective stress in prestressed reinforcement after all losses (ksi)
d_b = nominal diameter of wire or strand (in.)

In pretensioned members, high-tensile stresses exist at the end zones, for which special reinforcement must be provided. Such reinforcement in the form of vertical stirrups is uniformly distributed within a distance $h/5$ measured from the end of the beam. The first stirrup is usually placed at 1 to 3 in. from the beam end or as close as possible. It is a common practice to add nominal reinforcement for a distance d measured from the end of the beam. The area of the vertical stirrups, A_v, to be used at the end zone can be calculated approximately from the following expression:

$$A_v = 0.021 \frac{F_i h}{f_{sa} l_t} \tag{19.66}$$

where f_{ss} = allowable stress in the stirrups (usually 20 ksi) and l_t = 50 tendon diameters.

Example 19.8

Determine the necessary stirrup reinforcement required at the end zone of the beam given in Example 19.4.

Solution

$$F_i = 365.9 \text{ K} \quad h = 40 \text{ in.} \quad f_s = 20 \text{ ksi} \quad l_t = 50 \times \frac{7}{16} = 22 \text{ in.}$$

Therefore,

$$A_v = 0.021 \times \frac{365.9 \times 40}{20 \times 22} = 0.7 \text{ in.}^2$$

$$\frac{h}{2} = \frac{40}{5} = 8 \text{ in.}$$

Use four no. 3 closed stirrups within the first 8-in. distance from the support. A_v (provided) = $4 \times 0.22 = 0.88 \text{ in.}^2$.

19.10.2 Posttensioned Members

In posttensioned concrete members, the prestressing force is transferred from the tendons to the concrete, for both bonded and unbonded tendons, at the ends of the member by special anchorage devices. Within an anchorage zone at the end of the member, very high compressive stresses and transverse tensile stresses develop, as shown in Fig. 19.10. In practice, it is found that the length of the anchorage zone does not exceed the depth of the end of the member; nevertheless, the state of stress within this zone is extremely complex.

The stress distribution due to one tendon within the anchorage zone is shown in Fig. 19.11. At a distance h from the end section, the stress distribution is assumed uniform all over the section. Considering the lines of force (trajectories) as individual elements acting as curved struts, the trajectories tend to deflect laterally toward the centerline of the beam in zone A, inducing compressive stresses. In zone B, the curvature is reversed in direction and the struts deflect outward, inducing tensile stresses. In zone C, struts are approximately straight, inducing uniform stress distribution.

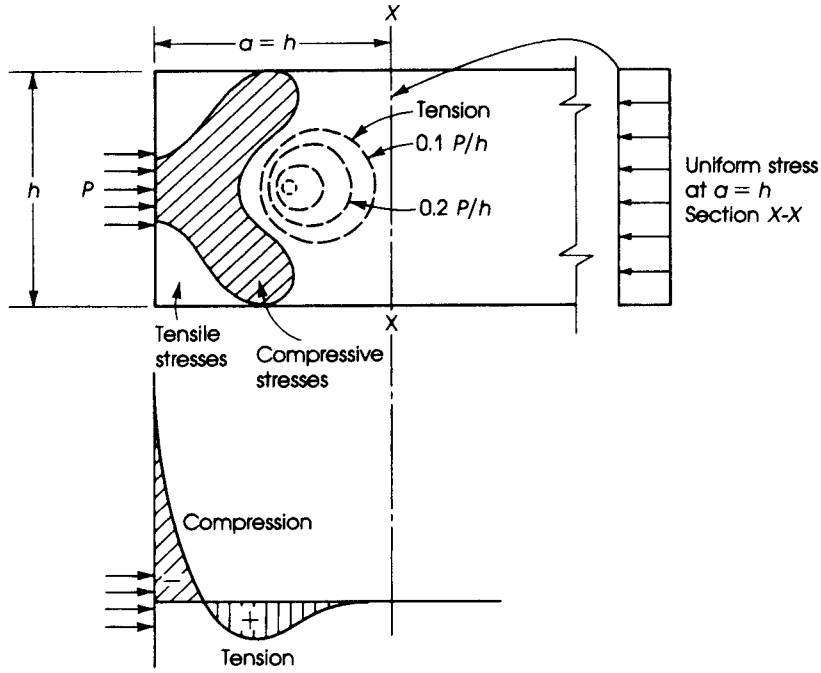


Figure 19.10 Tension and compression zones in a posttensioned member.

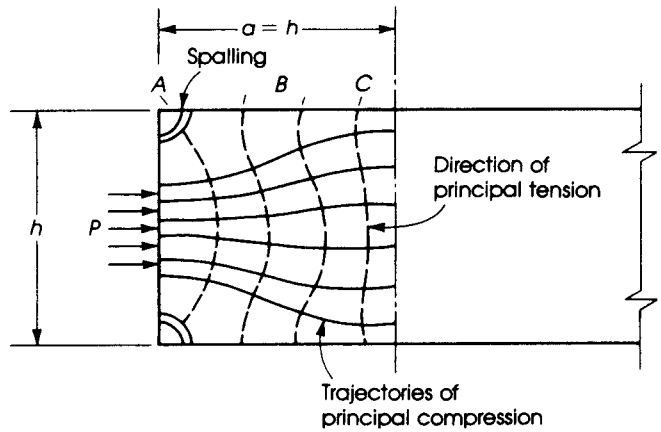


Figure 19.11 Tension and compression trajectories in a posttensioned member.

The reinforcement required for the end anchorage zones of posttensioned members generally consists of a closely spaced grid of vertical and horizontal bars throughout the length of the end block to resist the bursting and tensile stresses. It is a common practice to space the bars not more than 3 in. in each direction and to place the bars not more than 1.5 in. from the inside face of the bearing plate. Approximate design methods are presented in Refs. 24 to 27.

SUMMARY

Section 19.1

The main objective of prestressing is to offset or counteract all or most of the tensile stresses in a structural member produced by external loadings, thus giving some advantages over a reinforced concrete member. A concrete member may be pretensioned or posttensioned. Nonprestressed reinforcement may also be added to the concrete member to increase its ultimate strength.

Section 19.2

1. The allowable stresses in concrete at transfer are

   Maximum compressive stress $= 0.6f'_{ci}$

   Maximum compressive stress at end of simply supported $= 0.7f'_{ci}$

   Maximum tensile stress $= 3\sqrt{f'_{ci}}$

   Maximum tensile stress at end of simply supported $= 6\sqrt{f'_{ci}}$

   The allowable stresses after all losses are $0.45f'_c$ for compression and $6f'_c$ for tension.
2. The allowable stress in a pretensioned tendon at transfer is the smaller of $0.74f_{pu}$ or $0.82f_{py}$. The maximum stress due to the tendon jacking force must not exceed $0.85f_{pu}$ or $0.94f_{py}$; and the maximum stress in a posttensioned tendon after the tendon is anchored is $0.70f_{pu}$.

Section 19.3

The sources of prestress loss are the elastic shortening, shrinkage, and creep of concrete; relaxation of steel tendons; and friction. An approximate lump sum loss is 35 ksi for pretensioned members and 25 ksi for posttensioned members (friction is not included).

$$\text{Loss due to elastic shortening} = \frac{nF_i}{A_c} \tag{19.1}$$

$$\text{Loss due to shrinkage} = \varepsilon_{sh}E_s \tag{19.6}$$

$$\text{Loss due to creep} = C_c(\varepsilon_c E_s) \tag{19.7}$$

Loss due to relaxation of steel varies between 2.5% and 12%. Loss due to friction in posttensioned members stems from the curvature and wobbling of the tendon.

$$P_{px} = P_{pj}e^{-(kl_{px}+\mu_p\alpha_{px})} \tag{19.10}$$

$$P_{px} = P_{pj}(1 + Kl_{px} + \mu_p\alpha_{px})^{-1} \tag{19.11}$$

Section 19.4

Elastic stresses in a flexural member due to loaded and unloaded conditions are given by Eqs. 19.13 through 19.16. The limiting values of the eccentricity, e, are given by Eqs. 19.20, 19.22, 19.24, and 19.26. The minimum and maximum values of F_i are given by Eqs. 19.31 and 19.32, respectively.

Section 19.5

The nominal moment of a rectangular prestressed concrete member is

$$M_n = T\left(d - \frac{a}{2}\right) = A_{ps} f_{ps} d\left(1 - \frac{\rho_p f_{ps}}{1.7 f'_c}\right) \tag{19.37}$$

The values of f_{ps} are given by Eqs. 19.39 and 19.40. For flanged sections,

$$M_n = A_{pv} f_{ps}\left(d - \frac{a}{2}\right) + A_{pf} f_{ps}\left(d - \frac{h_f}{2}\right) \tag{19.43}$$

If nonprestressed reinforcement is used in the flexural member, then

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d - \frac{a}{2}\right) \tag{19.44}$$

where $a = (A_{ps} f_{ps} + A_s f_y)/0.85 f'_c b$. For M_n of flanged and rectangular sections with compression reinforcement, refer to Eqs. 19.46 and 19.47, respectively.

Sections 19.6–19.7

1. The cracking moment is

$$M_{cr} = \frac{I}{y_b}\left[7.5\lambda\sqrt{f'_c} + \frac{F}{A} + \frac{(Fe)y_b}{I}\right] \tag{19.48}$$

2. Midspan deflections of simply supported beams are summarized in Table 19.3.

Section 19.8

$$\text{Shear strength of concrete } (V_c) = \left(0.6\lambda\sqrt{f'_c} + 700\frac{V_u d}{M_u}\right) b_w d \tag{19.50}$$

$$\text{Minimum } V_c = 2\lambda\sqrt{f'_c} b_w d$$

$$\text{Maximum } V_c = 5\lambda\sqrt{f'_c} b_w d$$

The shear strength, V_{ci}, based on flexural shear, is given by Eq. 19.47, and the web-shear strength, V_{cw}, is given by Eq. 19.53:

$$V_s = \frac{1}{\phi}(V_u - \phi V_c) \quad \text{and} \quad A_v = \frac{A_{ps}}{80} \times \frac{f_{pu}}{f_y} \times \frac{S}{d} \times \sqrt{\frac{d}{b_w}} \tag{19.59}$$

Section 19.9

Empirical practical dimensions for the preliminary design of prestressed concrete members are suggested in this section.

Section 19.10

The development length of three- to seven-wire strands is

$$l_d = \left(f_{ps} - \frac{2}{3} f_{se}\right) d_b \tag{19.65}$$

The area of stirrups in an end block is

$$A_v = 0.021 \frac{F_i h}{f_{se}} l_t \tag{19.66}$$

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PROBLEMS

19.1 A 60-ft-span simply supported pretensioned beam has the section shown in Fig. 19.12. The beam is prestressed by a force $F_i = 395$ K at transfer (after the elastic loss). The prestress force after all losses is $F = 320$, f'_{ci}(compressive stress at transfer) $= 4$ ksi and f'_c(compressive stress after all losses) $=$ 6 ksi. For the midspan section and using the ACI Code allowable stresses, (a) calculate the extreme fiber stresses due to the prestressing force plus dead load and (b) calculate the allowable uniform live load on the beam.

19.2 For the beam of Problem 19.1 (Fig. 19.12), calculate the elastic loss and all time-dependent losses using the following data: $F_i = 405$ K, $A_{ps} = 2.39\,\text{in.}^2$ located at 6.5 in. from the base, $f'_{ci} = 4$ ksi, and $f'_c = 6$ ksi. $E_c = 57{,}000\sqrt{f'_c}$, and $E_s = 28{,}000$ ksi. The profile of the tendon is parabolic, and the eccentricity at the supports is 0.

19.3 The cross-section of a 56-ft-span simply supported posttensioned girder that is prestressed by 30 cables $\frac{7}{16}$in. in diameter (area of one cable is 0.1089) is shown in Fig. 19.13. The cables are made of seven-wire stress-relieved strands. The profile of the cables is parabolic with the centroid of the prestressing steel (C.G.S.) located at 9.6 in. from the base at the midspan section and located at the centroid of the concrete section ($e = 0$) at the ends. Calculate the elastic loss of prestress and all other losses. Given: $f'_{ci} = 4$ ksi, $f'_c = 6$ ksi, $E_c = 57{,}000\sqrt{f'_c}$, $E_s = 28{,}000$ ksi, $f_{pu} =$ 250 ksi, $F_o = 175$ ksi, D. L. $= 1.0$ K/ft (excluding self-weight), and L. L. $= 1.6$ K/ft.

19.4 For the girder of Problem 19.3,

a. Determine the location of the upper and lower limits of the tendon profile for the section at midspan and for at least two other sections between midspan and support. (Choose sections at 12 ft, 18 ft, and 25 ft from support.)

b. Check if the parabolic profile satisfies these limits.

19.5 For the girder of Problem 19.3, check the limiting values of the prestressing force at transfer F_i.

19.6 A 64-span simply supported pretensioned girder has the section shown in Fig. 19.14. The loads on the girder consist of a dead load $= 1.2$ K/ft (excluding its own weight) that will be applied at a

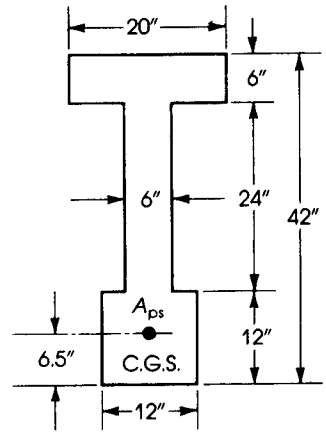


Figure 19.12 Problem 19.1.

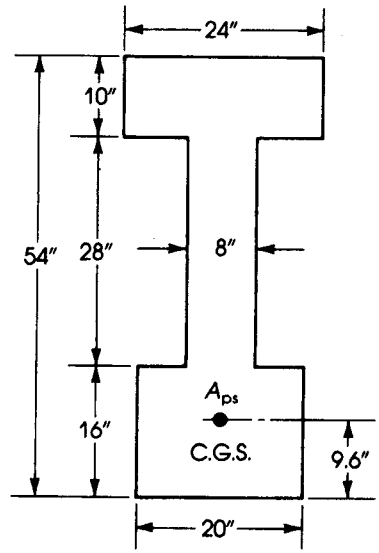


Figure 19.13 Problem 19.3.

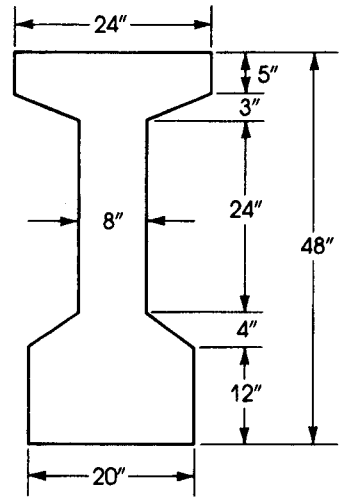


Figure 19.14 Problem 19.6.

later stage and a live load of 0.6 K/ft. The prestressing steel consists of 24 cables $\frac{1}{2}$ in. in diameter (area of one cable = 0.114 in.^{2}), with $E_s = 28{,}000$ ksi, $F_o = 175$ ksi, and $f_{pu} = 250$ ksi. The strands are made of seven-wire stress-relieved steel. The concrete compressive strength at transfer is $f_{ci} = 4$ ksi, and at 28 days, $f'_c = 5$ ksi. The modulus of elasticity is $E_c = 57{,}000\sqrt{f'_c}$. For the beam just described,

a. Determine the upper and lower limits of the tendon profile for the section at midspan and three other sections between the midspan section and the support. (Choose sections at 3 ft, 11 ft, and 22 ft from the support.)

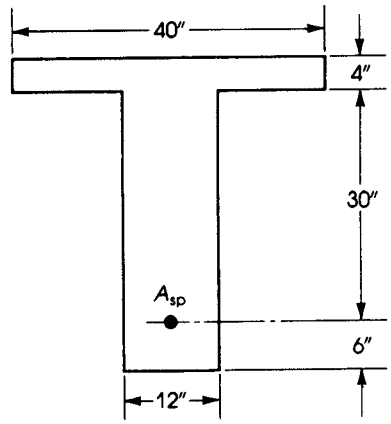


Figure 19.15 Problem 19.10.

b. Locate the tendons to satisfy these limits using straight horizontal tendons within the middle third of the span.

c. Check the limiting values of the prestressing force at transfer.

19.7 For the girder of Problem 19.6,

a. Harp some of the tendons at one-third points, and draw sections at midspan and at the end of the beam showing the distribution of tendons.

b. Revise the prestress losses, taking into consideration the variation of the eccentricity, e, along the beam.

c. Check the factored moment capacity of the section at midspan.

d. Determine the cracking moment.

19.8 For the girder of Problem 19.6,

a. Calculate the camber at transfer.

b. Calculate the immediate deflection at service load.

19.9 For the girder of Problem 19.6, determine the shear capacity of the section and calculate the necessary web reinforcement.

19.10 Determine the nominal moment capacity, M_n, of a pretensioned concrete beam that has the cross-section shown in Fig. 19.15. Given: $f'_c = 5$ ksi, $f_{pu} = 270$ psi, $f_{se} = 160$ ksi, and $A_{se} = 2.88\,\text{in}^2$.

CHAPTER 20

SEISMIC DESIGN OF REINFORCED CONCRETE STRUCTURES



Collapse of frame concrete structures due to an earthquake.

20.1 INTRODUCTION

Ground motions during an earthquake can severely damage the structure. The ground acceleration when transmitted through the structure is amplified, and it is called the response acceleration. The amplified motion can produce forces and displacements that can be larger than the motions the structure can sustain.

Many factors influence the intensity of shaking of the structure such as earthquake magnitude, distance from fault or epicenter, duration of strong shaking, soil conditions of the site, and frequency content of the motion.

A structure should be designed, depending on the type of structure and its function, to have acceptable levels of response generated in an earthquake. Economy of design is achieved by allowing the structure to deform above elastic limit.

20.2 SEISMIC DESIGN CATEGORY

Building Code Requirements for Structural Concrete (ACI 318-08) gives the procedure for design and detailing of structures subjected to earthquake loads but does not address the calculations of seismic forces. In this chapter the International Building Code (IBC 2006) will be utilized for the calculation of seismic forces.

The IBC 2006 section 1613.5.6 defines six seismic design categories (SDC): A, B, C, D, E, and F. It also defines four occupancy categories: I, II, III, and IV. To relate the SDC and the occupancy category, the design spectral response accelerated coefficients S_{DS} and S_{DI} are used. S_{DS} is the design spectral response acceleration coefficient for short periods and S_{DI} is the design response acceleration coefficient for 1-second period. Design spectral response acceleration coefficients are related to severity of the design earthquake ground motions at the site.

A seismic design category will determine which type of lateral force analysis must be performed and which type of lateral-force resisting system must be used.

20.2.1 Determination of Occupancy Category

Buildings shall be assigned an occupancy category according to Table 20.1 as described in IBC 2006 Section 1604.5. The first step is to define the nature of occupancy of the structure according to the occupancy category. The seismic factor, I_E, also called seismic occupancy factor, is also listed in Table 20.1 and will be utilized in a later section.

20.2.2 Determination of Design Spectral Response Acceleration Coefficients

Earthquake ground motion is usually recorded as an acceleration of the ground at a particular location. The acceleration of the ground generates the acceleration of the structure (response acceleration), which produces earthquake forces that act on the structure. Earthquake forces generate deformations, internal forces, and stresses in the structure. If the structure is not properly designed to sustained deformations and forces it will have great damage and may even collapse.

Therefore, the first step to design an earthquake-resistant structure is to determine the maximum possible response accelerations that can occur during the earthquake. It is also important to know that response of the given structure depends on period of vibration and damping characteristics of the structure.

The IBC 2006 Section 1613.5.4 gives a procedure to determine the design response spectrum curve, from which the design response accelerations, S_a, for any given period of vibration T are calculated. One part of this procedure is the determination of the spectral response acceleration coefficients for short periods, S_{DS}, and for a 1-second period, S_{D1}.

To calculate the design acceleration values for short periods, S_{DS} and 1-second periods, S_{D1}, the following equation can be utilized:

$$S_{DS} = \frac{2}{3} S_{MS} \tag{20.1a}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{20.1b}$$

where

S_{MS} = mapped maximum considered earthquake spectral response accelerations for short periods adjusted for soil type

S_{M1} = mapped maximum considered earthquake spectral response accelerations for 1-second period adjusted for soil type

S_{MS} and S_{M1} can be determined from

$$S_{MS} = F_a S_S \tag{20.2a}$$

$$S_{M1} = F_v S_1 \tag{20.2b}$$

where

S_S = mapped maximum considered earthquake spectral response accelerations at short periods determined from Fig. 20.1*a*

S_1 = Mapped maximum considered earthquake spectral response accelerations at 1-second period determined from Fig. 20.1*b*

F_a, F_v = Site coefficients

The values of F_a and F_v are determined from Tables 20.2 and 20.3 and are dependent on the mapped spectral values (S_S and S_1) and the site class as can be determined in Table 20.4.

Table 20.1 Classification of Structures Based on their Nature of Occupancy

Occupancy Category	Nature of Occupancy	Seismic Factor, I_E
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	1.00
II	Buildings and other structures except those listed in Categories I, III, and IV	1.00
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to • Buildings and other structures where more than 300 people congregate in one area • 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 5000 • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Category IV • Buildings and other structures not included in Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released	1.25
IV	Buildings and other structures designed 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 • Designed earthquake, hurricane, or other emergency shelters • Designed 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 Category IV structures • Structures containing highly toxic materials as defined by Section 307 of IBC 2006 where the quantity of the material exceeds the maximum allowable quantities of Table 307.7(2) of IBC 2006 • Aviation control towers, air traffic control centers, and emergency aircraft hangers • Buildings and other structures having critical national defense functions • Water treatment facilities required to maintain water pressure for fire suppression	1.50

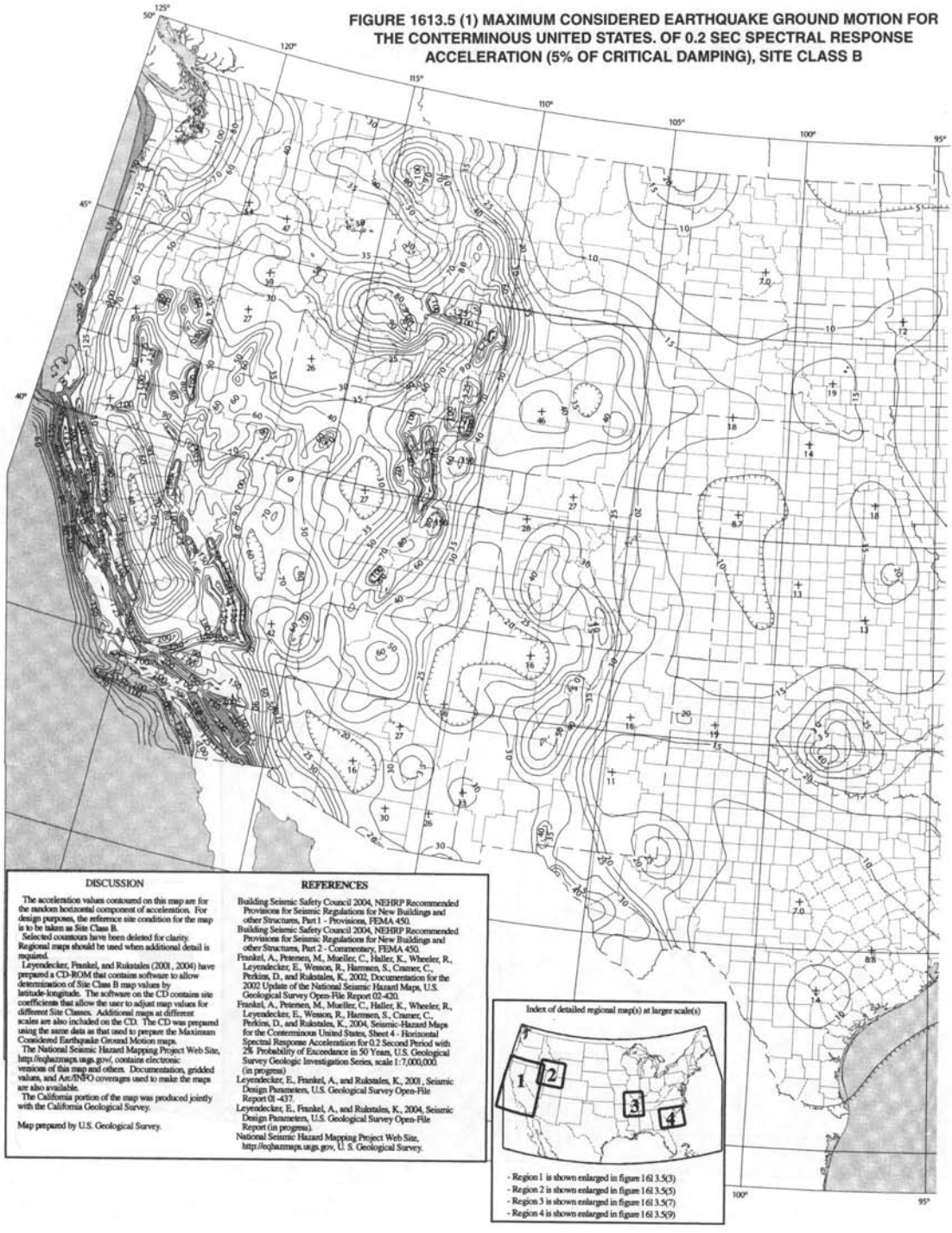


Figure 20.1a Maximum considered earthquake spectral response acceleration at short periods, S_S (IBC Fig. 1613.5 (1)). Courtesy of International Building Code (IBC 2006).

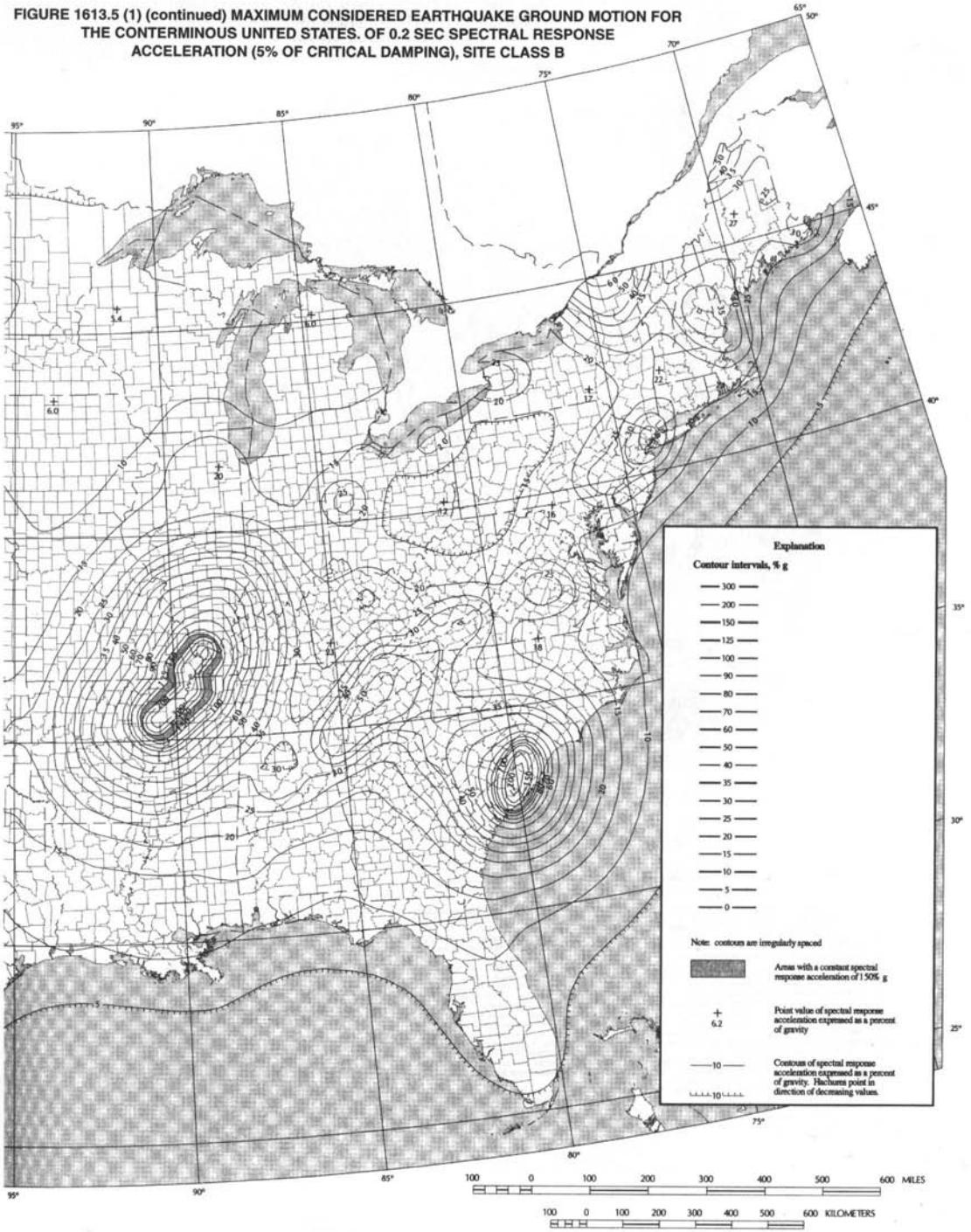


Figure 20.1a (*continued*)

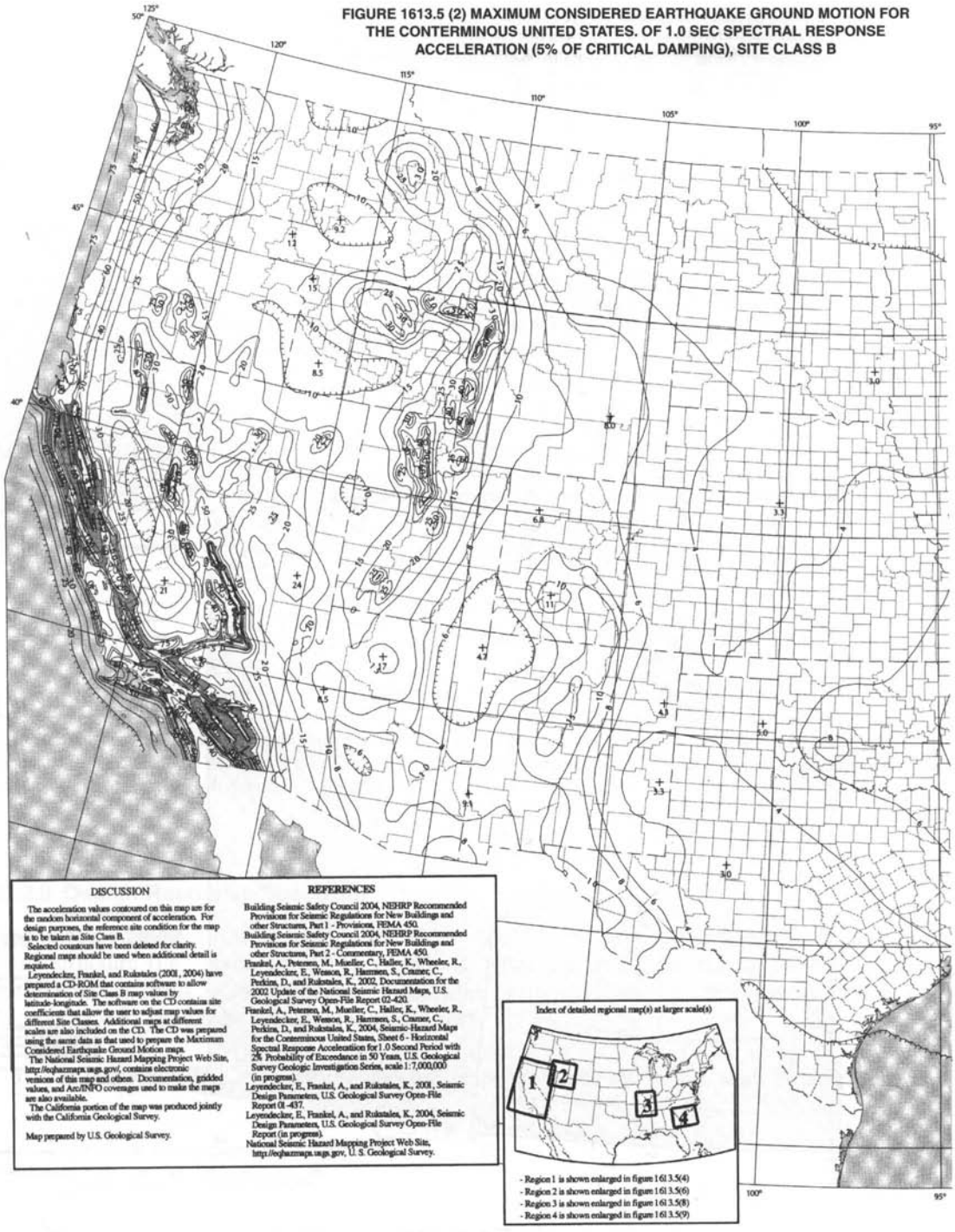


Figure 20.1b Maximum considered earthquake spectral response acceleration at 1-second periods, S_1 (IBC Fig. 1613.5(2)). Courtesy of International Building Code (IBC 2006).

FIGURE 1613.5 (2) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

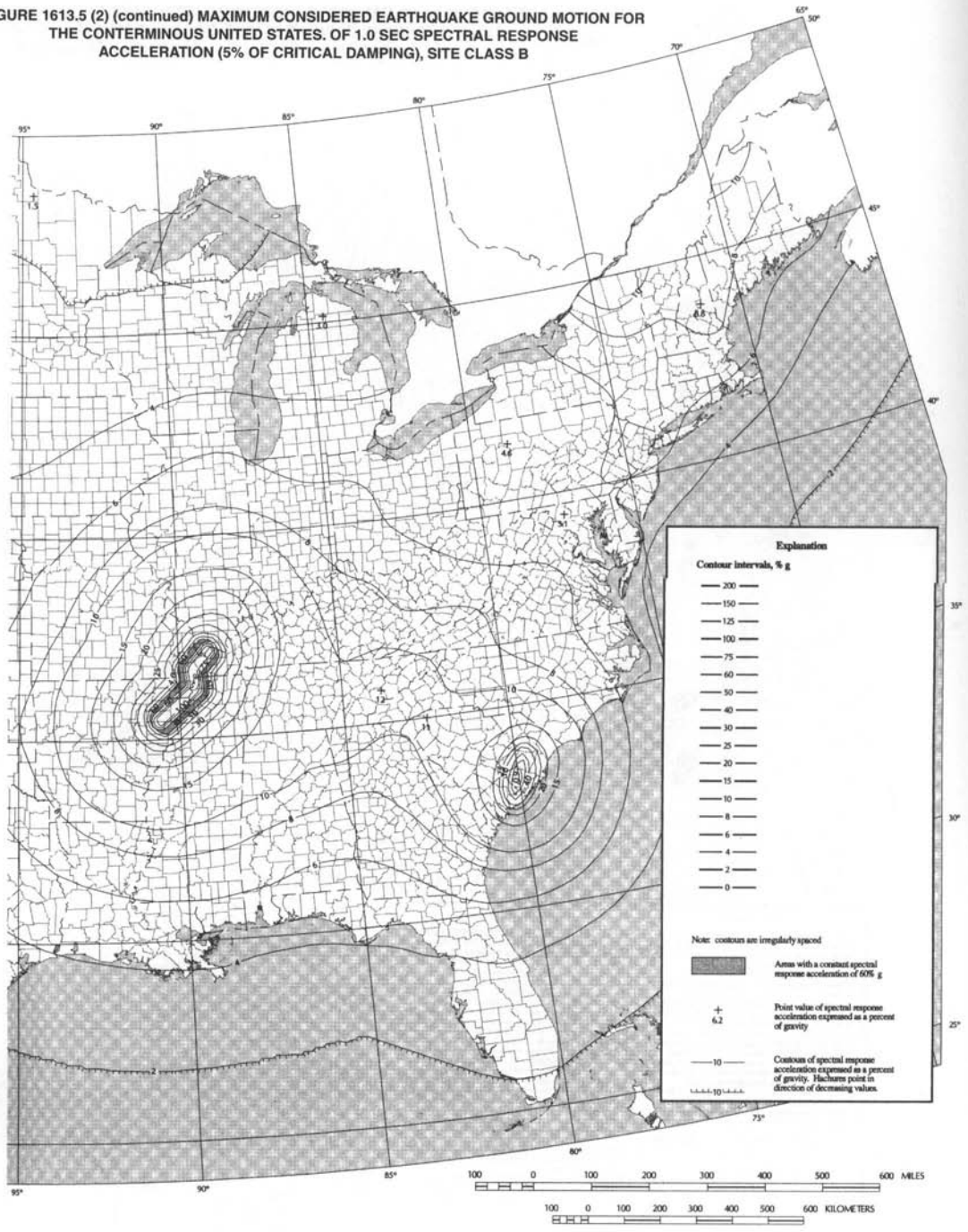


Figure 20.1b (*continued*)

Table 20.2 Values of Site Coefficient, F_a[a] (Table 1613.5.3(1) of IBC 2006)

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 b	Note b	Note b	Note b	Note b

[a]Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
[b]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine appropriate values, or in accordance with Section 11.4.7 of ASCE 7.

Table 20.3 Values of Site Coefficient, F_v[a] (Table 1613.5. 3(2) of IBC 2006)

Site Class	Mapped Spectral Response Acceleration at 1-Second Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
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 b	Note b	Note b	Note b	Note b

[a]Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
[b]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine appropriate values, or in accordance with Section 11.4.7 of ASCE7.

20.2.3 Design Response Spectrum

Design response spectrum is used to determine the design spectral response acceleration for a given structure (i.e., given period of vibration). After calculating design response acceleration coefficients S_{DS} and S_{D1} from Section 20.2.2, the design response spectrum curve (ASCE 7-05, Section 11.4.5) should be constructed as follows:

1. For periods $T \leq T_o$, the design spectral response acceleration, S_a, shall be determined as

$$S_a = 0.6 \frac{S_{DS}}{T_o} T + 0.4 S_{DS} \tag{20.3}$$

where

$$T_o = 0.2 \frac{S_{D1}}{S_{DS}} \tag{20.4}$$

T = Fundamental period of the structure (in seconds) and will be determined later in Section 20.3.1 (Eq. 20.15 or 20.16).

Table 20.4 Site Classification (Table 1613.5.2 of IBC 2006)

Site Class	Soil Profile Name	Average Properties in Top 100, as per Section 1613.5.5		
		Soil Shear Wave Velocity, $\overline{V}_S$, (ft/s)	Standard Penetration Resistance, $\overline{N}$	Soil Undrained Shear Strength, $\overline{S}_u$
A	Hard rock	$\overline{V}_S > 5{,}000$	N/A	N/A
B	Rock	$2500 < \overline{V}_S \leq 5000$	N/A	N/A
C	Very dense soil and soft rock	$1200 < \overline{V}_S \leq 2500$	$\overline{N}$)50	$S_u \geq 2000$
D	Stiff soil profile	$600 < \overline{V}_S \leq 1200$	$15 \leq \overline{N} \leq 50$	$1000 \leq \overline{S}_u \leq 2000$
E	Stiff soil profile	$\overline{V}_S \leq 600$	$\overline{N} < 15$	$\overline{S}_u < 1000$
E	—	Any profile with more than 10 ft of soil having the following characteristics: 1. Plasticity index (PI) > 20 2. Moisture content (w) ≥ 40% 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)		

2. For periods $T_o \leq T \leq T_s$, the design spectral response acceleration, S_a, shall be determined as

$$S_a = S_{\mathrm{DS}} \tag{20.5}$$

where

$$T_s = \frac{S_{\mathrm{D1}}}{S_{\mathrm{DS}}} \tag{20.6}$$

3. For periods $T_L \geq T > T_s$, the design spectral response acceleration, S_a, shall be determined as

$$S_a = \frac{S_{\mathrm{D1}}}{T} \tag{20.7}$$

4. For periods greater than T_L, S_a shall be taken as

$$S_a = \frac{S_{D1} T_L}{T^2} \tag{20.8}$$

where

T_L = long-period transition period(s) showns in Fig. 20.3a (Conterminous United States), Fig. 20.3b (Region 1), Fig. 20.3c (Alaska), Fig. 20.3d (Hawaii). Fig 20.3e (Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix), and Fig. 20.3f (Guam and Tutuila).

The shape of the design response spectrum curve is shown in Fig. 20.2.

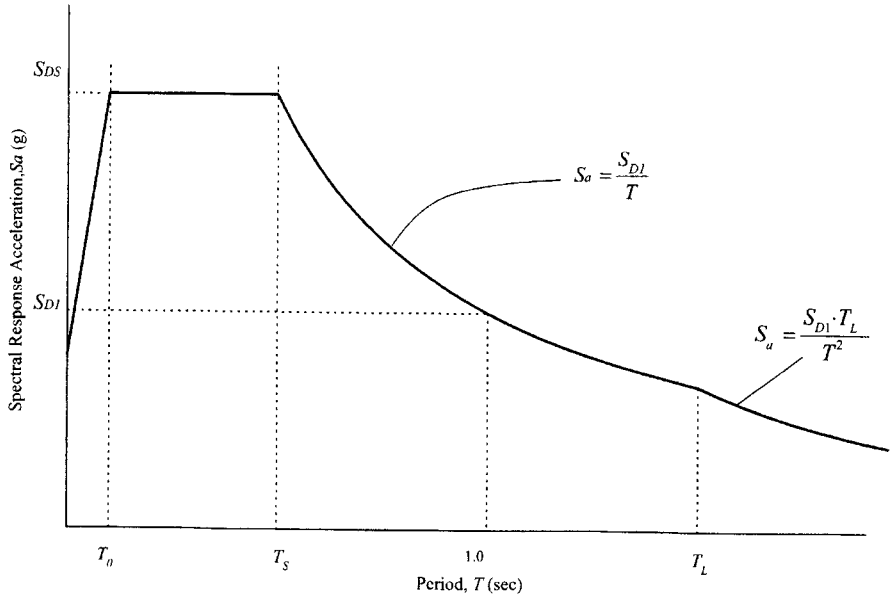


Figure 20.2 Design response spectrum (courtesy of ASCE 7.05, Section 11.4.5).

20.2.4 Determination of Seismic Design Category (SDC)

Structures shall be assigned SDCs, which are classified as A, B, C, D, E, and F are determined from Table 20.5 and Table 20.6. These have no relation to the site class types that, are also named A, B, C, D, E, and F as described in Table 20.4. To determine the SDC, the values of S_{DS} and S_{D1} are utilized and the occupancy category must be defined.

"Occupancy Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period. S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Occupancy Category IV structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a Seismic Design Category based on their Occupancy Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in Section 20.2.2. Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 20.5 or 20.6, irrespective of the fundamental period of vibration of the structure, T.

Where S_1 is less than 0.75, the Seismic Design Category is permitted to be determined from Table 20.5 alone where all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, determined in accordance with Section 20.3.1 is less than $0.8T_s$, where T_s is determined in accordance with Section 20.2.3.
2. In each of two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. The seismic response coefficient C_s is determined from $C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)}$
4. The diaphragms are rigid or for diaphragms that are flexible, the distance between vertical elements of the seismic force-resisting system does not exceed 40 ft.

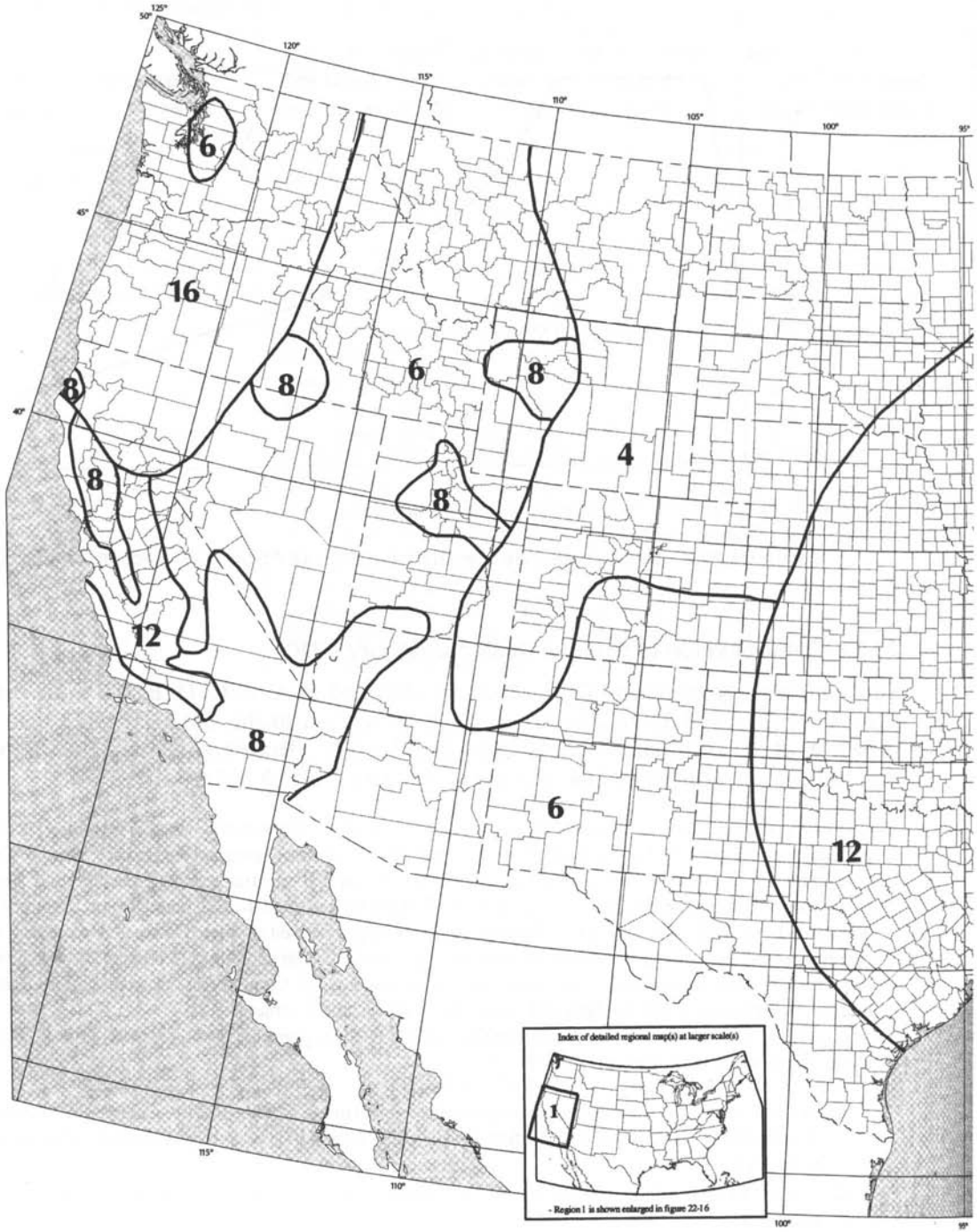


Figure 20.3a Long-period transition period, T_L(SEC), for the conterminous United States (courtesy of ASCE 7-05).

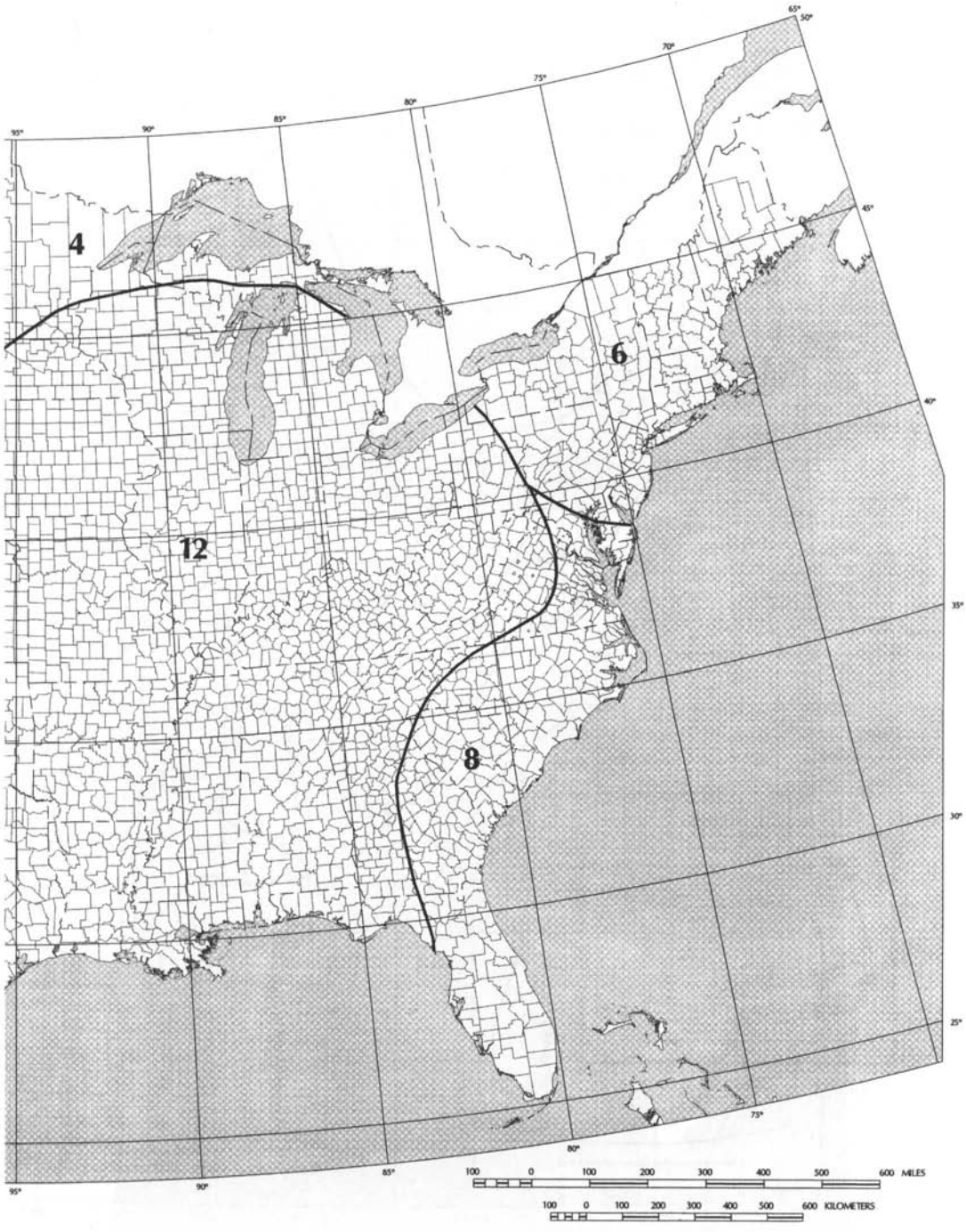


Figure 20.3b (*Continued*)

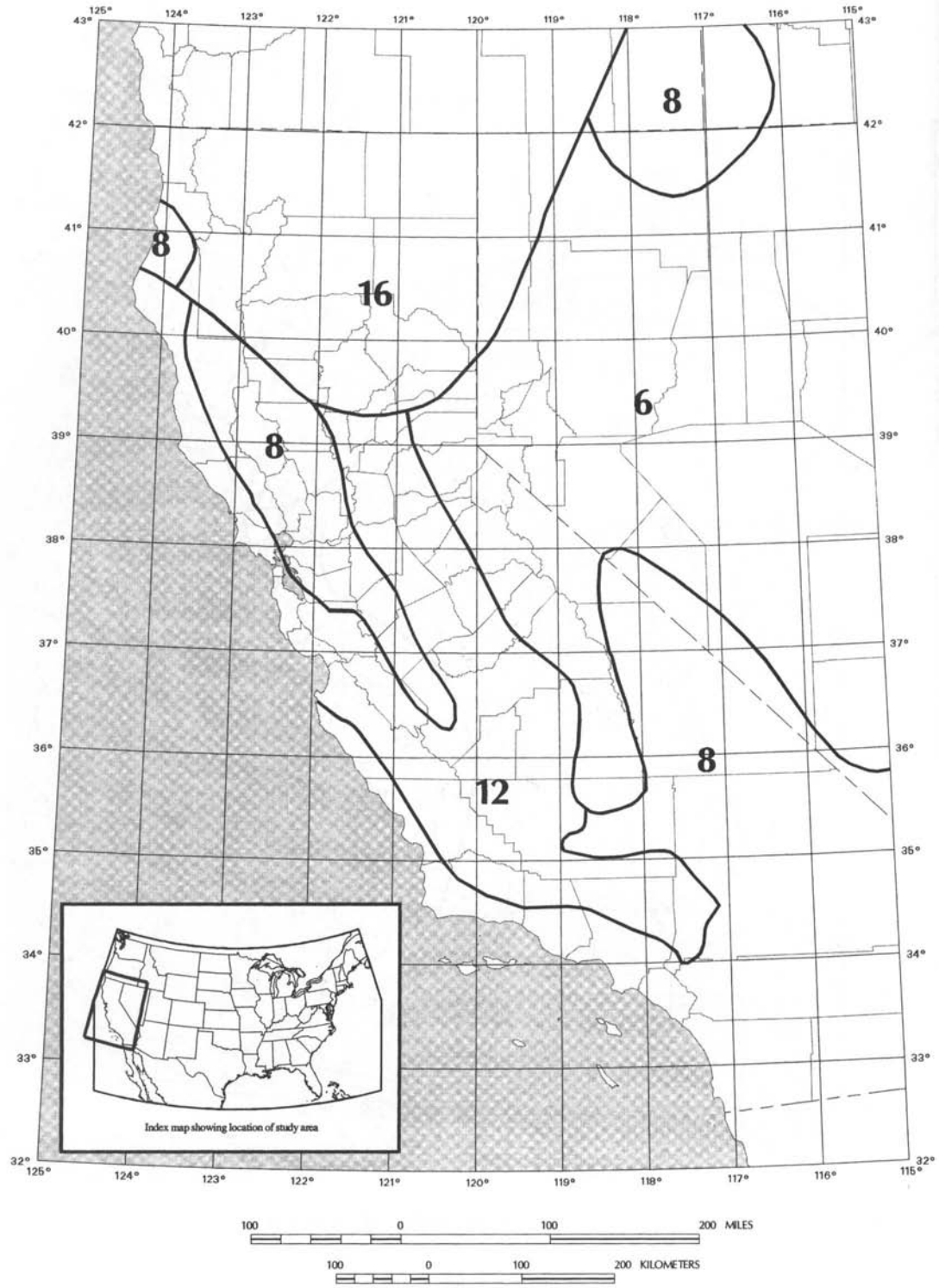


Figure 20.3c Long-period transition period, T_L(SEC), for region 1 (courtesy of ASCE 7-05).

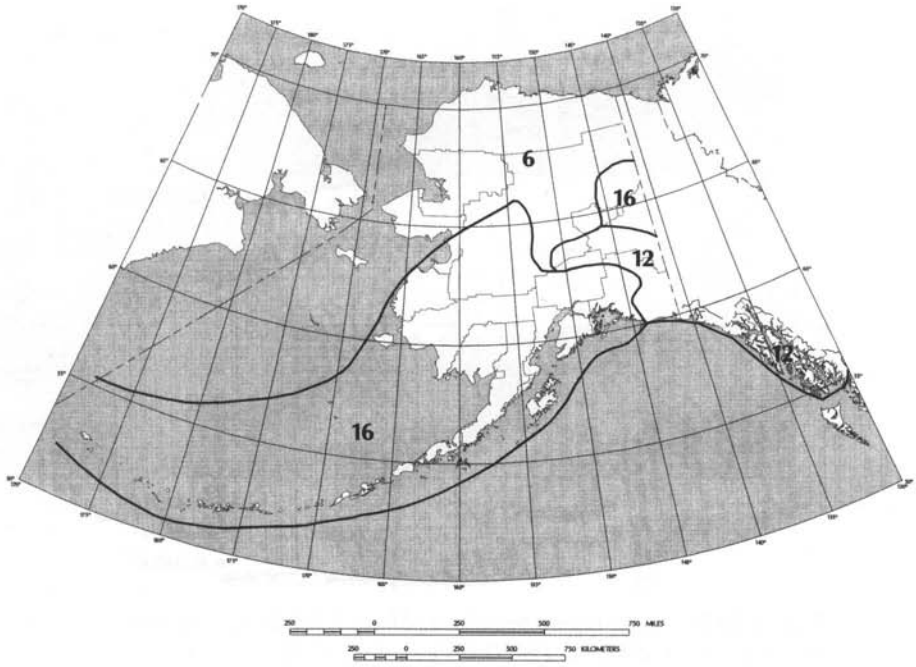


Figure 20.3d Long-period transition period, T_L(SEC), for Alaska (courtesy of ASCE 7-05).

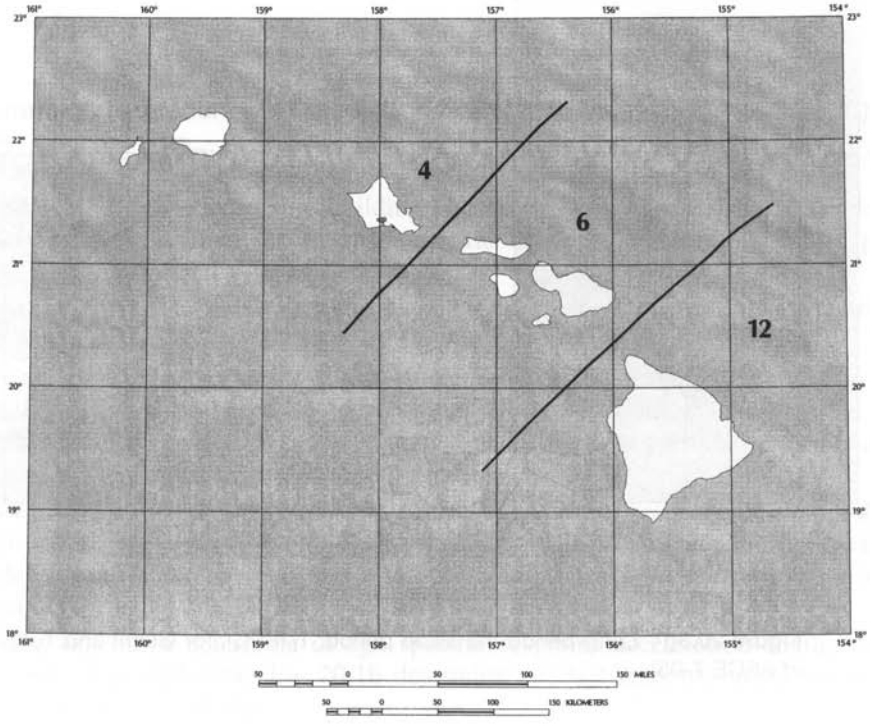


Figure 20.3e Long-period transition period, T_L(SEC), for Hawaii (courtesy of ASCE 7-05).

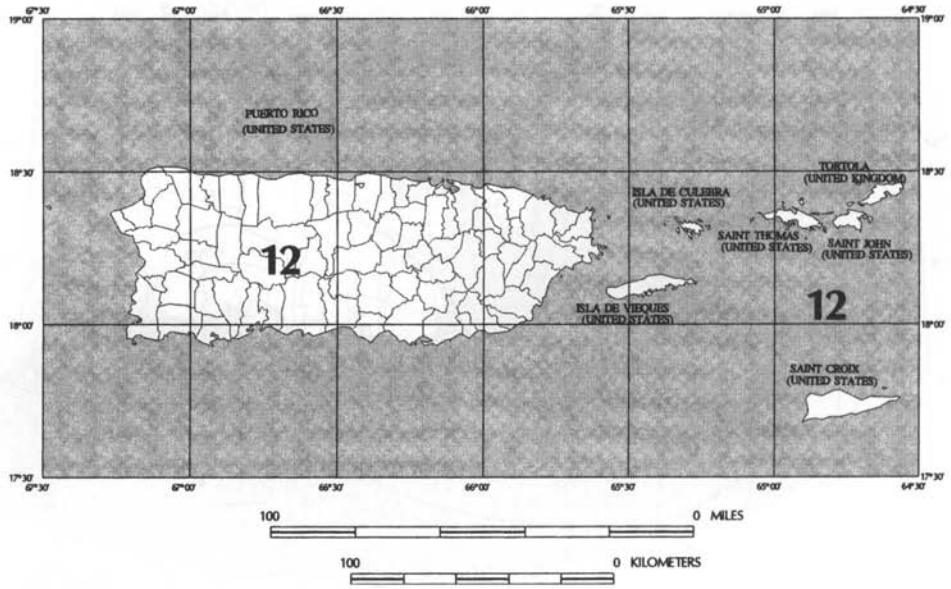


Figure 20.3f Long-period transition period, T_L(SEC), for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix (courtesy of ASCE 7-05).

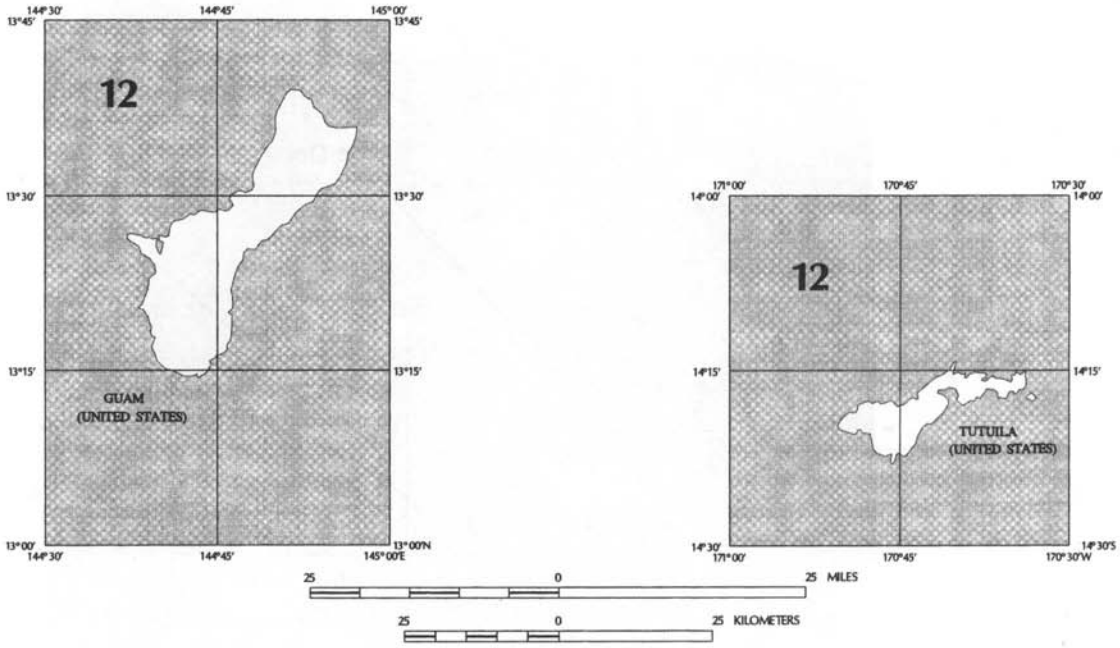


Figure 20.3g Long-period transition period, T_L(SEC), for Guam and Tutuila (courtesy of ASCE 7-05).

Table 20.5 Seismic Design Category Based on Short-Period Response Accelerations (Table 1613.5.6(1) of IBC 2006)

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167\,g$	A	A	A
$0.167\,g \leq S_{DS} < 0.33\,g$	B	B	C
$0.33\,g \leq S_{DS} < 0.50\,g$	C	C	D
$0.50\,g \leq S_{DS}$	D	D	D

Table 20.6 Seismic Design Category Based on 1-Second Period Response Acceleration (Table 1613.5.6(2) of IBC 2006)

Value of S_{D1}	Occupancy Category		
	I or II	III	IV
$S_{D1} < 0.067\,g$	A	A	A
$0.067\,g \leq S_{D1} < 0.133\,g$	B	B	C
$0.133\,g \leq S_{D1} < 0.20\,g$	C	C	D
$0.20\,g \leq S_{D1}$	D	D	D

Where the alternate simplified design procedure of Section 20.3.3 is used, the Seismic Design Category is permitted to be determined from Table 20.5 alone, using the value of S_{DS} determined in Section 20.2.2." (Source: ASCE 7-05, Section 11.6)

20.2.5 Summary: Procedure for Calculation of Seismic Design Category (SDC)

Step 1. Determine seismic use group as described in Section 20.2.1. (Table 20.1)

Step 2. Based on the location of the building determine the mapped spectral accelerations for short periods, S_s, and the mapped spectral accelerations for a 1-second period. Use Fig. 20.1*a* and Fig. 20.1*b* of Section 20.2.2

Step 3. Use Table 20.4 to determine site class based on the soil profile name and properties of soil.

Step 4. Using Table 20.2 determine site coefficient F_a based on mapped maximum considered earthquake spectral response accelerations at short periods, S_S. Also using Table 20.3 determine site coefficient F_v based on mapped maximum considered earthquake spectral response accelerations at 1-second period, S_1.

Step 5. Calculate the maximum considered earthquake spectral response accelerations for short periods for specific soil class, S_{MS}, using Eq. 20.2a. Also calculate the maximum considered earthquake spectral response accelerations for 1-second period for specific soil class, S_{M1}, using Eq. 20.2b.

Step 6. Using Eq. 20.1a determine design spectral response acceleration coefficient for short periods, S_{DS}, and using Eq. 20.1b determine spectral response acceleration coefficient for 1-second period, S_{D1}.

Step 7. Determine SDC according to Section 20.2.4. Utilize Table 20.5 and Table 20.6.

Example 20.1

Determine seismic design category for a minor storage facility building in San Francisco on soft rock.

Solution

1. According to Table 20.1, minor storage facilities buildings are classified in occupancy category I.
2. $S_S = 2.02\,\text{g}$ (Fig. 20.1*a*)
   $S_1 = 0.60\,\text{g}$ (Fig. 20.1*b*)
3. According to the Table 20.4, a site with soft rock is considered to be class C.
4. According to the Table 20.2 for the site class C and $S_S = 2.02 > 1.25$, $F_a = 1.0$.
   According to the Table 20.3 for the site class C and $S_1 = 0.60 > 0.5$, $F_v = 1.3$.
5. $$S_{MS} = F_a S_S = (1.0)(2.02) = 2.02\ \text{g} \qquad \text{(Eq. 20.2}a\text{)}$$
   $$S_{M1} = F_v S_1 = (1.3)(0.60) = 0.78\ \text{g} \qquad \text{(Eq. 20.2}b\text{)}$$
6. $$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(2.02) = 1.35\ \text{g} \qquad \text{(Eq. 20.1}a\text{)}$$
   $$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.78) = 0.52\ \text{g} \qquad \text{(Eq. 20.1}b\text{)}$$
7. According to Table 20.5 for $S_{DS} = 1.35\,\text{g} > 0.50\,\text{g}$, occupancy category I, and since $S_1 < 0.75\,\text{g}$, therefore SDC is D.
   According to Table 20.6 for $S_{D1} = 0.52\,\text{g} > 0.20\,\text{g}$, occupancy category I, and since $S_1 < 0.75\,\text{g}$, therefore SDC is D.

Therefore, seismic design category D is assigned to the structure.

Example 20.2

Determine seismic design category for a hospital building in Oakland, California, on soft soil.

Solution

1. According to Table 20.1, hospital buildings are classified in the occupancy category IV.
2. $$S_S = 2.08\ \text{g} \qquad \text{(Fig. 20.1}a\text{)}$$
   $$S_1 = 0.92\ \text{g} \qquad \text{(Fig. 20.1}b\text{)}$$
3. According to Table 20.4, the site class for soft soil is E.
4. According to the Table 20.2, for the site class E and $S_S = 2.08 > 1.25$, $F_a = 0.9$.
   According to the Table 20.3, for the site class E and $S_1 = 0.92 > 0.5$, $F_v = 2.4$.
5. $$S_{MS} = F_a S_S = (0.9)(2.08) = 1.87\ \text{g} \qquad \text{(Fig. 20.2}a\text{)}$$
   $$S_{M1} = F_v S_1 = (2.4)(0.92) = 2.21\ \text{g} \qquad \text{(Fig. 20.2}b\text{)}$$

6.

$$S_{DS} = \frac{2}{3}S_{MS} = \frac{2}{3}(1.87) = 1.25 \text{ g} \qquad \text{(Fig. 20.1}a\text{)}$$

$$S_{D1} = \frac{2}{3}S_{M1} = \frac{2}{3}(2.21) = 1.47 \text{ g} \qquad \text{(Fig. 20.1}a\text{)}$$

7. According to Table 20.5, for $S_{DS} = 1.25\,\text{g} > 0.50\,\text{g}$, occupancy category IV, and since $S_1 > 0.75\,\text{g}$, SDC is F.

   According to Table 20.6, for $S_{D1} = 1.47\,\text{g} > 0.20\,\text{g}$, occupancy category IV, and since $S_1 > 0.75\,\text{g}$, SDC is F.

Therefore, seismic design category F is assigned to the structure.

20.3 ANALYSIS PROCEDURES

During the earthquake motions, the structure is subjected to the deformation that produces internal forces and stresses. Earthquake engineering philosophy is to relate earthquake dynamic forces to the equivalent static forces, and then using static analysis of the structure, determine deformations, internal forces, and stresses in the structure. IBC describes two analysis procedures to determine the equivalent static forces that will simulate an earthquake action on the structure. These are

1. The equivalent lateral force procedure (used for SDC B, C, D, E, and F)
2. The simplified analysis (used for SDC B, C, D, E, and F, and for constructions limited to two stories in height and three stories in height for light frame constructions)

It should be noted that for the structures in SDC A neither the simplified analysis nor the equivalent lateral force procedure can be utilized. This type of structure should be designed so that the lateral resisting-force system can resist the minimum design lateral force, F_x, applied at each floor level (ASCE 7-05, Section 11.7.2). The design lateral force can be determined for this type of structure using the following equation:

$$F_x = 0.01 w_x \tag{20.9}$$

where

w_x = the portion of the dead load of the structure located or assigned to level x.

20.3.1 Equivalent Lateral Force Procedure

This procedure describes how to calculate the seismic base shear and lateral seismic forces. (ASCE 7-05, Section 12.8)

Seismic Base Shear Calculation. The total seismic force that acts at the base of the structure, called seismic base shear, can be determined according to the following equation:

$$V = C_s W \tag{20.10}$$

where

C_s = seismic response coefficient
W = the effective weight of the structure including the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25% of the reduced floor live load (floor live load in public garages and open parking structures need not be included)
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater (0.48 kN/m^2)
3. Total weight of permanent operating equipment
4. 20 percent of flat roof snow load where flat snow load exceeds 30 psf (1.44 kN/m^2)

Seismic Response Coefficient Calculation. The seismic response coefficient, C_s, shall be determined from:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} \tag{20.11}$$

where

S_{DS} = design spectral response acceleration parameter
R = response modification factor given in Table 20.7
I_E = occupancy importance factor determined from Table 20.1

The value of C_s should not exceed

$$C_{s\,\max} = \frac{S_{D1}}{T\left(\frac{R}{I_E}\right)} \quad \text{for } T \leq T_L \tag{20.12}$$

where

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4
T = the fundamental period of the structure (s) determined in Section 20.3.1 (Eq. 20.15 or Eq. 20.16)
T_L = lone-period transition period (s) determined in Section 20.2.3
S_1 = the mapped maximum considered earthquake spectral response accerelation parameter determined in accordance with Figure 20.1b

Also, C_S should not be less than the following:

1. For buildings and structures in seismic design categories A, B, C, and D and in buildings and structures for which 1-second spectral response acceleration, S_1 is less than 0.6 g, the value of the seismic coefficient, C_s, should not be taken less than

$$C_{s\,\min} = 0.01 \tag{20.13}$$

Table 20.7 Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems (ASCE 7-05, Section 12.2.1)

Basic Seismic-Force— Resisting System	R^a	$\Omega_o{}^b$	$C_d{}^c$
1.Bearing wall systems			
Special reinforced concrete shear walls	5	2.5	5
Ordinary reinforced concrete shear walls	4	2.5	4
Detailed plain concrete shear walls	2	2.5	2
Ordinary plain concrete shear wall	1.5	2.5	1.5
2. Building frame systems			
Special reinforced concrete shear walls	6	2.5	5
Ordinary reinforced concrete shear walls	5	2.5	4.5
Detailed plain concrete shear walls	2	2.5	2
Ordinary plain concrete shear walls	1.5	2.5	1.5
3. Moment-resisting frame systems			
Special reinforced concrete moment frames	8	3	5.5
Intermediate reinforced concrete moment frames	5	3	4.5
Ordinary reinforced concrete moment frames	3	3	2.5
4. Dual systems with special moment frames			
Special reinforced concrete shear walls	7	2.5	5.5
Ordinary reinforced concrete shear walls	6	2.5	5
5. Dual systems with intermediate moment frames			
Special reinforced concrete shear wall	6.5	2.5	5
Ordinary reinforced concrete shear wall	3	3	2.5
6. Shear wall-frame intermediate system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls	4.5	2.5	4
7. Inverted pendulum systems			
Special reinforced concrete moment frames	2.5	1.25	2.5

[a]Response modification coefficient
[b]System overstrength factor
[c]Deflection amplification factor

2. For buildings and structures in seismic design categories E and F and in buildings and structures for which the 1-second spectral response acceleration, S_1, is equal to or greater than 0.6 g, the value of the seismic coefficient, C_s, should not be taken less than

$$C_{s\,\min} = \frac{0.5S_1}{\dfrac{R}{I_E}} \tag{20.14}$$

The response modification factor, R, is a function of several factors. Some of them are ductility capacity and inelastic performance of structural materials and systems during past earthquakes. Values of R for concrete structures are given in Table 20.7 and are selected by defining the type of basic seismic force resisting system. (Table 12.2-1 of ASCE 7-05)

Fundamental period. Elastic fundamental period, T, is a function of the mass and the stiffness of the structure. If the building is not designed, the period T cannot be precisely determined. On the other hand, to design a building, the period of vibration should be known and included in equations for design. That is why building codes provide equations for calculation of approximate

periods of vibrations, T_a. Calculated approximate periods are shorter than the real periods of structure, which leads to the higher base shear and safe design.

An approximate period of vibration, T, can be determined using the following equation:

$$T_a = C_t\, h_n^x \tag{20.15}$$

where h_n is the height in ft above the base to the highest level of the structure and the coefficients C_t and x are determined from Table 20.8.

For the concrete moment-resisting frame buildings that do not exceed 12 stories in height and have a minimum story height of 10 ft, the approximate period of vibration, T, can be determined using the following equation:

$$T_a = 0.1N \tag{20.16}$$

where

N = number of stories in the building

The lateral seismic force calculation. Vertical distribution of the base shear force produces seismic lateral forces, F_x, at any floor level. Seismic lateral forces act at the floor levels because masses of the structure are concentrated at the floor levels. It is known that the force is a product of mass and acceleration. Earthquake motions produce accelerations of the structure and induce forces at the places of mass concentrations (i.e., floor levels).

The lateral force that will be applied to level x of the structure, F_x, can be determined from the following equation:

$$F_x = C_{vx} V \tag{20.17}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{20.18}$$

where

C_{vx} = vertical distribution factor

k = distribution exponent related to the building period

= 1 for building having a period of $T \leq 0.5$ sec

Table 20.8 Values of Approximate Period Parameters C_t and x

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028	0.8
Concrete moment-resisting frames	0.016	0.9
Eccentrically braced steel frames	0.03	0.75
All other structural systems	0.02	0.75

Source: ASCE 7-05, Section 12.8.2.1

$= 2$ for building having a period of $T \geq 2.5$ sec

$= 2$, or linear interpolation between 1 and 2, for building having a period of $0.5 \text{ sec} \leq T \leq 2.5 \text{ sec}$

h_i, h_x = height from the base to level i and x

w_i, w_x = portion of W assigned to level i or x

20.3.2 Summary: Equivalent Lateral Procedure

Step 1. Determine seismic design category according to Section 20.2 and choose an appropriate I_E value from Table 20.1.

Step 2. Choose R value from Table 20.7

Step 3. Determine T using Eq. 20.15 or Eq. 20.16, as applicable.

Step 4. Calculate C_s using Eq. 20.10 and check for $C_{s\,\max}$ (Eq. 20.11) and $C_{s\,\min}$ (Eq. 20.12 or Eq. 20.13, whichever is applicable). Ensure that $C_{s\,\min} \leq C_s \leq C_{s\,\max}$ and

$$\text{if } C_s > C_{s\,\max}, \text{ then choose } C_s = C_{s\,\max}.$$

$$\text{if } C_s < C_{s\,\min}, \text{ then choose } C_s = C_{s\,\min}.$$

Step 5. Calculate total gravity load, W, as described in Section 20.3.1.

Step 6. Calculate seismic base shear using Eq. 20.10.

Step 7. Using Eq. 20.17 calculate seismic lateral load, F_x, for every level of the structure.

20.3.3 The Simplified Analysis

The simplified analysis procedure for seismic design described in Section ASCE 7-05, Section 12.14.8.1 is applicable to any structure that satisfies the following limitations and conditions:

1. Seismic design category B, C, D, E, or F.
2. Light-framed construction not exceeding three stories in height, excluding basement, or any construction.

The seismic base shear and lateral seismic forces are calculated as follows:

"**1.** The seismic base shear, V, in a given direction shall be determined in accordance with

$$V = \frac{FS_{DS}}{R} W \tag{20.19}$$

where

$$S_{\text{DS}} = \frac{2}{3} F_a S_s$$

where F_a is permitted to be taken as 1.0 for rock sites, 1.4 for soil sites, or determined in accordance with Section 20.2.2. For the purpose of this section, sites are permitted to be considered to be rock if there is no more than 10 ft (3 m) of soil between the rock surface and the bottom of spread footing or mat foundation. In calculating S_{DS}, S_s shall be in accordance with Section 20.2.2, but need not be taken larger than 1.5.

$F = 1.0$ for one-story buildings

$F = 1.1$ for two-story buildings

$F = 1.2$ for three-story buildings

R = the response modification factor from Table 20.7 (ASCE 7-05, Table 12.2)

W = effective seismic weight of structure that shall include the total dead and other loads listed in the following text

1. In areas used for storage, a minimum of 25 percent of the floor live load (floor live load in public garages and open parking structures need not be included.)
2. Where provision for partitions is required by Section 4.2.2 (Provision for Partitians) in the floor load design, the actual partition weight, or minimum weight of 10 psf (0.48 kNm^2) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Where the flat roof snow load, P_f, exceeds 30 psf (1.44 kN/m^2), 20 percent of the uniform design snow load, regardless of actual roof slope." (Source: ASCE 7-05, Section 12.14.8.1)

2. The lateral seismic forces calculation. The lateral seismic forces can be determined from (ASCE 7-05, Section 12.14.8.2)

$$F_x = \frac{FS_{DS}}{R} w_x \tag{20.20}$$

where

F_x = the seismic force applied at level x
w_x = the portion of the effective seismic weight of the structure, W, at level x.

20.3.4 Summary: Simplified Analysis Procedure

Step 1. Check whether the structure satisfies the three conditions described in Section 20.3.6 for qualification for the simplified analysis procedure.
Step 2. Determine the value of S_{DS} as described in Section 20.2.2.
Step 3. Choose appropriate R factor from Table 20.7.
Step 4. Determine the total gravity load, W, of the structure as described in Section 20.3.1.
Step 5. Utilize Eq. 20.19 to calculate seismic base shear, V.
Step 6. Determine the seismic lateral forces acting on the structure, F_x, using Eq. 20.20.

20.3.5 Design Story Shear

The seismic lateral forces will produce seismic design story shear, V_x, at any story x that can be determined from the following equation:

$$V_x = \sum_{i=1}^{n} F_i \tag{20.21}$$

where

F_i = the portion of seismic base shear, V, assigned to level i
n = number of stories

The seismic story shear in any story x should be collected and transferred to the story below by vertical elements of lateral-force-resisting system (walls). The distribution of story shear on vertical elements depends on flexibility of the diaphragm, which those elements (walls) support.

There are two types of diaphragm:

1. Flexible diaphragm
2. Rigid diaphragm

Diaphragm is flexible when the lateral deformation of diaphragm is more than two times the average story drift of the story that supports diaphragm. Lateral deformation of diaphragm

is maximum in-plane deflection of the diaphragm under lateral load, and the story drift is the difference between the deflections of the center of mass at the top and the bottom of the story being considered.

A diaphragm that is not flexible by the above definition is rigid.

For flexible diaphragms, the seismic story shear, V_x, is distributed to vertical elements in the story x based on the area of the diaphragm tributary to each line of resistance. The vertical elements of the seismic-force–resisting system may be considered to be in the same line of resistance if the maximum out-of-plane offset between such elements is less than 5% of the building dimension perpendicular to the direction of the lateral force.

For rigid diaphragms, V_x is distributed to the vertical elements in the story x based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

20.3.6 Torsional Effect

For rigid diaphragms the eccentricity between center of mass and center of rigidity can occur. The lateral shear force is applied to the center of mass. Distribution of V_x to the vertical elements can be determined when the shear force acts to the center of rigidity. When the shear force moves from center of mass to the center of rigidity it produces torsional moment. Effect of torsion will increase horizontal forces on vertical elements. Forces are not to be decreased due to torsional effects.

$$T = V_x e \tag{20.22}$$

where

V_x = base shear at level x in any direction

e = eccentricity between center of mass and center of rigidity. It can occur in both directions x and y.

20.3.7 Overturning Moment

The lateral seismic force F_x produces overturning moments. Overturning moment M_x should be calculated using the following equation:

$$M_x = \tau \sum_{i=1}^{n} F_i(h_i - h_x) \tag{20.23}$$

where

F_i = portion of the seismic base shear, V, induced at level i

h_i, h_x = height from the base to level i and x

τ = overturning moment reduction factor

= 1.0 for the top 10 stories

= 0.8 for the twentieth story from the top and below

= linear interpolation between 1.0 and 0.8 for stories between the twentieth and tenth stories below the top

τ is permitted to be taken as 1.0 for the full height of the structure.

20.3.8 Lateral Deformation of the Structure

The seismic lateral forces should be used in calculating deformations of the structure. The value that is of interest for engineers is story drift—the difference between the deflections of the

center of mass at the top and the bottom of the story being considered. The value of story drift under seismic forces is important from different perspectives: stability of the structure, potential damage to nonstructural elements, and human comfort. The allowable values for story drift are shown in Table 20.9 (Table 12.12-1 of ASCE 7–05).

For structures that can be designed based on the simplified analysis procedure described in Section 20.3.3 the drift can be taken as 1% of the story height unless a more exact analysis is provided.

$$\Delta = 0.01h_x \tag{20.24}$$

The value of the design story drift should be less than or equal to the value of allowable story drift, Δ_a, given in Table 20.9.

For all other structures that cannot be analyzed using the simplified analysis procedure, the drift should be determined as follows:

1. Calculate the deflection δ_x at level x from the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E} \tag{20.25}$$

where

δ_{xe} = the elastic lateral deflection at floor level x under seismic lateral forces
C_d = deflection amplification factor from Table 20.7
I_E = occupancy importance factor from Table 20.1

2. The design story drift can then be calculated as the difference between the deflections of the centers of mass of any two adjacent stories. Definition of story drift is shown in Fig. 20.3.

$$\Delta = \delta_x - \delta_{x-1} \tag{20.26}$$

3. Check for the P-delta effect and adjust for magnification factor if needed.

Table 20.9 Allowable Story Drift, Δ_a (in.)[a]

Building	Occupancy Category		
	I or II	III	IV
Buildings, other than masonry shear wall or masonry wall frame buildings four stories or less in height with interior walls, partitions, ceilings, and exterior wall system that have been designed to accommodate the story drift	$0.025h_{sx}$[b]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall buildings[c]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall buildings	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other buildings	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a]There shall be no drift limit for single-story building with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drift.
[b]h_{sx} is the story height below level x.
[c]Building in which the basic structural system consist of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

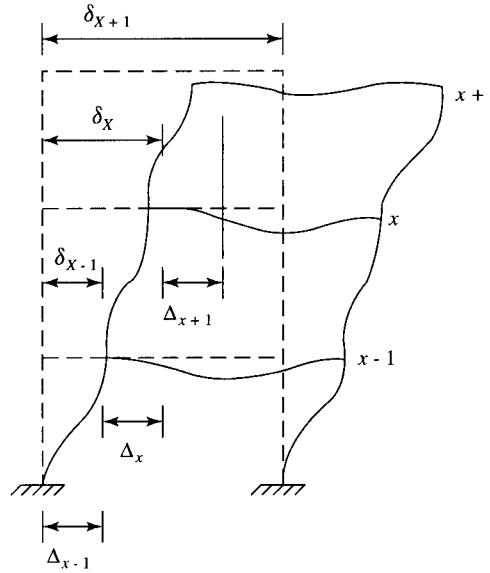


Figure 20.4 Definition of drift.

P-Delta Effect. An accurate estimate of story drift can be obtained by the P-delta analysis. In first order structural analysis the equilibrium equations are formulated for the undeformed shape of structure. When deformations are significant the second-order analysis must be applied and the P-delta effect must be considered in determining the overall stability of the structure. The P-delta effect does not need to be applied when the ratio of secondary to primary moment, θ, does not exceed 0.1. This ratio is given by the following equation:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{20.27}$$

where

θ = stability coefficient
P_x = total unfactored vertical design load at and above level x (dead, floor live and snow load)
Δ = design story drift (in.)
V_x = seismic shear force between level x and x-1
h_{sx} = story height below level x (ft)
C_d = deflection amplification factor in Table 20.7 (Table 12.2-1 of ASCE 7–05)

The stability coefficient, θ, should not exceed

$$\theta_{max} = \frac{0.5}{C_d \beta} \tag{20.28}$$

where

β = ratio of shear demand to shear capacity for the story between level x and x − 1.break A value $\beta = 1$ can be used where the ratio is not calculated.

If $\theta > \theta_{max}$, then the structure is potentially unstable and must be redesigned. For $0.1 < \theta < \theta_{max}$, the interstory drift and element forces need to be computed using the P-delta effect. The design story drift considering P-delta effect, Δ_p, can be calculated from

$$\Delta_p = \Delta \frac{1}{(1-\theta)} \tag{20.29}$$

The computed values of story drift should not exceed the allowable values described in Table 20.9.

20.3.9 Summary: Lateral Deformation of the Structure

Step 1. If the structure satisfies the limitations for the simplified analysis procedure listed in Section 20.3.3, use Eq. 20.24 to determine the story drift.

Step 2. For structures that do not satisfy the limitations for the simplified analysis procedure listed in Section 20.3.3, use Eqs. 20.24, 20.25, 20.26,and 20.27 to calculate δ_x, δ_{x-1}, Δ, θ, and θ_{max}. Check whether the P-delta effect must be considered and adjust Δ to Δ_p using Eq. 20.29.

Step 3. Determine allowable drift from Table 20.9 and compare with the calculated design drift. If calculated drift exceeds the allowable drift, redesign the structure.

Example 20.3: Equivalent Lateral Procedure

Determine the design seismic force and seismic shear for a six-story concrete special moment-resisting frame building located in the area of high seismic risk where $S_s = 1.5$ g and $S_1 = 0.6$ g, on the soil class B. The story heights are all 12 ft, and the story weights are all 1700 k. Check the lateral deformation of the structure. Building elevation is given in Fig. 20.5.

Solution

1.

$I_E = 1.25$ (Table 20.1)

$S_s = 1.5$ g, $S_1 = 0.6$ g (Fig. 20.1*a*, 20.1*b*)

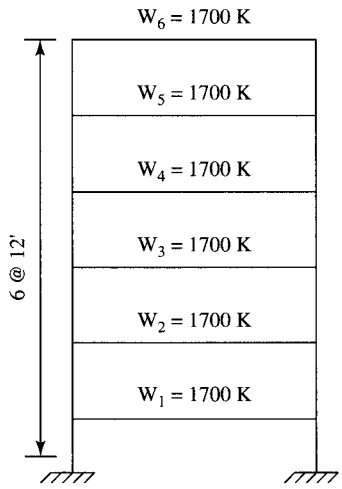


Figure 20.5 Example 20.3 building elevation.

Soil class B:

$$Fa = 1.0,\ Fv = 1.0 \qquad \text{(Table 20.3}a\text{, 20.3}b\text{)}$$

$$S_{MS} = 1.5\text{ g},\ S_{M1} = 0.6\text{ g} \qquad \text{(Eq. 20.2}a\text{, 20.2}b\text{)}$$

$$S_{DS} = 1.0\text{ g} \qquad \text{(Eq. 20.1}a\text{)}$$

$$S_{D1} = 0.4\text{ g} \qquad \text{(Eq. 20.1}b\text{)}$$

SDC is D.

2. According to Table 20.7 for special moment-resisting frame, select $R = 8$.
3. Equation 20.18 is not applicable since $h_x > 10$ ft. Period of vibration of structure is calculated according to Eq. 20.14 as follows:

$$T_a = C_T h^{3/4} = 0.030 \times (6 \times 12)^{3/4} = 0.74\text{ s}$$

4. Calculate seismic response coefficient as follows and check for the limits:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{1}{\left(\frac{8}{1.25}\right)} = 0.156$$

$$C_{s\max} = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.4}{\left(\frac{8}{1.25}\right)0.74} = 0.084$$

Since $S_1 = 0.6$ g, Eq. 20.14 should be used to calculate $C_{s\min}$:

$$C_{s\min} = \frac{0.5S_1}{\frac{R}{I_E}} = \frac{0.5 \times (0.6)}{\frac{8}{1.25}} = 0.047$$

Since $C_s > C_{s\max}$, $C_s = 0.084$.

5. The total gravity load is calculated as follows:

$$W = w_1 + w_2 + w_3 + w_4 + w_5 + w_6 = 6 \times (1700) = 10{,}200\text{ kips}$$

6. Calculate the seismic base using Eq. 20.10:

$$V = C_s W = 0.084 \times 10200 = 857\text{ kips}$$

7. Calculation of F_x, V_x, and M_x (Fig. 20.6). Use Eq. 20.17 to calculate the seismic lateral force, F_x, as shown in the following table. The table also calculates the shear force for each floor level and the overturning moments as described in Eqs. 20.21 and 20.23.

Floor Level	Weight W_i (kips)	Height h_i (ft)	$W_i h_i^{k*}$ (kips-ft)	C_{vx}	Lateral Force, F_x (kips)	Shear Force, V_x (kips)	Overturning Moment M_x (kips-ft)
6	1700	72	204,485	0.30	257	257	0
5	1700	60	166,716	0.24	206	463	3084
4	1700	48	129,849	0.19	163	626	8640
3	1700	36	94,082	0.14	120	746	16,152
2	1700	24	59,743	0.09	77	823	25,104
1	1700	12	27,487	0.04	34	857	34,980
0			682,362			857	45,264

*To calculate k, use Section 20.3.1. For $T = 0.74$ s, using linear interpolation, $k = 1.12$

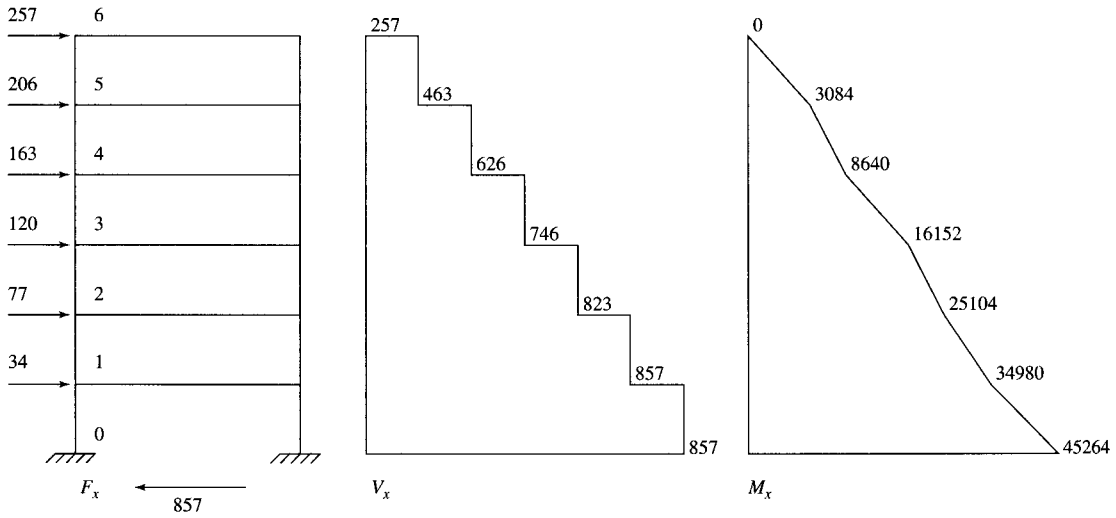


Figure 20.6 Example 20.3: distribution of lateral seismic force, F_x, base shear, V_x, and overturning moment, M_x.

8. Calculation of drift. According to Table 20.7 for special moment resisting frame $C_d = 5.5$,

$$I_E = 1.25 \quad \text{(Table 20.1)}$$

$$h_{sx} = 12 \text{ ft} = 144 \text{ in.}$$

Floor Level	δ_{xe} (in.)	δ_x (in.)	Δ (in.)	P_x (kip)	V_x (kip)	θ
6	1.26	5.54	1.23	1882	257	0.011
5	0.98	4.31	1.19	3943	463	0.013
4	0.71	3.12	1.1	6004	626	0.013
3	0.46	2.02	0.96	8065	746	0.013
2	0.24	1.06	0.8	10,126	823	0.012
1	0.06	0.26	0.26	12,187	857	0.005

$$\theta_{\max} = \frac{0.5}{C_d \beta} = \frac{0.5}{(5.5) \times (1.0)} = 0.09 > \theta \text{ in every floor level}$$

Which is o.k. (Eq. 20.28) Also, $\theta < 0.1$ in every floor level, which means that the P-delta effect can be disregarded.

9. Allowable drift, according to the Table 20.9, is $\Delta_a = 0.010\, h_{sx} = 0.010 \times (12 \times 12) = 1.44$ in $> \Delta$ in every floor level, which is o.k.

Example 20.4: Simplified Analysis

Calculate the seismic base shear for a two-story concrete building assuming that the first floor weight is $w_x = 35$ kip and the second floor weight is 40 kip. The height of the first floor is $h_x = 15$ ft, and of the second floor is 12 ft. Seismic-force– resisting system is ordinary reinforced shear wall system. Utilize the value of S_{DS} from in Example 20.1. Check the lateral deformation of the structure.

Solution

1. The building is classified SDC D (from Example 20.1), and is two stories in height. This building satisfies the conditions for simplified analysis.
2. $S_{DS} = 1.35\,\text{g}$ (Example 20.1).
3. The R factor is chosen from Table 20.7 based on the seismic-force–resisting system of the structure. For ordinary reinforced concrete shear wall, R is equal to 4.
4. Calculate the total gravity load (Fig. 20.7): $W = w_1 + w_2 = 35 + 40 = 75\,\text{kip}$
5. For two story building F = 1.1 as desribed in 20.3.3

$$V = \frac{1.1S_{\text{DS}}}{R}W = \frac{1.1(1.35)}{4} \times 75 = 27.8 \text{ kip} \qquad \text{(Eq. 20.19)}$$

6. Calculate the seismic lateral forces acting at the first and second floors using Eq. 20.20, (Fig. 20.8).

$$F_1 = \frac{1.1S_{\text{DS}}}{R}w_1 = \frac{1.1(1.35)}{4} \times 35 = 13 \text{ kip} \qquad \text{(first floor)}$$

$$F_2 = \frac{1.1S_{\text{DS}}}{R}w_2 = \frac{1.1(1.35)}{4} \times 40 = 14.9 \text{ kip} \qquad \text{(second floor)}$$

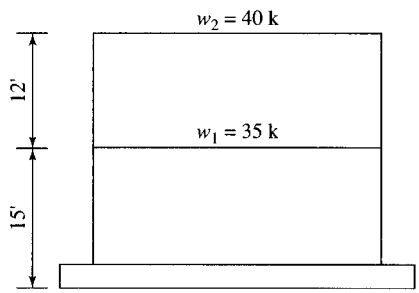


Figure 20.7 Example 20.4 building elevation.

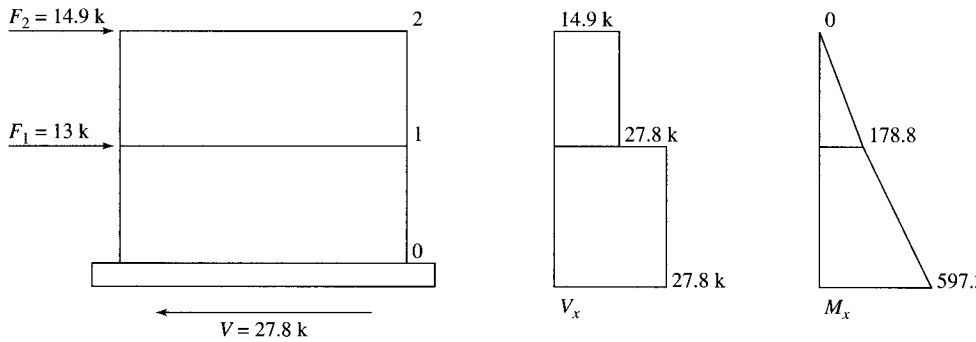


Figure 20.8 Example 20.4: distribution of lateral seismic force, F_x, base shear, V_x, and overturning moment, M_x.

7. Calculate the story shear force using Eq. 20.20:

$$V_2 = 13 \text{ kip} \qquad \text{(second floor)}$$

$$V_1 = 27.8 \text{ kip} \qquad \text{(first floor)}$$

8. Calculate the overturning moment using Eq. 20.23:

$$M_2 = 0 \qquad \text{(second floor)}$$

$$M_1 = 14.9 \times 12 = 178.8 \text{ kip-ft} \qquad \text{(first floor)}$$

$$M_0 = 14.9(12 + 15) + 13 \times 15 = 597.3 \text{ kip-ft} \qquad \text{(at the base of the structure)}$$

9. Determine the seismic lateral story drift using Eq. 20.24:

$$\Delta_1 = 0.01h_1 = 0.01 \times 15 = 0.15 \text{ feet} = 0.0125 \text{ in.} \qquad \text{(first floor)}$$

$$\Delta_2 = 0.01h_2 = 0.01 \times 12 = 0.12 \text{ feet} = 0.01 \text{ in.} \qquad \text{(second floor)}$$

Check for allowable drift using Table 20.9.

$\Delta_a = 0.020h_{sx}$, where h_{sx} is the story height below level x.

$$\Delta_{a_1} = 0.020h_1 = 0.020 \times 15 = 0.3 \text{ ft} = 0.025 \text{ in.} > 0.0125 \text{ in.} \quad \text{(o.k.)} \quad \text{(first floor)}$$

$$\Delta_{a_2} = 0.020h_2 = 0.020 \times 12 = 0.24 \text{ ft} = 0.02 \text{ in.} > 0.01 \text{ in.} \quad \text{(o.k.)} \quad \text{(second floor)}$$

Example 20.5: Torsional Effect

Determine the shear forces V_1 and V_2 acting on the shear wall 1 and 2 of the building with floor plan shown in Fig. 20.9. Assume that the value of story shear, V_x, is 15 kip. Consider torsional effect.

Solution

Center of mass is in the centroid of the rigid diaphragm. The center of rigidity can be determined as follows (Fig. 20.10):

$$x = (25 \times 30 \times 2 + 10 \times 120 \times 2)/(2 \times 25 + 2 \times 10) = 55.7 \text{ ft}$$

$$e_x = 150/2 - 55.7 = 19.3 \text{ ft}$$

For the story shear force $V_x = 15$ kip and excentricity of 19.3 ft, the torsional moment is

$$T = 15 \times 19.3 = 289.5 \text{ kip-ft} \qquad \text{(Eq. 20.22)}$$

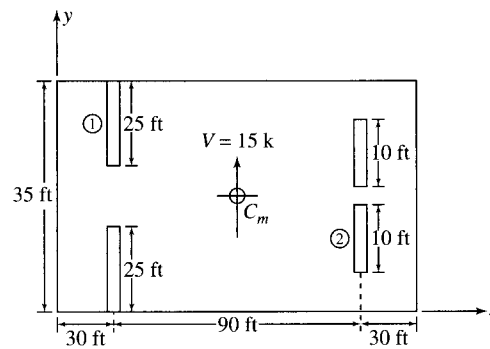


Figure 20.9 Example 20.5: floor plan.

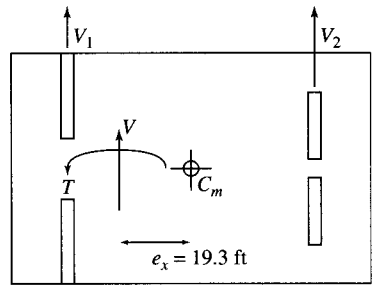


Figure 20.10 Example 20.5: torsional effect.

The shear force acting on the wall is the sum of the shear force due to story shear, V_x, and shear force due to torsional moment, T_x.

For wall 1, shear force V_1 is

$$V_1 = 15 \times 25/(25 + 25 + 10 + 10) = 5.4 \text{ kip} \quad \text{(due to } V_x\text{)}$$

$$V_1 = 289.5\ (25 \times 25.7)/(2 \times 25 \times 25.7^2 + 2 \times 10 \times 64.3^2) = 1.6 \text{ kip-ft} \quad \text{(due to } T\text{)}$$

Therefore, $V_1 = 5.4 + 1.6 = 7$ kip. For wall 2, shear force V_2 is

$$V_2 = 15 \times 10/(25 + 25 + 10 + 10) = 2.1 \text{ kip} \quad \text{(due to } V_x\text{)}$$

$$V_2 = 289.5\ (10 \times 64.3)/(2 \times 25 \times 25.7^2 + 2 \times 10 \times 64.3^2) = 1.6 \text{ kip-ft} \quad \text{(due to } T\text{)}$$

Therefore, $V_2 = 2.1 + 1.6 = 3.7$ kip.

20.4 LOAD COMBINATIONS

A structure should be designed to resist the combined effects of the loadings. Basic load combinations for strength design are given (IBC 2006, Section 1605.2.1):

1. $4D$
2. $1.2\ D + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (f_1 L \text{ or } 0.8\ W)$
4. $1.2D + 1.6W + f_1 L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + f_1 L + f_2 S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + (1.0E \text{ or } 1.6W)$

where

$f_1 = 1.0$ for the floors in places of public assembly, for live loads in excess of 100 pounds per square foot, and for parking garage live load

$f_1 = 0.5$ for other live loads

$f_2 = 0.7$ for roof configurations that do not shed snow off the structure

$f_2 = 0.2$ for the other roof configurations

D = dead load
L = live load excluding roof live load
L_r = roof live load
S = snow load
R = rain load
W = wind load
E = seismic load effect

Special seismic load combinations for strength design should be used in a case of structures having certain plan or vertical irregularities in SDC B or higher. Special seismic load combinations for strength design are (IBC 2006, Section 1605.4):

1. $1.2D + f_1 L + E_m$
2. $0.9D + E_m$

where

f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 100 psf, and for parking garage live load
f_1 = 0.5 for other live loads
E_m = the maximum effect of horizontal and vertical forces

20.4.1 Calculation of Seismic Load Effect, *E*

The seismic load effect (ASCE 7-05, Section 12.4) can be determined from the following two conditions:

1. The seismic load effect E is calculated from

$$E = \rho Q_E + 0.2 S_{DS} D \tag{20.30}$$

where

Q_E = effect of horizontal seismic forces
ρ = redundancy coefficient
S_{DS} = the design spectral response acceleration at short periods determined in Section 20.2.2
D = effect of dead load

2. When the effect of gravity and seismic ground motions are counteractive, the seismic load effect is calculated from

$$E = \rho Q_E - 0.2 S_{DS} D \tag{20.31}$$

20.4.2 Redundancy Coefficient, ρ

Redundancy coefficient can be determined as follows (ASCE 7-05, Section 12.3.4)

1. For structures assigned to seismic design category A, B, or C, the value of the redundancy coefficient, ρ, is 1.
2. For structures assigned to seismic design category D, E, or F, the redundancy coefficient ρ, shall be taken equal to 1.3.

20.4.3 Seismic Force Effect, E_m

When the effects of gravity and seismic forces are additive, the seismic force effect, E_m, should be calculated using the following equation:

$$E_m = \Omega_0 Q_E + 0.2 S_{DS} D \tag{20.32}$$

where

Ω_0 = the system overstrength factor given in Table 20.7

When the effects of gravity and seismic forces counteract the seismic force effect, E_m, should be calculated using the following equation:

$$E_m = \Omega_0 Q_E - 0.2 S_{DS} D \tag{20.33}$$

20.5 SPECIAL REQUIREMENTS IN DESIGN OF STRUCTURES SUBJECTED TO THE EARTHQUAKE LOADS

The ACI Code (2008), Section 20.1.1.9.1 and 21.1, define five seismic design categories (SDCs) for earthquake-resistant structures. These are A, B, C, D, E, and F. The classification of these zones described in ACI Section R21.1.1 can be given in three different categories:

1. SDC D, E, and F indicate high seismic risk zones with strong ground shaking
2. SDC C indicates moderate/intermediate seismic risk zones with moderately strong ground shaking.
3. SDC A and B indicate low seismic risk zones with SDC A corresponding to the lowest seismic hazard zone.

For structures in high seismic risk (SDC D, E, and F) special requirements in flexural design and detailing are required. Special moment frames (Section 20.5.1) and special structural walls (Section 20.5.1.0) should be used as the structural system of a building.

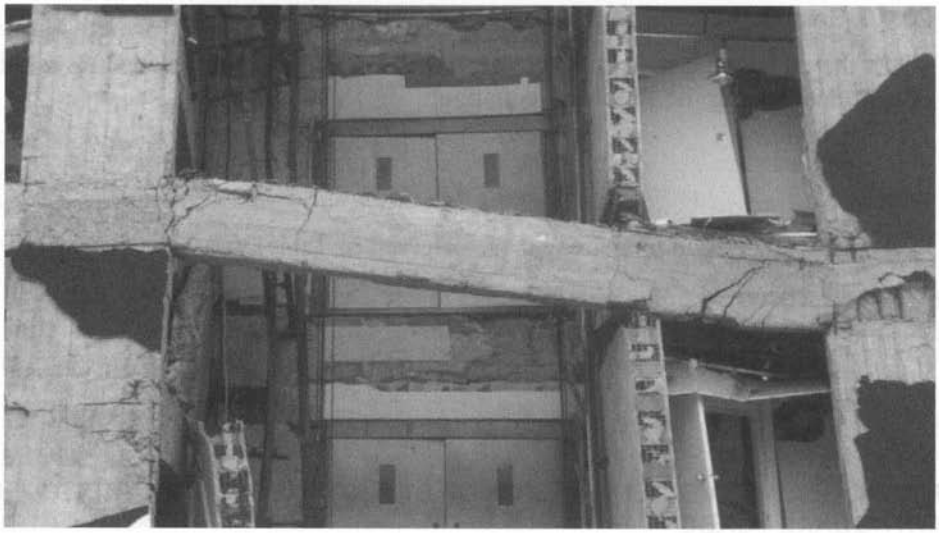
For the structures in moderate seismic risk (SDC C) some special provisions are required for satisfactory intermediate seismic performance (Section 20.5.2). Structure can be designed as intermediate moment-frame or intermediate structural-walls systems. Structures from a higher category can also be utilized.

For the structures in low seismic risk (SDC A and B), no special requirements in flexural design and detailing are required. Ordinary moment frames and ordinary structural walls and systems should be utilized as the structural system of a building.

20.5.1 Structures in the High Seismic Risk: Special Moment Frames (ACI 2008, Section 21.5)

A special moment frame is a structural system that is designed and detailed to sustain strong earthquakes. Special provisions for designing and detailing are given for

1. Flexural members of special moment frames such as members subjected to only bending
2. Special moment frame members subjected to bending and axial load such as columns
3. Joints of special moment frames



Strong column-weak beam connection.

Flexural members of special moment frame (Section 20.5.1.1).

General requirements (Section 20.5.1.1.1). If factored axial compressive force $P_u < A_g f'_c/10$, then the member is considered to be subjected to bending. A_g represents the gross area of the concrete member. Flexural member should satisfy following the conditions (ACI 2008, Section 21.5.1):

1. Clear span $l_n \geq 4 \times$ effective depth (d).
2. The flexural member width-to-depth ratio, $b_w/d \geq 0.3$.
3. Flexural member width (b_w) ≥ 10 in.
4. Flexural member width (b_w) $<$ width of supporting member (column), $b_s + (1.5 \times$ depth of the flexural member, $h)$.

Longitudinal Reinforcement Requirements (Section 20.5.1.1.2). According to the ACI Code (2008), Section 21.5.2, the longitudinal reinforcement at any section should satisfy the following (Fig. 20.11):

1. Longitudinal reinforcement for both top and bottom steel (A_s) should be in the range defined as follows:

$$\left.\begin{array}{c} \dfrac{3\sqrt{f'_c}bd}{f_y} \\ \dfrac{200bd}{f_y} \end{array}\right\} \leq (A_s) \leq 0.025bd \tag{20.34}$$

At least two bars should be provided continuously at both top and bottom. For the statically determined T-sections with flanges in tension the value of b in the expression $3\sqrt{f'_c}bd/f_y$ should be replaced by either $2b$ (width of web) or the width of the flange, whichever is smaller (ACI 2008, Section 10.5.2).

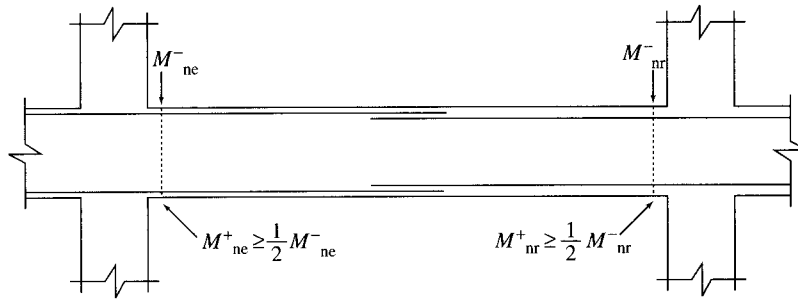


Figure 20.11 Longitudinal reinforcement requirements.

2. The positive moment strength at joint face should be greater or equal $\frac{1}{2}$ negative moment strength at that face of the joint (ACI Section 21.5.2.2):

$$\phi M^+_{n_l} \geq \frac{1}{2}\phi M^-_{n_l} \qquad \text{(left joint)} \tag{20.35a}$$

$$\phi M^+_{n_r} \geq \frac{1}{2}\phi M^-_{n_r} \qquad \text{(right joint)} \tag{20.35b}$$

where

M_{n_l} = moment strength at left joint of flexural member
M_{n_r} = moment strength at right joint of flexural member

3. Neither the negative nor positive moment strength at any section along the member should be less than $\frac{1}{4}$ the maximum moment strength provided at the face of either joint.

$$(\phi M^+_n \text{ or } \phi M^-_n) \geq \frac{1}{4}(\max \phi M_n \text{ at either joint}) \tag{20.36}$$

4. Anchorage of flexural reinforcement in support can be calculated using the following equation:

$$l_{dh} \geq \begin{cases} \dfrac{f_y d_b}{65\sqrt{f'_c}} \\ 8d_b \\ 6 \text{ in.} \end{cases} \tag{20.37}$$

where d_b is the diameter of longitudinal reinforcement.

5. Lap splices of flexural reinforcement are permitted only if hoop or spiral reinforcement is provided over the lap length. Hoop or spiral reinforcement spacing should not exceed $d/4$ or 4 in., whichever is smaller. Lap splices should not be used within a joint, within a distance of twice the member depth from the face of the joint, or at locations of plastic hinges.

Transverse Reinforcement Requirements (Section 20.5.1.1.3) For the special moment-resisting frame, plastic hinges will form at the ends of flexural members. Those locations should be specially detailed to ensure sufficient ductility of the frame members. Transverse reinforcement

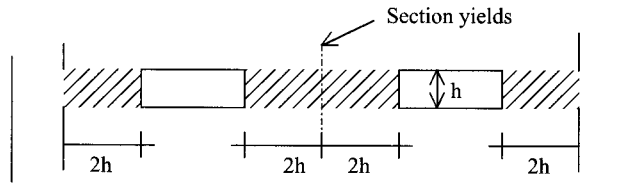


Figure 20.12 Areas of the flexural member where hoops are required. (Note: These areas do not necessarily occur at midspan.)

gives lateral support for the longitudinal reinforcement and assists concrete to resist shear. It should satisfy the following: (ACI 2008, Section 2.1.5.3)

1. Hoops are required over a length equal to twice the member depth from the face of the support at both ends of flexural member. Also, hoops are required over lengths equal to twice the member depth on both sides of section where flexural yielding may occur, as shown in Fig. 20.12.
2. The spacing of the hoops, s, should not exceed the smallest of the following values:
   a. $d/4$
   b. Eight times the diameter of the smallest longitudinal bar
   c. 24 times the diameter of the hoop bars
   d. 12 in.

   The first hoop should be located not more than 2 in. from the face of the support.
3. Where hoops are not required, stirrups with seismic hooks at both ends should be used. Spacing between stirrups should be less than or equal to $d/2$.
4. Transverse reinforcement should be designed to resist the design shear force (Figs. 20.13 and 20.14). Design shear force for flexural members of special moment frames can be determined using the following equation (Fig. 20.15):

$$V_l = \frac{M_{\text{pr}}^- + M_{\text{pr}}^+}{l_n} + \frac{w_u l_n}{2} \tag{20.38a}$$

$$V_r = \frac{M_{\text{pr}}^+ + M_{\text{pr}}^-}{l_n} - \frac{w_u l_n}{2} \tag{20.38b}$$

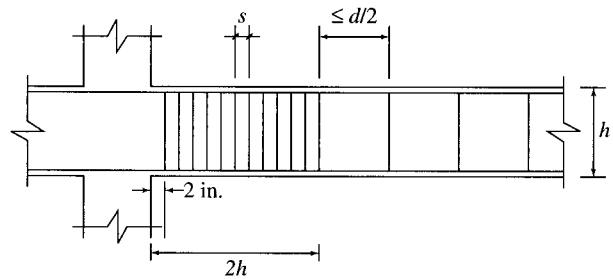


Figure 20.13 Transverse reinforcement requirements.

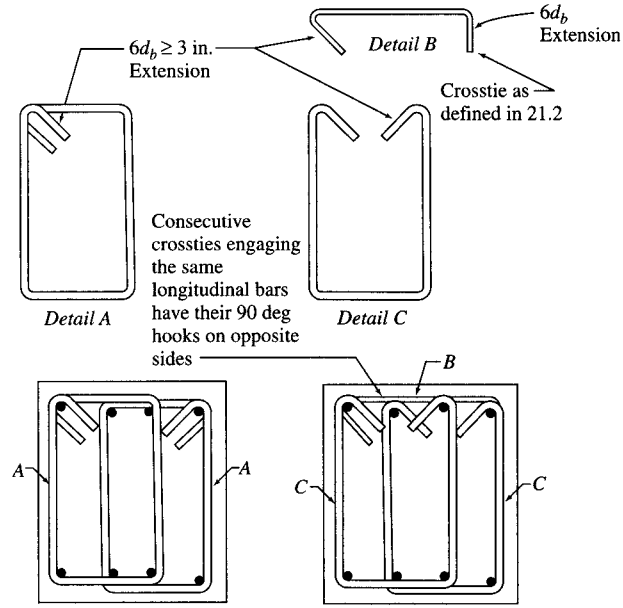


Figure 20.14 Transverse reinforcement requirements. Courtesy of American Concrete Institute (ACI 2008).

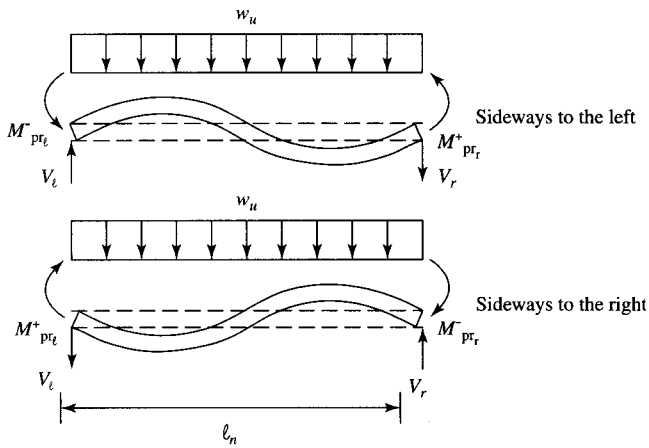


Figure 20.15 Design shear force.

where

V_l = design shear force at left joint of flexural member

V_r = design shear force at right joint of flexural member

M_{pr} = probable moment strength at the end of the beam determined as strength of the beam with the stress in the reinforcing steel equal to 1.25 f_y and a strength reduction factor of $\phi = 1.0$.

l_n = clear span of flexural member
w_u = factored distributed load determined by Eq. 20.47

$$w_u = 1.2D + 1.0L + 0.2S \tag{20.39}$$

where

D = dead load
L = live load
S = snow load

Probable moment strength at the end of the beam, M_{pr}, can be calculated from the following equation:

$$M_{\text{pr}} = A_s(1.25f_y)\left(d - \frac{a}{2}\right) \tag{20.40}$$

where

$$a = \frac{A_s(1.25f_y)}{0.85f_c'b} \tag{20.41}$$

The shear strength of concrete can be taken to be 0 when the earthquake-induced shear force is greater than or equal to 50% of the total shear force and the factored axial compressive force is less than $A_g f_c'/20$, where A_g is the gross area of the beam.

Summary: Design of the Special Moment-Resisting Frame Members Subjected to Bending (Section 20.5.1.1.4)

Step 1. Determine the seismic design category, base shear, lateral seismic force, and seismic shear according to Sections 20.2 and 20.3.

Step 2. Calculate the member forces, and use the different load combinations to determine the values of member forces that govern the design (Section 20.4). Design for flexural reinforcement.

Step 3. Check whether the frame member is a flexural member and check the general requirements for the special moment frame member according to Section 20.5.1.1.1.

Step 4. Check the special requirements for the longitudinal reinforcement according to Section 20.5.1.1.2.

Step 5. Design the transverse reinforcement for confinement and shear resistant using Section 20.5.1.1.3.

Example 20.6

Design a beam AB on the second floor of a building, as shown on Fig. 20.16. The building is constructed in the region of high seismic risk on soil class B. Additional information:

Material properties: Concrete: $f_c' = 4000$ psi, $w_c = 150$ pcf
Steel $f_y = 60{,}000$ psi

Loads: Live loads = 40 psf
Superimposed dead load = 35 psf

Member dimensions: Beams = 20 × 24 in.
Columns = 24 × 24 in.
Slab thickness = 7 in.

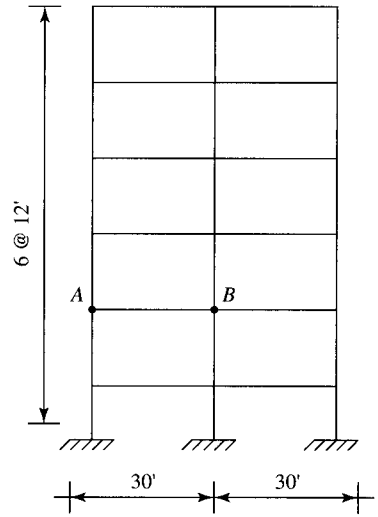


Figure 20.16 Example 20.6:1 building elevation.

Solution

1. Seismic design category, base shear, lateral seismic force, and seismic shear are determined in Example 20.3.
2. Load combinations are given as follows:

$$1.4D \tag{I}$$

$$1.2D + 1.6L \tag{II}$$

$$1.2D + 1.0E + f_1 L \quad f_1 = 0.5 \text{ according to Section 20.4} \tag{III}$$

$$0.9D + 1.0E \tag{IV}$$

Redundancy coefficient, ρ, can not be taken less than 1. For seismic design catagory D to F use $f = 1.3$.

Seismic load effect, E, can be determined using Eqs. 20.30 and 20.31:

$$E = \rho Q_E + 0.2 S_{DS} D = Q_E + 0.2(1.0)D = Q_E + 0.2D$$

$$E = \rho Q_E - 0.2 S_{DS} D = Q_E - 0.2(1.0)D = Q_E - 0.2D$$

Replacing the E in Eq. III gives

$$1.4D + 0.5L + Q_E$$

$$D + 0.5L + Q_E$$

Replacing the E in Eq. IV gives

$$1.1D + Q_E$$

$$0.7D + Q_E$$

The member forces for the beam AB on the second floor (Fig. 20.16) are calculated using the software for load analysis, and the values of required flexural strengths are determined using different load combinations, as shown in Table 20.10.

Table 20.10 Calculated Member Forces

Load Cases	Location	Bending Moment (kip-ft)	Shear (kip)
D	Support	−95	24
	Midspan	−465	
L	Support	−22	11
	Midspan	15	
Q_E	Support	±290	
	Midspan	0	±25
Load Combinations			
$1.4\,D$	Support	−133	33.6
	Midspan	91	
$1.2D + 1.6L$	Support	−149	46.4
	Midspan	102	
$1.4D + 0.5L + Q_E$	Support	−434/146*	64.1
	Midspan	98.5	
$D + 0.5L + Q_E$	Support	−396/184*	54.5
	Midspan	−473	
$1.1D + Q_E$	Support	−395/186*	51.4
	Midspan	−471.5	
$0.7D + Q_E$	Support	−357/224*	41.8
	Midspan	−446	

*Q_E has negative and positive value.

Table 20.11 Calculation of Longitudinal Reinforcement

Location	M_u (kip-ft)	A_s (in.^{2})	Reinforcement Used	ϕM_n (kip-ft)
Support	−434	5.20	7 no. 8 ($A_s = 5.53$ in.^{2})	−474
	224	3.01	6 no. 7 ($A_s = 3.6$ in.^{2})	358
Midspan	102	1.21	2 no. 7 ($A_s = 1.2$ in.^{2})	113

From the previous table the most critical loads are chosen and summarized in Table 20.11. Longitudinal reinforcement for the beam is also determined in Table 20.11.
Table 20.12 summarizes the reinforcement used for the beam.

3. General requirements for flexural members of special moment frame are checked as follows:
   a. Clear span ≥ 4 effective depth

$$28 \text{ ft} \geq 4\frac{21.5}{12} = 7.2 \text{ ft} \quad \text{(o.k.)}$$

   b. Width-to-depth ratio ≥ 0.3

$$\frac{20}{24} = 0.83 > 0.3 \quad \text{(o.k.)}$$

   c. Width = 20 in. ≥ 10 in. (o.k.)
   d. Width ≤ width of supporting member + distance on each side of the supporting member not exceeding three-fourths of the depth of the flexural member

$$20 \text{ in.} \leq 24 \text{ in. (o.k.)}$$

$$20 \text{ in.} \leq 24 + (1.5 \times 26) = 63 \text{ in.} \quad \text{(o.k.)}$$

Table 20.12 Summary of Reinforcement

Location	Reinforcement Provided	
	Top	Bottom
Support	7 no. 8 (5.53 in.^{2})	6 no. 7 (3.6 in.^{2})
Midspan	2 no. 8*(1.58 in.^{2})	2 no. 7 (1.2 in.^{2})

*Two no. 8 bars are extended from seven no. 8 support bars into the negative moment zone at midspan.

4. Special requirements for longitudinal reinforcement are

a.

$$(A_s^- \text{ or } A_s^+) \geq \begin{cases} \dfrac{3\sqrt{f'_c}b_w d}{f_y} = \dfrac{3\sqrt{4000} \times 20 \times 21.5}{60000} = 1.36 \text{ in.}^2 \\ \dfrac{200 b_w d}{f_y} = \dfrac{200 \times 20 \times 21.5}{60000} = 1.43 \text{ in.}^2 \end{cases}$$

$$\max A_S = 0.025 b_w d = 0.025 \times 20 \times 21.5 = 10.75 \text{ in.}^2$$

Check the reinforcement limits against the required reinforcement, as shown in Table 20.13.

b. Positive moment strength at joint face $\geq \frac{1}{2}$ negative moment strength at that face of the joint:

$$M_n^+ = 358 \text{ kip-ft} \geq \frac{1}{2} Mn^- = \frac{1}{2} 474 = 237 \text{ kip-ft} \qquad \text{(o.k.)}$$

c. (M_n^- or M_n^+) at any section $\geq \frac{1}{4}$ (max M_n at either joint) (ACI 20.3.2.2)

$$M_n = 148 \text{ kip-ft} > \frac{1}{4} 474 = 119 \text{ kip-ft} \quad \text{(o.k.)}$$

Anchorage of flexural reinforcement in exterior column is determined as follows:

For no. 8 bars,

$$l_{dh} = \begin{cases} \dfrac{60000 \times 1.0}{65\sqrt{4000}} = 14.6 \text{ in.} \\ 8 \times 1.0 = 8 \text{ in.} \\ 6 \text{ in.} \end{cases}$$

Therefore, $l_{dh} = 14.6$ in.

Table 20.13 Longitudinal Reinforcement Requirements According to the Limits of the Reinforcement

	Reinforcement Used	Limits			
	A_s (in.^{2})	Min A_s (in.^{2})	Max A_s (in.^{2})	Required A_s (in.^{2})	ϕM_n (kip-ft)
Support (joint face)	5.53 (7 no. 8 at the top)			5.53	−474
	3.6 (6 no. 7 at the bottom)	1.43	10.75	3.6	358
Midspan	1.58 (2 no. 8 at the top)			1.58	−148
	1.2* (2 no. 7 at the bottom)			1.8* (3 no. 7)	168

*Since 1.2 in.^{2} < min A_s = 1.43 in.^{2}, use three no. 7 bars at the bottom (A_s = 1.8 in.^{2}).

For no. 7 bars,

$$l_{dh} = \begin{cases} \dfrac{60000 \times 0.875}{65\sqrt{4000}} = 12.8 \text{ in.} \\ 8 \times 0.875 = 7 \text{ in.} \\ 6 \text{ in.} \end{cases}$$

Therefore, $l_{dh} = 12.8$ in.

5. Transverse reinforcement is determined as follows:

$$V_e = \frac{(M_{pr}^{\pm})_l + (M_{pr}^{\pm})_r}{l_n} + \frac{w_u l_n}{2}$$

$$M_{pr} = A_s(1.25 f_y)\left(d - \frac{a}{2}\right)$$

For six no. 7 bottom bars,

$$a = \frac{A_s(1.25 f_y)}{0.85 f_c' b} = \frac{3.6(1.25 \times 60)}{0.85 \times 4 \times 20} = 3.97 \text{ in.}$$

$$M_{pr} = A_s(1.25 f_y)\left(d - \frac{a}{2}\right) = 3.6(1.25 \times 60)\left(21.5 - \frac{3.97}{2}\right) = 5269 \text{ kip-in.} = 439 \text{ kip-ft}$$

For seven no. 8 bars,

$$a = \frac{A_s(1.25 f_y)}{0.85 f_c' b} = \frac{5.53(1.25 \times 60)}{0.85 \times 4 \times 20} = 6.1 \text{ in.}$$

$$M_{pr} = A_s(1.25 f_y)\left(d - \frac{a}{2}\right) = 5.53(1.25 \times 60)\left(21.5 - \frac{6.1}{2}\right)$$

$$= 7652 \text{ kip-in.} = 638 \text{ kip-ft}$$

$$w_u = 1.2 w_D + 0.5 w_L = 2.78 \text{ kip/ft}$$

$$V_l = \frac{M_{pr}^{-} + M_{pr}^{+}}{l_n} + \frac{w_u l_n}{2} = \frac{638 + 439}{26} + \frac{2.78 \times 26}{2} = 77.6 \text{ kip}$$

$$V_r = \frac{M_{pr_l}^{+} + M_{pr_r}^{-}}{l_n} - \frac{w_u l_n}{2} = \frac{638 + 439}{26} - \frac{2.78 \times 26}{2} = 5.3 \text{ kip}$$

Maximum earthquake induced shear force $= \dfrac{439 + 638}{26} = 41.4 \text{ kip} > \dfrac{77.6}{2} = 38.8 \text{ kip}$

$$\Rightarrow V_c = 0$$

$$\phi V_s = V_u - V_c$$

$$V_s = \frac{77.6}{0.75} - 0 = 104 \text{ kip}$$

$$V_s = 104 \text{ k} < (V_{s\max} = 8\sqrt{f_c'} b_w d = 8\sqrt{4000} \times 20 \times 21.5 = 217.6 \text{ kip} \quad \text{(o.k.)}$$

$$V_s = 104 \text{ k} < 4\sqrt{f_c'} b_w d = 4\sqrt{4000} \times 20 \times 21.5 = 109 \text{ kip} \quad \text{(o.k.)}$$

Required spacing for no. 3 stirrups is determined as

$$s = \frac{A_s f_y d}{V_s} = \frac{(4 \times 0.11) \times 60 \times 21.5}{104} = 5.5 \text{ in.}$$

Maximum spacing of the hoops within a distance of $2h = 2 \times 24 = 48$ in. shall not exceed the smallest of

$$\frac{d}{4} = \frac{21.5}{4} = 5.4 \text{ in.}$$

Eight times the diameter of the smallest longitudinal bar $= 8 \times 0.875 = 7$ in.
24 times the diameter of the hoop bars $= 24 \times 0.375 = 9$ in.
12 in.

Therefore, use 10 no. 3 hoops at each end of the beam at 5 in. center-to-center with the first hoop located at 2 in. from the face of the support.

At the distance 48 in. from the face of the support shear strength is

$$V_u = 77.66 - 2.78 \times \frac{48}{12} = 66.48 \text{ kip}$$

The shear strength contributed by concrete is

$$V_c = 2 \times 1 \times \sqrt{4000} \times 20 \times 21.5 = 54.4 \text{ kip}$$

$$V_s = \frac{66.48}{0.75} - 54.4 = 34.2 \text{ kip}$$

Spacing of the stirrups should not be taken greater than

$$s = \frac{d}{2} = \frac{24}{2} = 12 \text{ in.}$$

or

$$s = \frac{A_s f_y d}{50b} = \frac{(2 \times 0.11) \times 60 \times 21.5}{50 \times 20} = 13.2 \text{ in.}$$

or

$$s = \frac{d}{2} = \frac{24}{2} = 12 \text{ in.}$$

Therefore, use stirrups with seismic hoops spaced 8 in. center-to-center starting at 48 in. from the face of the support. Figure 20.17 shows reinforcement detailing.

Special moment frame members subjected to bending and axial load (Section 20.5.1.2).

General requirements (Section 20.5.1.2.1) The requirements of this section apply to columns and other flexural members that carry a factored axial load $> A_g f'_c/10$. These members should satisfy both of the following conditions (ACI 2008, Section 21.6):

1. Shortest cross-section dimension $\geq$ 12 in.
2. The ratio of shortest cross-sectional dimension to the perpendicular dimension ≥ 0.4

Longitudinal reinforcement requirements (Section 20.5.1.2.2) According to the ACI Code 2008, Section 21.5.2, the flexural strengths of columns should satisfy the following:

$$\sum M_{nc} \geq \frac{6}{5} \sum M_{nb} \tag{20.42}$$

where

$\sum M_{nc}$ = sum of nominal flexural strengths of the columns framing into the joint, evaluated at the faces of the joint.

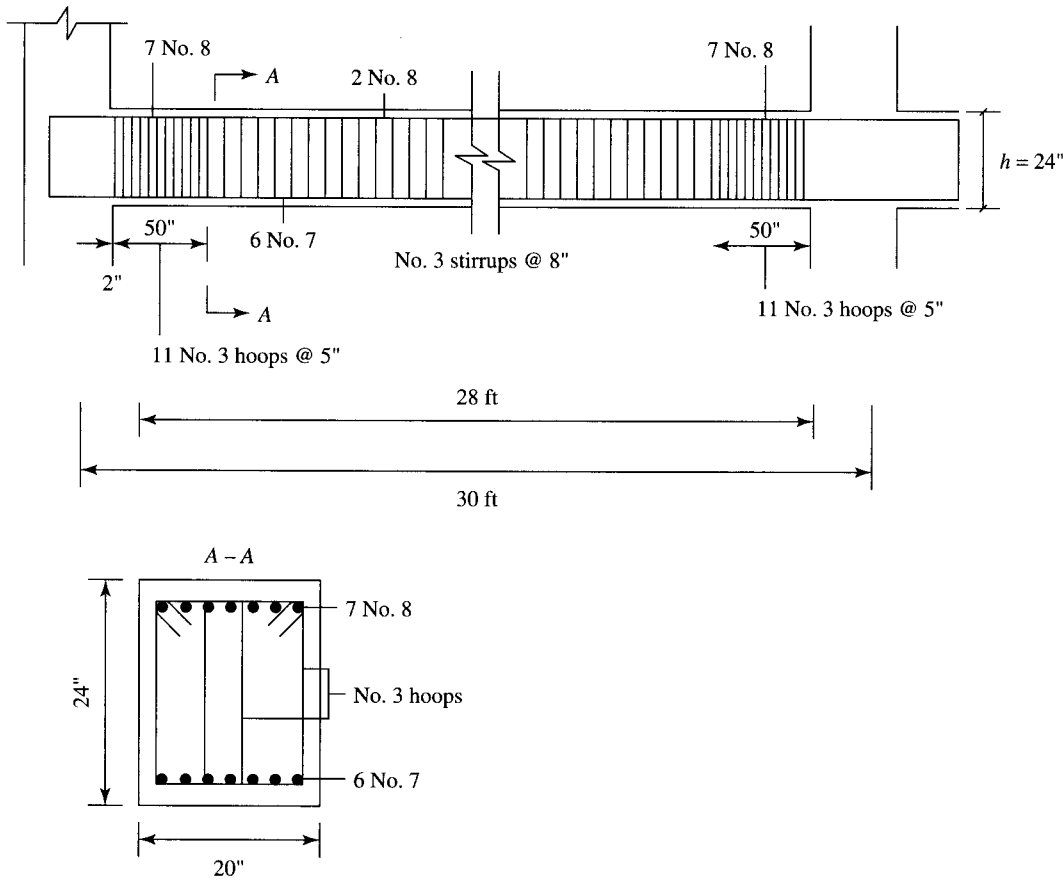


Figure 20.17 Example 20.6 reinforcement detailing.

$\sum M_{nb}$ = sum of nominal flexural strengths of the beams framing into the joint, evaluated at the faces of the joint.

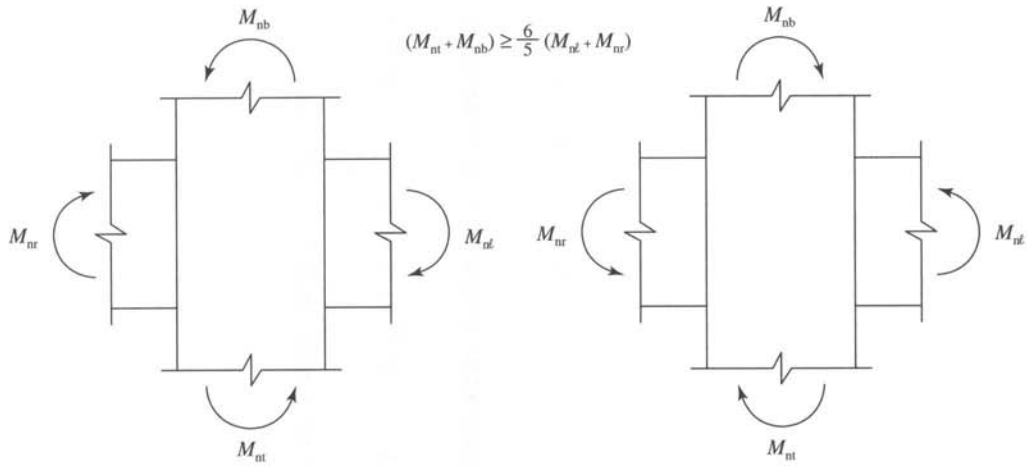
This approach, called strong column—weak beam concept (Fig. 20.18), ensures that columns will not yield before the beams. The main steel reinforcement should be chosen to satisfy Eq. 20.61.

The reinforcement ratio should satisfy the following:

$$0.01 \leq \rho_g \leq 0.06 \tag{20.43}$$

Transverse reinforcement requirements (Section 20.5.1.2.3) Columns should be properly detailed to ensure column ductility in the case of plastic hinge formation, and should also have the adequate shear strength to prevent shear failure.

The following transverse reinforcement requirements need to be provided only over the length l_o greater or equal to depth of the member, $\frac{1}{6}$ clear span, 18 in., from the each joint face and on both sides of any section where yielding is likely to occur (ACI code 2008, Section 21.6.4.1). The requirements are



Subscripts ℓ, r, t, and b stand for left support, right support, top of column, and bottom of column, respectively.

Figure 20.18 Strong column — weak beam concept. Courtesy of American Concrete Institute (ACI 2008).



Lack of transverse reinforcement.

1. Ratio of spiral reinforcement, ρ_s, should satisfy the following (Fig. 20.19):

$$\rho_s \geq \begin{cases} 0.12\dfrac{f'_c}{f_{yt}} \\ 0.45\left(\dfrac{A_g}{A_c} - 1\right)\dfrac{f_c}{f_{yt}} \end{cases} \tag{20.44}$$

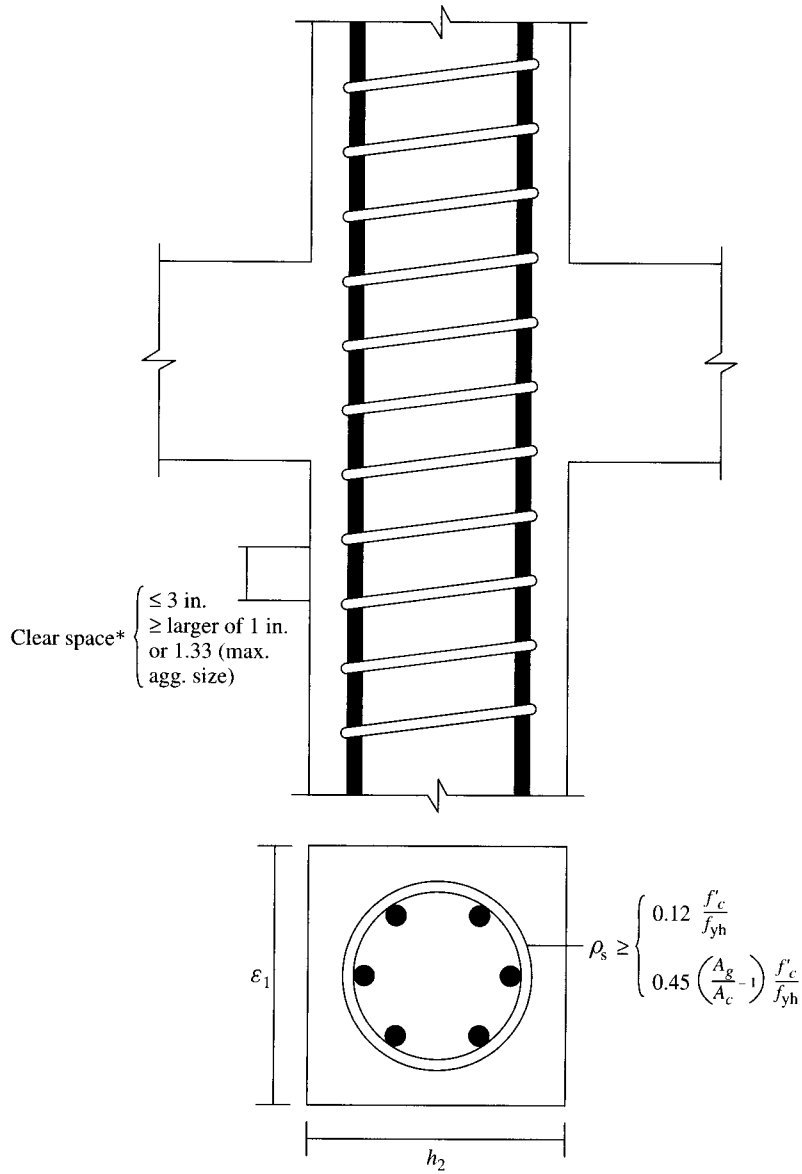


Figure 20.19 Transverse reinforcement requirements for spiral reinforcement. Courtesy of Portland Cement Association (notes on ACI 318).

where

f_{yt} = yield stress of transverse reinforcement
A_c = area of core of spirally reinforced compression member measured to outside diameter of spiral
A_g = gross area of section

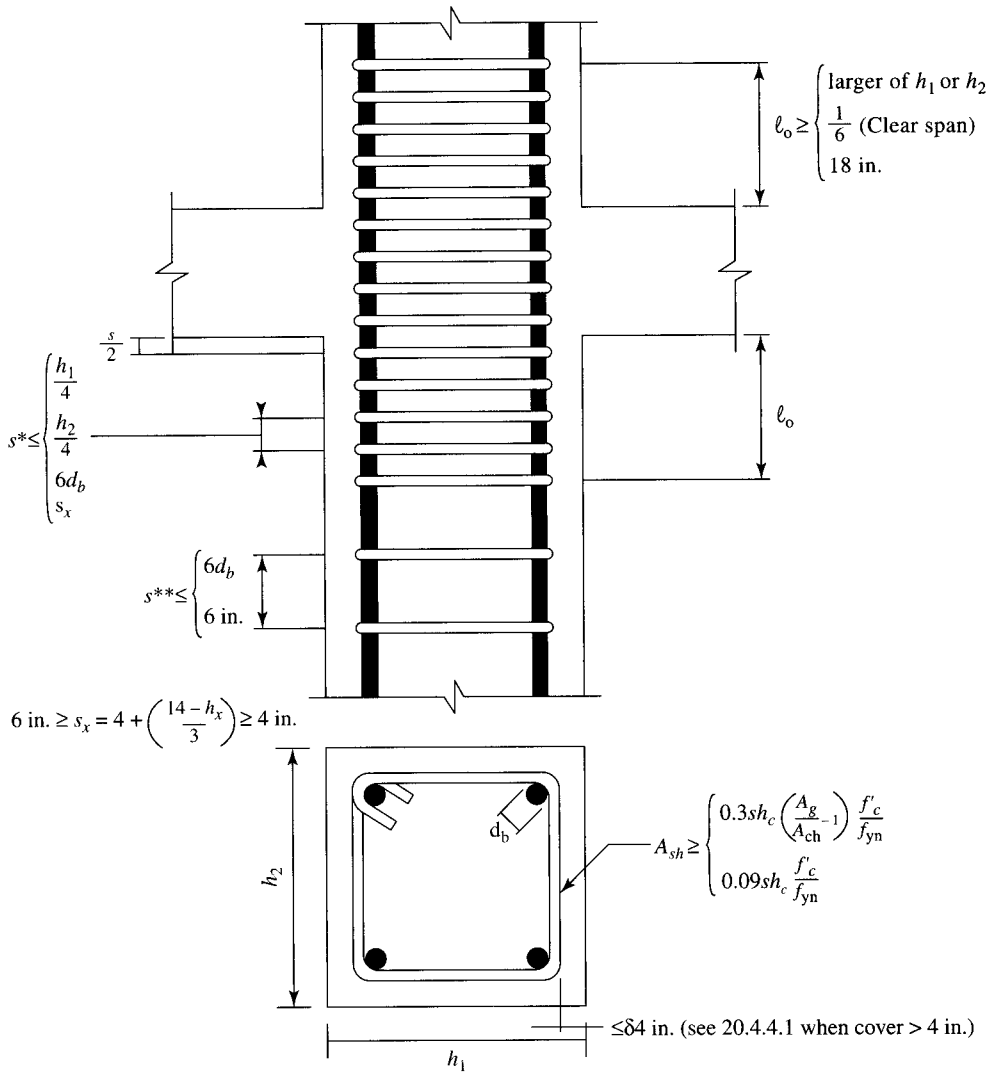


Figure 20.20 Transverse reinforcement requirements for rectangular hoop reinforcement. Courtesy of Portland Cement Association (notes on ACI 318).

2. Total cross-section area of rectangular hoop reinforcement, A_{sh}, should satisfy the following (Fig. 20.20):

$$A_{sh} \geq \begin{cases} 0.3\left(\dfrac{sh_c f'_c}{f_{yt}}\right)\left(\dfrac{A_g}{A_{ch}} - 1\right) \\ 0.09\dfrac{sh_c f'_c}{f_{yt}} \end{cases} \tag{20.45}$$

where

s = spacing of transverse reinforcement
h_c = cross-section dimension of column core measured center-to-center of the confining reinforcement.

3. If the thickness of the concrete outside the confining transverse reinforcement exceeds 4 in., additional transverse reinforcement should be provided at a spacing $\leq$ 12 in. Concrete cover on additional reinforcement should not exceed 4 in.
4. Spacing of the transverse reinforcements should satisfy the following:

$$s \leq \begin{cases} \dfrac{h}{4} \\ 6 \times \text{longitudinal diameter bar} \\ s_o \end{cases} \tag{20.46}$$

$$\text{Also, } 4 \text{ in.} \leq s_o = 4 + \frac{14 - h_x}{3} \leq 6 \text{ in.} \tag{20.47}$$

where

s_o = longitudinal spacing of transverse reinforcement within the length l_o.
h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the column.

The remaining member length should be reinforced with the spiral or hoop transverse reinforcement spaced as follows:

$$s \leq \begin{cases} 6 \times \text{longitudinal bar diameter} \\ 6 \text{ in.} \end{cases} \tag{20.48}$$

Transverse reinforcement should be designed to resist the design shear force. Design shear force for flexural members of special moment frames can be determined using the following equation:

$$V_u = \frac{M_{\text{Pr}_t} + M_{\text{Pr}_b}}{l_c} \tag{20.49}$$

index t is for top and index b is for bottom of the column) where,
l_c = length of the column

Summary: Design of the Special Moment-Resisting Frame Members Subjected to Bending and Axial Force (Section 20.5.1.2.4)

Step 1. Determine seismic design category, base shear, lateral seismic force and seismic shear according to Sections 20.2 and 20.3.

Step 2. Calculate the member forces and using the different load combinations determine the values of member forces that govern the design. Design the reinforcement.

Step 3. Check whether the frame member is a flexural member or whether the member is subjected to the bending and axial force, and check general requirements for the special moment frame member according to Section 20.5.1.2.1.

Step 4. Check the special requirements for the longitudinal reinforcement according to Section 20.5.1.2.2.

Step 5. Design the transverse reinforcement for confinement and shear resistant using Section 20.5.1.2.3.

Example 20.7

Design the edge column on the second floor of a building from Example 20.6.

Solution

1. The load combinations gave the following results:

$$P_u = 1022 \text{ kip} \quad \text{(maximum force at the first floor)}$$

$$P_u = 935 \text{ kip} \quad \text{(maximum force at the second floor)}$$

2.
$$P_u = 1022k > \frac{A_g f'_c}{10} = \frac{(24 \times 24) \times 4}{10} = 230 \text{ kip}$$

Member is subjected to bending and axial load. General requirements should be checked as follows:

a. Shortest cross-section dimension = 24 in. ≥ 12 in., which is o.k.

b. The ratio of shortest cross-sectional dimension to the perpendicular dimension, $\frac{24}{24} = 1 \geq 0.4$, which is o.k.

3. Longitudinal reinforcement for the column with $P_u = 1022$ kip is eight no. 8 bars. The reinforcement ratio is $\rho_g = 0.011 < 0.06$, which is o.k. and > 0.01, which is also o.k.

$$\sum M_{nc} \geq \frac{6}{5} \sum M_{nb}$$

For $P_u = 1022$ kip, $M_n = 580$ kip-ft. For $P_u = 935$ kip, $M_n = 528$ kip-ft. A minimum nominal flexural strength of the beam at the joint including the slab reinforcement is $M_n = 723$ kip-ft.

$$\sum M_{nc} = 580 + 528 = 1108 \text{ kip-ft}$$

$$\sum M_{nb} = 723 \text{ kip-ft}$$

$$\sum M_{nc} = 1108 \text{ kip-ft} \geq \frac{6}{5} \sum M_{nb} = \frac{6}{5} 723 = 868 \text{ kip-ft} \qquad \text{(o.k.)}$$

4. Length l_o is determined as follows:

$$l_o \geq \begin{cases} \text{depth of the member} = 24\text{in.} \\ \frac{1}{6}\text{clear height} = (12 \times 12)/6 = 24 \text{ in.} \\ 18 \text{ in.} \end{cases}$$

Choose $l_o = 24$ in.

$$\text{Spacing is} s \leq \begin{cases} \frac{h}{4} = \frac{24}{4} = 6 \text{ in.} \\ 6 \times \text{longitudinal diameter bar} = 6 \times 1.0 = 6 \text{ in.} \\ s_o = 4 + \left(\frac{14 - 11}{3}\right) = 5 \text{ in.} \end{cases}$$

Therefore, $s = 5$ in.

Required cross-section area of reinforcement is

$$A_{sh} \geq \begin{cases} 0.3\left(\dfrac{sh_c f'_c}{f_{yt}}\right)\left(\dfrac{A_g}{A_{ch}} - 1\right) = 0.3\left(\dfrac{5 \times 20.5 \times 4}{60}\right)\left(\dfrac{576}{441} - 1\right) = 0.63 \text{ in.}^2 \\ 0.09\dfrac{sh_c f'_c}{f_{yt}} = 0.09\dfrac{5 \times 20.5 \times 4}{60} = 0.62 \text{ in.}^2 \end{cases}$$

Choose no. 4 hoops and no. 5 crossties:

$$A_{sh} = 2 \times 0.2 + 0.31 = 0.71 \text{ in.}^2 > 0.63 \text{ in.}^2$$

Detailing of the reinforcement can be found in Fig. 20.21.

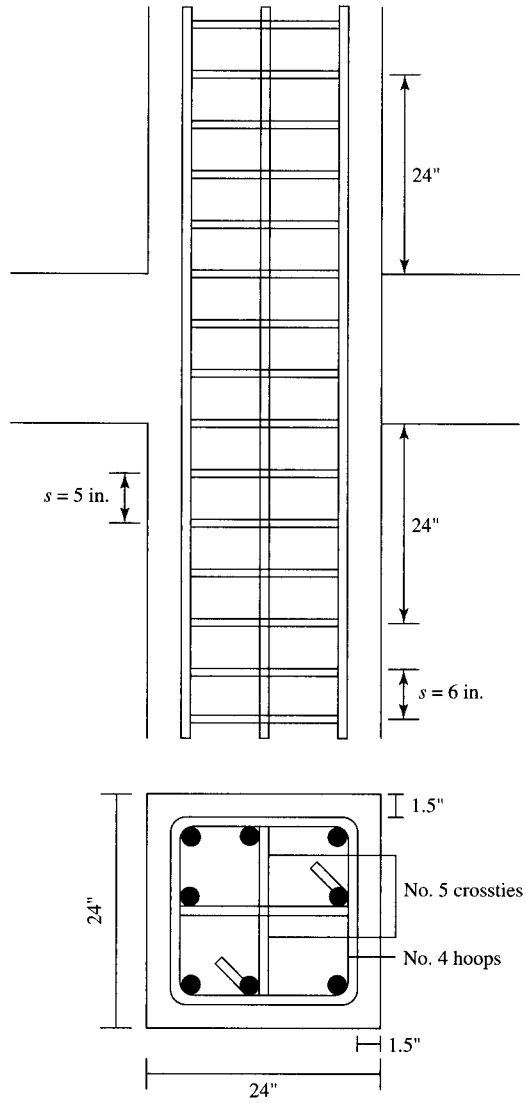


Figure 20.21 Example 20.7 reinforcement detailing.

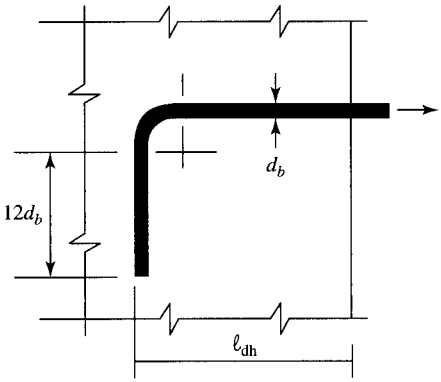


Figure 20.22 Standard 90° hooks. Courtesy of American Concrete Institute (ACI 2008).

Joints of the special moment-resisting frame (Section 20.5.1.3). Joint of special moment-resisting frame should be detailed according to the ACI Code 2008, Section 20.5, as follows:

Longitudinal Reinforcement Requirements (Section 20.5.1.3.1) The development length l_{dh} for a bar with a standard 90° hook using normal-weight concrete, for bar size no. 3 through no. 11, should be determined according to the following (Fig. 20.22):

$$l_{\text{dh}} \geq \begin{cases} \dfrac{f_y d_b}{65\sqrt{f_c'}} \\ 8d_b \\ 6 \text{ in.} \end{cases} \tag{20.50}$$

where d_b is the diameter of longitudinal reinforcement.

The development length, l_d, for a strait bar for bar size no. 3 through no. 11 should not be less than

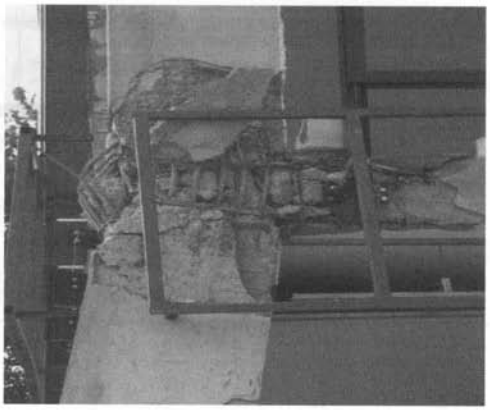
1. 2.5 l_{dh} if the depth of the concrete cast in one lift beneath the bar does not exceed 12 in.
2. 3.5 l_{dh} if the depth of the concrete cast in one lift beneath the bar exceeds 12 in.

When the longitudinal reinforcement passes through the joint, the column dimension parallel to the beam reinforcement should not be less than 20 times the diameter of the largest longitudinal bar for normal-weight concrete. For lightweight concrete, this dimension should not be less than 26 times the bar diameter.

Shear strength requirements (Section 20.5.1.3.2) The nominal shear strength of the joint for normal-weight concrete should not exceed the following:

1. $20\sqrt{f_c'}A_j$ for joints confined on all four faces
2. $15\sqrt{f_c'}A_j$ for joints confined on three faces or on two opposite faces
3. $12\sqrt{f_c'}A_j$ for all other joints

where A_j is the effective area, as shown in Fig. 20.23.



Beam-column connection (joint).

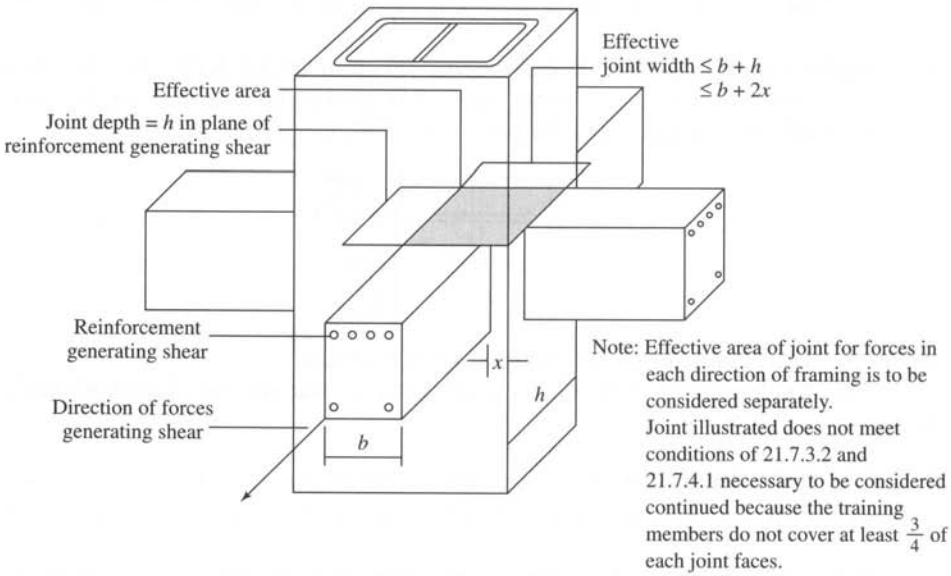


Figure 20.23 Effective *Aj* of joint. Courtesy of American Concrete Institute (ACI 2008).

20.5.2 Structures in the High Seismic Risk: Special Reinforced Concrete Structural Walls and Coupling Beams (ACI 2008 Section 21.9)

Wall system is a structural system that provides support for all gravity loads and all lateral loads applied to the structure. A structural wall system is much stiffer than a frame system and its performance during an earthquake is better than the performance of the frame system.

A structural wall should be properly designed to sustain all loads acting on it. Boundary elements of structural walls are the areas around the structural wall edges, as shown in Fig. 20.24, that strengthen by the longitudinal and transverse reinforcement. Boundary elements increase

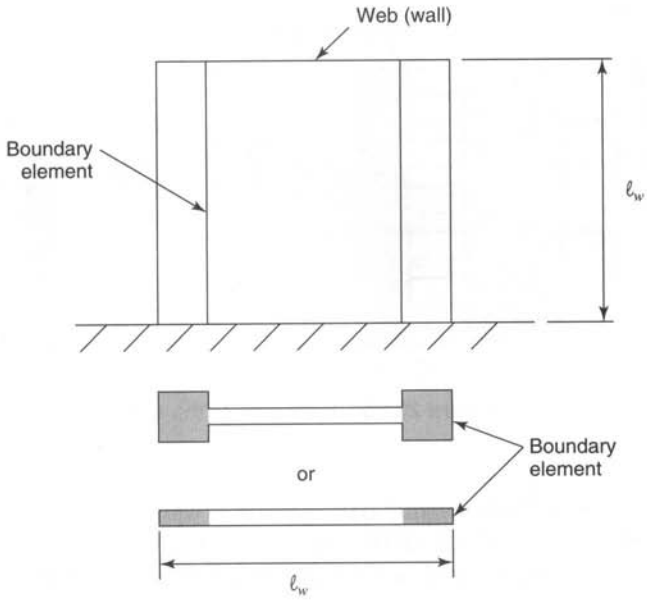


Figure 20.24 Boundary elements of structural wall.



Shear wall after an earthquake.

the rigidity and strength of wall panels. The web reinforcement is anchoraged into the boundary elements.

Figure 20.25 shows the elements of the wall with openings. The vertical wall segment bounded by two openings is called pier. A horizontal wall section between the openings is called a horizontal wall segment. When the openings are aligned vertically over the building height, the horizontal wall segments between the openings are called coupling beams.

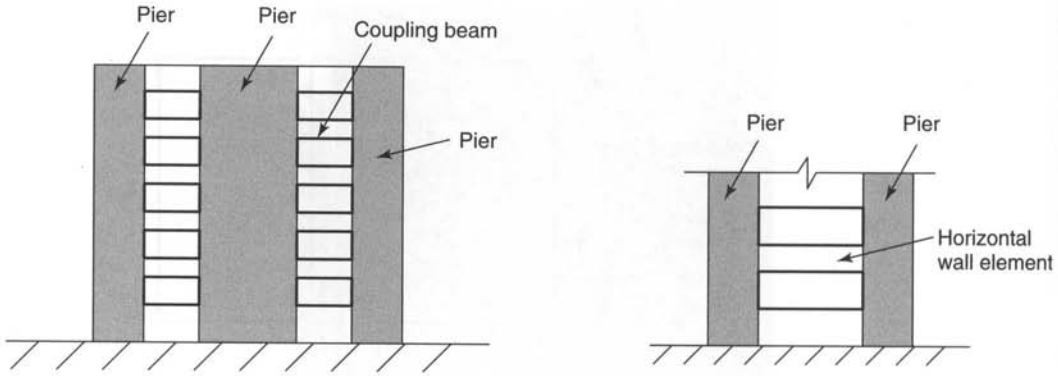


Figure 20.25 Elements of the wall with openings.

In the regions of high seismic risk, structural walls with special reinforcement requirements should be used. The ACI Code (2008) Section 21.9, gives provisions for the design and detailing of structural walls. These are described in the following sections.

Reinforcement Requirements (Section 20.5.2.1). Shear reinforcement should be provided in two orthogonal directions in the plane of the wall. (ACI 2008, Section 21.9.2.1) The minimum reinforcement ratio for both longitudinal and transverse directions can be determined as follows:

1. If the design shear $V_u > A_{cv}\lambda\sqrt{f'_c}$, the distributed web reinforcement ratios, ρ_v and ρ_n, should not be less than 0.0025.

$$\rho_l = \frac{A_{sv}}{A_{cv}} = \rho_n \geq 0.0025 \tag{20.51}$$

where

ρ_t = ratio of area of distributed reinforcement parallel to the plane of A_{cv} to gross concrete area perpendicular to that reinforcement (Fig. 20.26)

ρ_l = ratio of area of distributed reinforcement perpendicular to the plane of A_{cv} to gross concrete area A_{cv}. (Fig. 20.26)

A_{cv} = gross area of concrete section (product of thickness and length of the section in the direction of shear force)

A_{sv} = Projection on A_{cv} of area of shear reinforcement crossing the plane of A_{cv}

λ = factor for lightweight aggregate concrete

2. If the design shear $(V_u) < A_{cv}\lambda\sqrt{f'_c}$, the minimum reinforcement for ordinary structural walls can be utilized:

Minimum vertical reinforcement ratio, $\rho_l = 0.0012$ for no. 5 bars and smaller

$= 0.0015$ for no. 6 bars and larger

Minimum horizontal reinforcement ratio, $\rho_t = 0.0020$ for no. 5 bars and smaller

$= 0.0025$ for no. 6 bars and larger

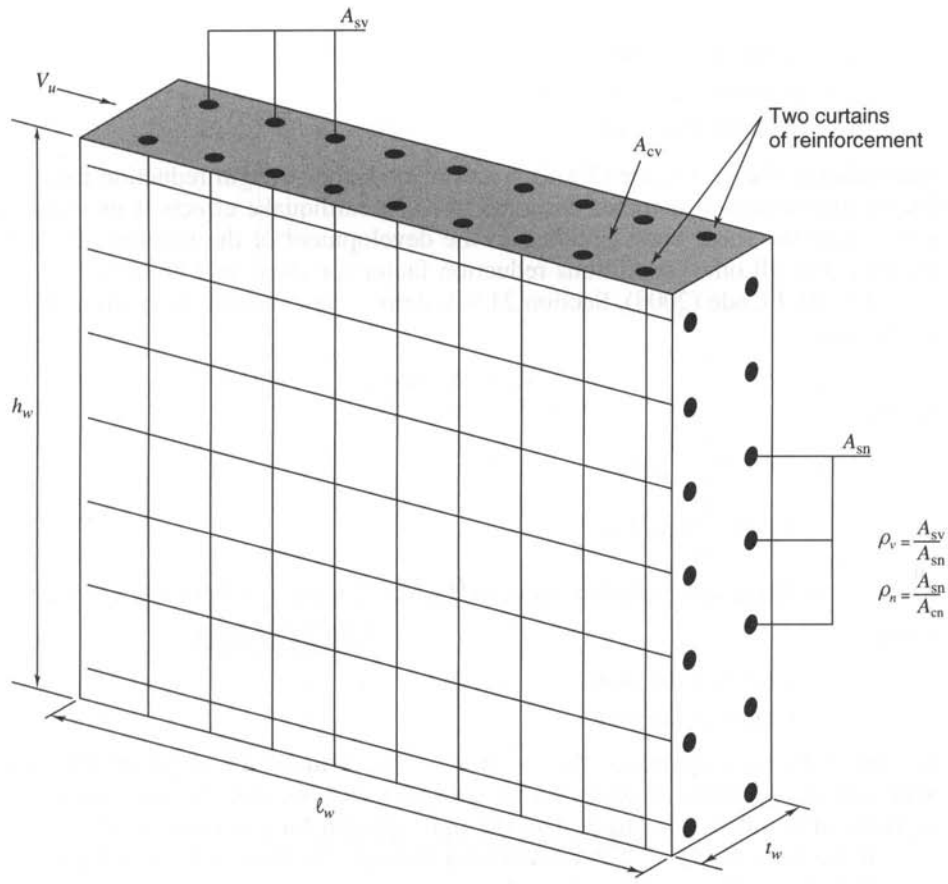


Figure 20.26 Reinforcement requirements.

The spacing of the reinforcement can be calculated as follows:

$$s = 2A_s^1/A_s \text{required (per foot of wall)}$$

where

A_s^1 = area of one bar (Fig. 20.26)

Maximum spacing of reinforcement is 18 in. each way according to ACI Section 21.9.2.1.

If the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\lambda\sqrt{f_c'}$, at least two curtains of reinforcement should be provided, as shown in Figure 20.26.

All continuous reinforcement in structural walls should be anchored and spliced as reinforcement in tension for special moment frame (Section 21.9.2.3).

Shear Strength Requirements (Section 21.9.2.2). The shear strength of structural wall is adequate if the following condition is satisfied:

$$V_u \leq \phi V_n \tag{20.52}$$

where

V_u = factored axial force
V_n = nominal shear strength
ϕ = strength reduction factor

According to the ACI Code (2008), Section 9.3.4, the strength reduction factor for shear will be 0.6 for any structural member designed to resist earthquake effects if its nominal shear strength is less than the shear corresponding to the development of the nominal flexural strength of the member. For all other conditions reduction factor for shear will be 0.75.

The ACI Code (2008), Section 21.9.4, defines the nominal shear strength of structural walls as follows:

$$V_n = A_{cv}(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y) \tag{20.53}$$

where

$$\alpha_c = 3.0 \text{ for} \frac{h_w}{l_w} \leq 1.5$$
$$= 2.0 \text{ for} \frac{h_w}{l_w} \geq 2.0$$
$$= \text{linear interpolation between 3.0 and 2.0 for } \frac{h_w}{l_w} \text{ between 1.5 and 2.0}$$

where

h_w = height of the wall
l_w = length of the wall

For the walls with openings, the value of h_w/l_w shall be the larger of the ratios for the entire wall and the segment of wall considered. This ensures that the assigned unit strength of any segment of a wall is not larger than the unit strength for the whole wall.

If the ratio $h_w/l_w \leq 2$, reinforcement ratio ρ_v should not be less than ρ_n.

For the walls with openings, the nominal shear strength, V_n, for vertical and horizontal walls segments should satisfy the following:

1. If the factored shear force is resisted several pier, the nominal shear strength, V_n, for all wall segments should be $\leq 8A_{cv}\sqrt{f'_c}$, where A_{cv} is the total cross-section area of the walls (piers) and $V_n \leq 10A_{cp}\sqrt{f'_c}$, where A_{cp} is the cross-section area of the pier considered.
2. Nominal shear strength of horizontal wall segment and coupling beams should be $\leq 10A_{cp}\sqrt{f'_c}$, where A_{cp} is the cross-section area of the horizontal wall segment or coupling beam.

Design for flexure and axial loads (Section 20.5.2.3). Flexural strength of walls should be determined according to the procedure used for columns subjected to flexure and axial loads (ACI 2008, Section 21.9.5). The reinforcement in the whole cross-section of the wall, including boundary elements and web, should be included in calculations of the capacity of the wall. Openings in walls should also be considered.

Where the wall sections intersect, they from L-sections, T-sections, or other cross-section shapes of the flanges (as shown in Fig. 20.27), which need to be considered in design. Flange width should be determined as follows:

Effective flange width from the face of the web should extend a distance equal to or smaller than $\frac{1}{2}$ the distance to an adjacent wall web or 25% of the total wall height (Fig. 20.28), (ACI Section 21.9.5.2).

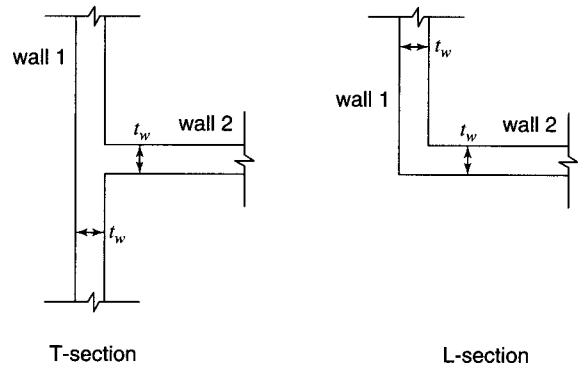


Figure 20.27 Shapes of the wall flanges.

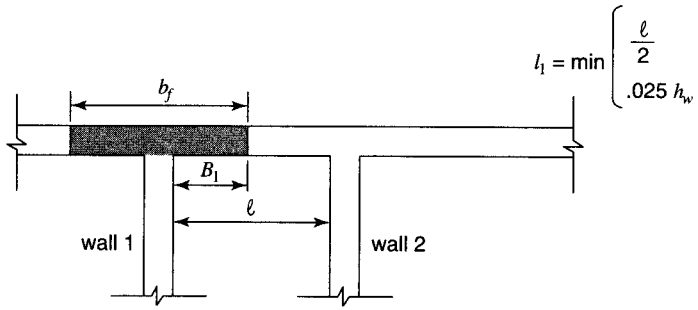


Figure 20.28 Effective flange width, b_f.

Special Boundary Elements of Special Reinforced Structural Walls (Section 20.5.2.4). During an earthquake, a structural wall behaves as cantilever beam (Fig. 20.29). Boundary elements can be very heavily loaded due to earthquake loads. A plastic hinge can form at the base of the wall, which requires special reinforcement detailing to provide necessary strength and ductility of the structural wall. According to the ACI Code (2008), Section 21.9.6.1, there are two design approaches for evaluating the detailing requirements of wall boundary element. These are defined as follows:

1. Displacement based design (ACI Section 21.9.6.2). For the walls or walls pier that are effectively continuous from the base of the structure to the top of the wall, design to have a single critical section for flexure and axial load compression zones should be reinforced with special boundary elements if

$$c \geq \frac{l_w}{600\left(\frac{\delta_u}{h_w}\right)} \tag{20.54}$$

where

$$\frac{\delta_u}{h_w} \geq 0.007$$

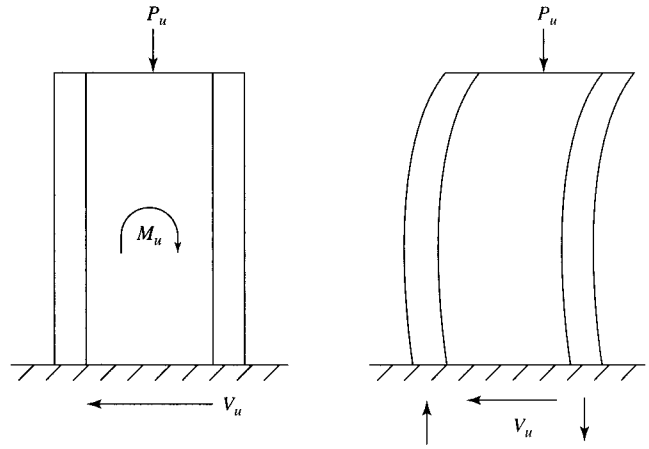


Figure 20.29 Deformation of wall due to earthquake loads.

c = the distance from the extreme compression fiber to the neutral axis, calculated for the factored axial force and nominal moment strength

l_w = the length of the wall in the direction of shear force

δ_u = design displacement

The special boundary reinforcement should extend vertically from a critical section a distance (Fig. 20.30).

$$\geq \begin{cases} l_w \\ \dfrac{M_u}{4V_u} \end{cases} \tag{20.55}$$

2. Shear based design (ACI Section 21.9.6.3). Structural walls not designed to the displacement based approach shall have special boundary elements at boundaries and edges around openings of the structural wall. A special boundary element should be provided where the maximum extreme fiber compressive stress due to factored forces including earthquake effects exceeds $0.2f'_c$. The boundary elements may be discontinued when the compressive stress becomes less than $0.15f'_c$.

Detailing of the special boundary elements should satisfy the following:

1. Extend horizontally from the extreme compression fiber a distance (Fig. 20.30).

$$\geq \begin{cases} c - 0.1l_w \\ \dfrac{c}{2} \end{cases}$$

where

c = the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with δ_u.

2. Transverse reinforcement should be designed by the provisions given for the special moment frame members subjected to bending and axial forces (Fig. 20.31).

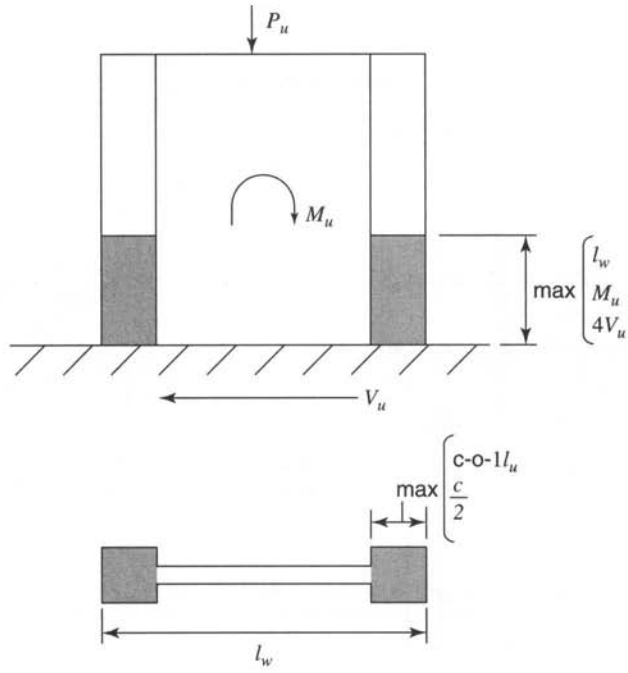


Figure 20.30 Area where special reinforcement requirements should be provided.

Coupling Beams (Section 20.5.2.5). The coupling beam is the structural element that rigidly connects two walls. In a properly designed earthquake-resistant coupled wall system, the coupled beams should yield first, before the base of the wall where the bending moment has the highest value. Also, the beam should have significant ductility and dissipate the energy through the inelastic deformation.

According to the ACI Code, Section 21.9.7, the coupled beams should be designed as follows:

1. If $l_n/h \geq 4$, where l_n is the length and h is the height of the coupled beam, design the coupled beam to satisfy requirements given for flexural members of special moment frame.
2. If $l_n/h < 4$, the beam should be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan. The diagonal bars are also required for coupling beam with aspect ratio $l_n/h < 2$ and $V_u > 4\lambda\sqrt{f'_c}A_{cw}$, where A_{cw} is the area of concrete section, resisting shear, of individual pier or horizontal wall segment.

   Two confinement options are described in ACI 318-08 as shown in Figure 20.32. According to ACI Section 21.9.7.4(c) each diagonal element consists of a cage of longitudinal and transverse reinforcement as shown in Figure 20.32(a). Each cage contains at least four diagonal bars and confines a concrete core. The requirement on side dimensions of the cage and its core is to provide adequate toughness and stability to the core section when the bars are loaded beyond yielding.

   ACI Section 21.9.7.4(d) describes a second option for confinement of the diagonals as shown in Figure 20.32(b). This second option is to confine the entire beam cross section instead of confining the individual diagonals. This option can considerably simplify field

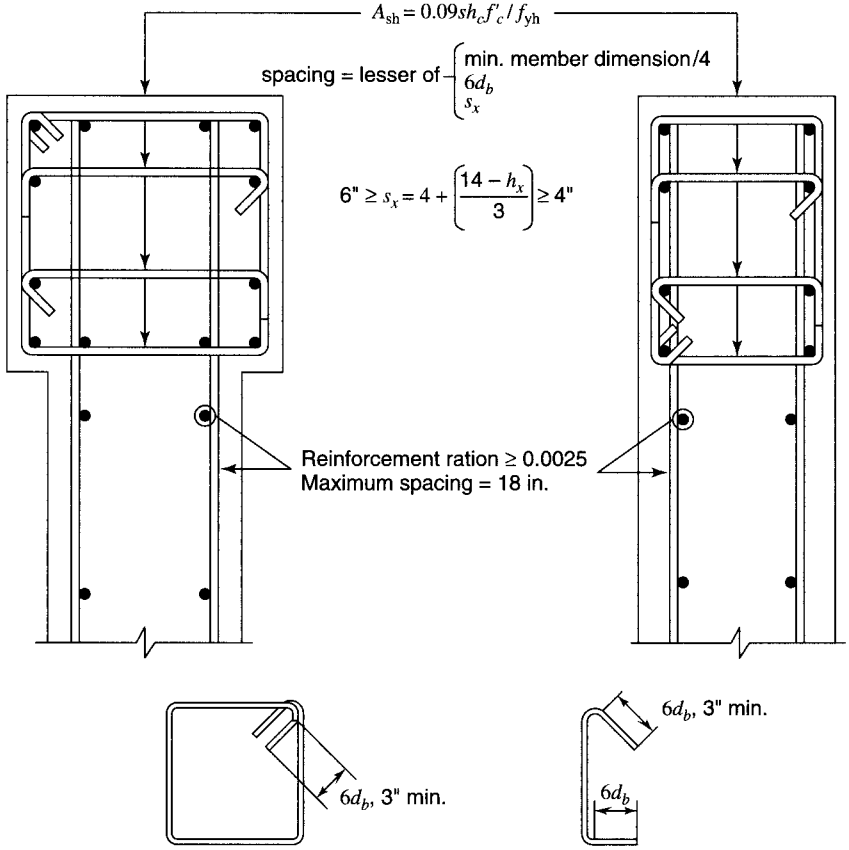


Figure 20.31 Reinforcement details for special boundary elements. Courtesy of Portland Cement Association (notes on ACI 318).

placement for hoops, which can be challenging where diagonal bars intersect each other or entire wall boundary.

Nominal shear strength can be determined using the following equation:

$$V_u = 2A_{vd} f_y \sin\alpha \leq 10\sqrt{f'_c} A_{cw} \tag{20.56}$$

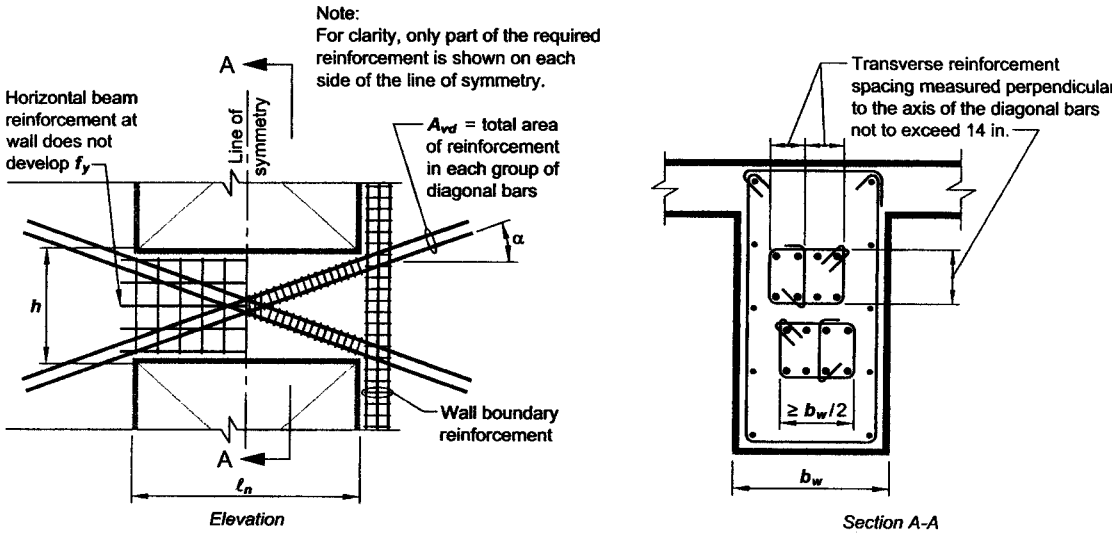
where

A_{vd} = total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam

α = angle between the diagonal reinforcement and the longitudinal axis of a diagonally reinforced coupling beam

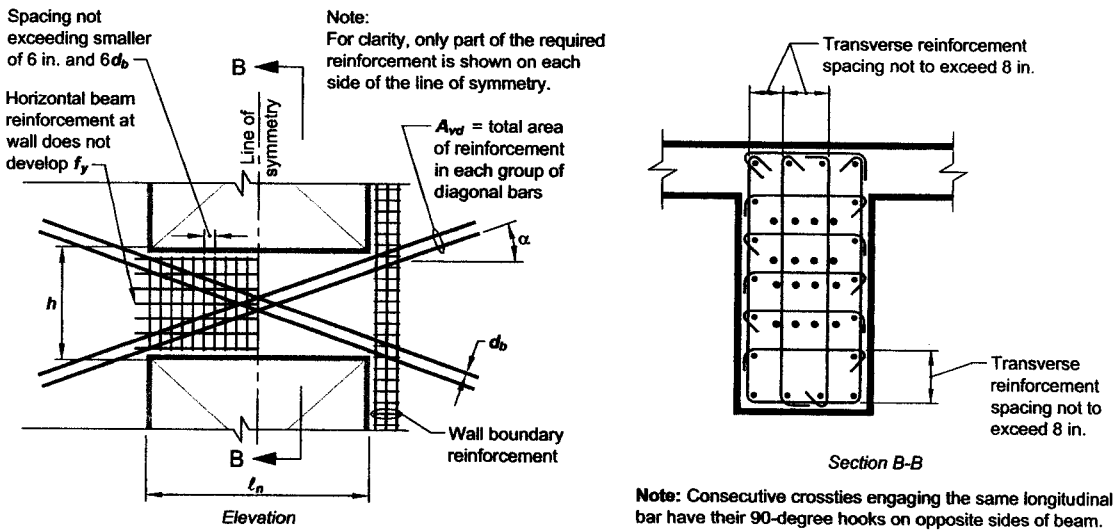
Transverse reinforcement for each group of diagonally placed bars should be designed as transverse reinforcement for the members of a special moment frame subjected to bending and axial force.

Detailing of coupling beam reinforcement should be in accordance with Fig. 20.32.



(a) Confinement of individual diagonals.

Note: For clarity in the elevation view, only part of the total required reinforcement is shown on each side of the line of symmetry.



Note: Consecutive crossties engaging the same longitudinal bar have their 90-degree hooks on opposite sides of beam.

(b) Full confinement of diagonally reinforced concrete beam section.

Figure 20.32 Reinforced detailing for coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity. Courtesy of American Concrete Institute (ACI 2008).

Summary: Design of Special Structural Wall (Section 20.5.2.6).

Step 1. Determine minimum reinforcement ratio according to Section 20.5.2.1 and design horizontal and vertical reinforcement for wall web.

Step 2. Check the shear strength of the wall according to Section 20.5.2.2.

Step 3. Design the wall for flexure and axial force assuming that the wall behaves as a column and include all reinforcement in cross-section of the wall and reinforcement in boundary elements and web in calculations (Section 20.5.2.3).

Step 4. Check whether the boundary elements need to be specially detailed according to Section 20.5.2.4. If conditions are satisfied, design the transverse reinforcement of boundary elements by the provisions given for the special moment frame members subjected to bending and axial forces.

Step 5. Design the coupling beams as shown in Section 20.5.2.5

Example 20.8

Design the wall section given in Fig. 20.33 as a special structural wall.

Given: Forces are $Pu = 4000$ kip, $Mu = 45{,}000$ kip-ft, $Vu = 900$ kip; boundary elements are 24 × 24 in. columns; wall web thickness is 16 in.; wall length is 28 ft; wall height is 12 ft; normal-weight concrete with $f'_c = 4000$ psi; normal-weight concrete, and $f_y = 60{,}000$ psi. Boundary elements are reinforced with 16 no. 11 bars.

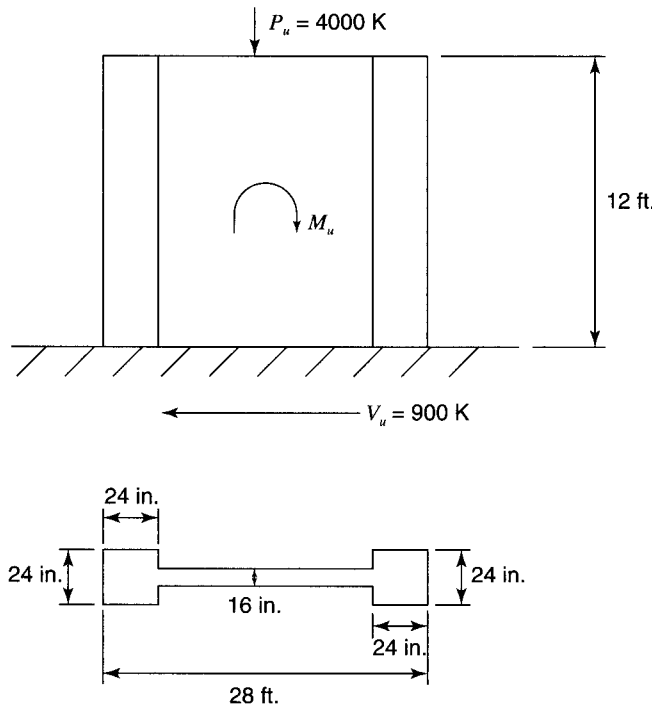


Figure 20.33 Example 20.8: structural wall.

Solution

1. Reinforcement requirements. To determine minimum reinforcement ratio check whether $V_u > A_{cv}\lambda\sqrt{f'_c}$

$$A_{cv} = 16 \times (28 \times 12) = 5376 \text{ in.}^2$$

$$A_{cv}\lambda\sqrt{f'_c} = 5376 \times 1 \times \sqrt{4000}/1000 = 340 \text{ kip} < V_u = 900 \text{ kip}$$

$$\Rightarrow \min \rho_l = \frac{A_{sv}}{A_{cv}} = \rho_n = 0.0025$$

Minimum reinforcement in both directions, longitudinal and transverse, per foot of wall can be determined as follows:

$$A_{cv} = 16 \times 12 = 192 \text{ in.}^2 \text{ per foot of wall}$$

$$A_s = 0.0025 \times 192 = 0.48 \text{ in.}^2/\text{ft}$$

Check whether two curtains of reinforcement are needed:

$$2A_{cv}\lambda\sqrt{f'_c} = 2 \times 1 \times 340 = 680 \text{ kip} < Vu = 900 \text{ kip}$$

Two curtains of reinforcement are required.

Choose no. 5 bars:

$$A_s = 2 \times (0.31) = 0.62 \text{ in.}^2$$

$$\text{Spacing}(s) = \frac{0.62}{0.48} \times 12 = 15.5 \text{ in} < 18 \text{ in.}$$

Choose $s = 15$ in. (See Fig. 20.34.)

2. Shear strength requirements. Check whether the two curtains of no. 5 bars spaced 15 in. on center can sustain applied shear force at the base. For $h_w/l_w = 12/28 = 0.43 < 1.5$,

$$\alpha_c = 3.0$$

$$\rho_t = \frac{0.62}{16 \times 15} = 0.0026$$

$$\phi V_n = \phi A_{cv}(\alpha_c\lambda\sqrt{f'_c} + \rho_t f_y)$$

$$= 0.75 \times 5376(3 \times 1 \times \sqrt{4000} + 0.0026 \times 60000)/1000 = 1394 \text{ kip} > 900 \text{ kip}$$

Two curtains of no. 5 bars spaced 15 in. center-to-center can sustain applied shear force at the base.

3. Design for flexure and axial force. Wall is designed as column subjected to axial load and bending.

$$Pu = 4000 \text{ kip}$$

$$Mu = 45{,}000 \text{ kip}$$

$$e = \frac{Mu}{Pu} = \frac{45,000}{4000} \times 12 = 135 \text{ in.}$$

$$M_n = \frac{M_u}{\phi} = \frac{45,000}{0.65} = 69{,}230 \text{ kip-ft}$$

$$Pn = \frac{Pu}{\phi} = \frac{4000}{0.65} = 6153 \text{ kip}$$

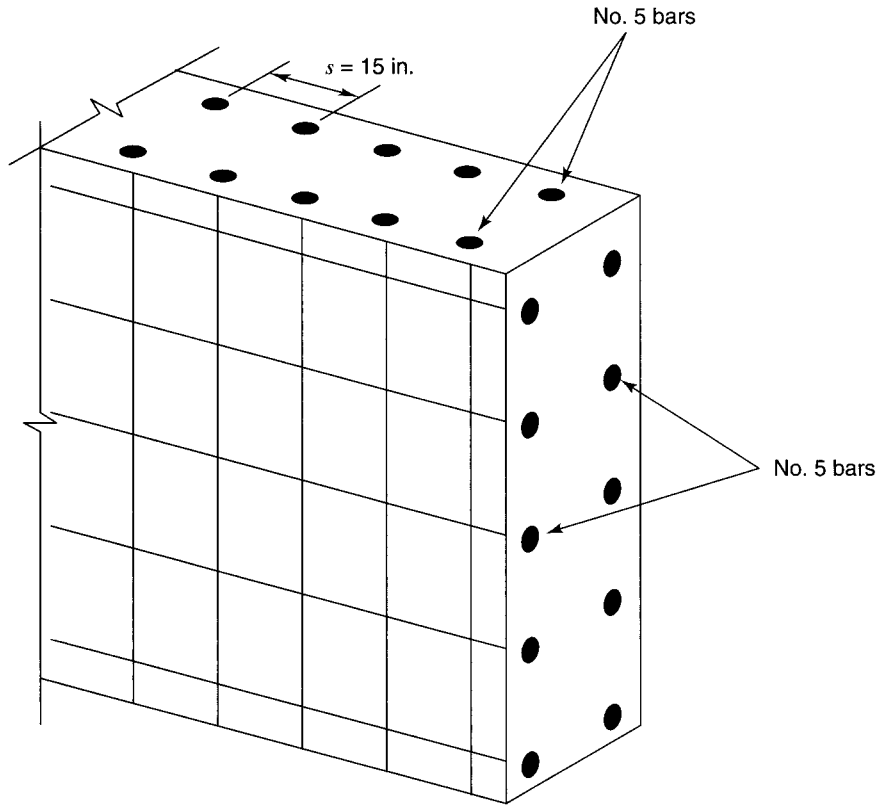


Figure 20.34 Example 20.8: reinforcement detailing of wall web.

Total area of reinforcement consists of 32 no. 11 bars in boundary elements and 40 no. 5 bars in the web.

$$A_s = 32 \times 1.56 + 40 \times 0.31 = 62.3 \text{ in.}^2$$

$$A_g = 5760 \text{ in.}^2$$

$$\rho = \frac{62.3}{5760} = 0.0109 > 0.01 \quad \text{and} \quad < 0.06 \text{ (o.k.)}$$

$$\frac{P_n}{f'_c A_g} = \frac{6{,}153{,}000}{4000 \times 5760} = 0.267$$

From the interaction diagram,

$$\frac{M_n}{f'_c A_g h} = 0.162$$

$$M_n = 0.162 \times 4000 \times 5760 \times 28 \times 12 = 104{,}509 \text{ kip-ft} > 69{,}230 \text{ kip-ft (o.k.)}$$

4. Special boundary elements requirements. The shear-based approach is used to determine whether the special boundary elements are required.

$$A_g = 5376 \text{ in.}^2$$

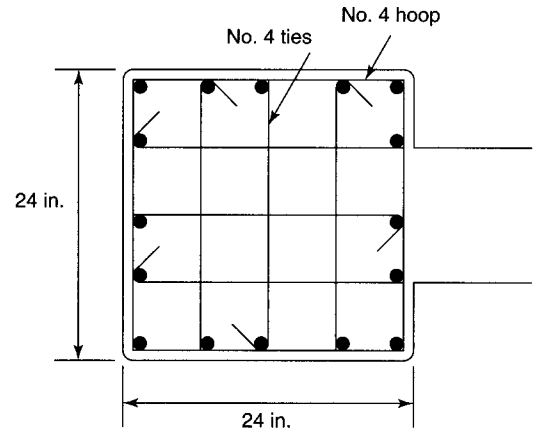


Figure 20.35 Example 20.8: boundary element reinforcement.

$$I_g = \frac{16 \times (28 \times 12)^3}{12} = 50{,}577{,}408 \text{ in.}^4$$

$$c = \frac{28 \times 12}{2} = 168 \text{ in.}$$

Maximum compressive stress in the wall

$$= \frac{P_u}{A_g} + \frac{M_u\, c}{I_g} = \frac{4{,}000{,}000}{5376} + \frac{45{,}000{,}000 \times 12}{50{,}577{,}408} 168 = 2538 \text{ psi}$$

$$0.2 f'_c = 0.2 \times 4000 = 800 \text{ psi} < 2538 \text{ psi}$$

A special boundary element is needed. Transverse reinforcement of boundary element should be designed as for members of special moment frame subjected to axial load and bending (Fig. 20.35).

Use no. 4 hoops and crossties around longitudinal bars in both directions. Maximum spacing of transverse reinforcement should be determined as follows:

$$s_{\max} = \begin{cases} 0.25 \times (\text{smallest member dimension}) = 0.25 \times 24 = 6 \text{ in.} \\ 6 \times (\text{diameter of longitudinal bar}) = 6 \times 1.41 = 8.5 \text{ in.} \\ s_x = 4 + \left(\dfrac{14 - h_x}{3}\right) = 4 + \left(\dfrac{14 - 6}{3}\right) = 6.7 > 6 \end{cases}$$

Use $s = 6$ in. (governs).

Required cross-section area:

$$A_{sh} = \frac{0.09 s h_c f'_c}{f_y} = \frac{0.09 \times 6 \times [24 - (2 \times 1.5) - 0.5] \times 4}{60} = 0.738 \text{ in.}^2$$

$$A_{sh} = 0.3 s h_c \left[\frac{A_g}{A_{ch}} - 1\right] \frac{f'_c}{f_{yh}} = 0.3 \times 6 \times 20.5 \left(\frac{576}{420.25} - 1\right) \frac{4}{60} = 0.911 \text{ in.}^2 \quad \text{(governs)}$$

No. 4 hoops with crossties around every longitudinal bar provide

$$A_{sh} = 5 \times 0.2 = 1.0 \text{ in.}^2 > 0.911 \text{ in.}^2 \quad \text{(o.k.)}$$

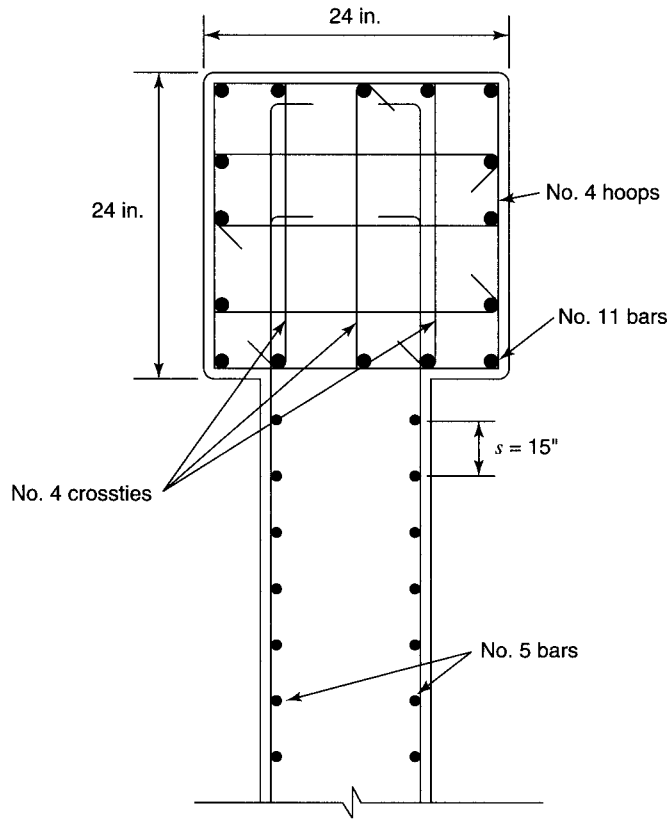


Figure 20.36 Example 20.8: reinforcement detailing.

Development length of no. 5 bars assuming that the hooks are used (ACI Section 21.7.5.1) is

$$l_{\text{dh}} \geq \begin{cases} \dfrac{f_y d_b}{65\sqrt{f_c'}} = \dfrac{60000 \times 0.625}{65\sqrt{4000}} = 9 \text{ in.} \\ 8d_b = 8 \times 0.625 = 5 \text{ in.} \\ 6 \text{ in.} \end{cases}$$

Therefore, $l_{dh} = 9$ in. $l_d = 3.5 l_{dh} = 3.5 \times 9 = 32$ in. > the dimension of boundary element = 24 in. Use the hooks to anchor reinforcement (Fig. 20.36).

20.5.3 Structures in the Areas of Moderate Seismic Risk: Intermediate Moment Frames (ACI 2008, Section 21.3)

In regions of moderate seismic risk (SDC C), the moment frames should be designed as intermediate moment frames. The ACI Code, ACI Section 21.1.1.5, gives provisions for the design and detailing of intermediate moment frames as follows:

Longitudinal Reinforcement Requirements (Section 20.5.3.1). If the compressive axial load for the member $< A_g f_c'/10$, the member is considered to be subjected only to bending and the following is applicable (ACI 2008, Section 21.3.2):

Positive moment strength at joint face $\geq \frac{1}{3}$ negative moment strength at that face of the joint.

$$M_{\text{nl}}^{+} \geq \frac{1}{3} M_{\text{nl}}^{-} \qquad \text{(left joint)} \tag{20.57a}$$

$$M_{\text{nr}}^{+} \geq \frac{1}{3} M_{\text{nr}}^{-} \qquad \text{(right joint)} \tag{20.57b}$$

Neither the positive nor the negative moment strength at any section along the length of the member should be less than $\frac{1}{5}$ the maximum moment strength provided at the face of either joint.

$$(\phi M_n^{+} \text{ or } \phi M_n^{-}) \geq \frac{1}{5} \max(\phi M_n \text{ at either joint}) \tag{20.58}$$

Transverse Reinforcement Requirements (Section 20.5.3.2).

Beams. It is assumed that the plastic hinges will form at the end of the beams. According to this, the beam ends should be specially detailed to provide the beam with necessary ductility.

Hoops should also be provided over a length equal to $2d$ (d is the effective depth of the beam) measured from the face of support towards midspan. The first hoop should be located at distance ≤ 2 in. from the face of support.

Maximum spacing of transverse reinforcement should not exceed the smallest of:

$$S_{\max} \leq \begin{cases} \dfrac{d}{4} \\ 8 \times (\text{diameters of the smallest longitudinal bar enclosed}) \\ 24 \times (\text{diameter of the hoop bar}) \\ 12 \text{ in.} \end{cases} \tag{20.59}$$

When hoops are not required, stirrups should be used. Spacing of stirrups should be $\leq d/2$ through the length of the member (ACI Section 21.3.4.3).

Columns. Transverse reinforcement of columns of intermediate moment frame should be designed with spiral reinforcement or with hoops and stirrups as follows: Spiral reinforcement should satisfy requirements for ordinary compression member (ACI 7.10.4): Hoops should be provided at both ends of the member over a length l_o measured from the face of the joint, spaced a distance s_o. (ACI Section 21.3.5.2). Spacing S_o shall not exceed the smallest of the four items listed below or

$$s_o \leq \begin{cases} 8 \times (\text{diameter of the smallest longitudinal bar}) \\ 24 \times (\text{diameter of the hoop bar}) \\ \dfrac{1}{2} \text{of the smallest cross-section dimension of the member } 12 \end{cases} \tag{20.60}$$

Length l_o shall not be less than the largest of the three items listed below or

$$l_o \geq \begin{cases} \dfrac{1}{6} \text{ of the clear length of the member} \\ \text{Maximum cross-section dimension of the member} \\ 18 \text{ in.} \end{cases} \tag{20.61}$$

The first hoop should be located at distance $\leq s_o/2$ from the joint face. Outside the length l_o, spacing s_o should confirm to ACI Section 7.10 and ACI Section 11.4.5.1 or

$$s_o \leq \begin{cases} \dfrac{d}{2} \\ 24 \text{ in.} \end{cases} \tag{20.62}$$

CODE AND DESIGN REFERENCES

1. American Concrete Institute. "Building Code Requirements for Structural Concrete." ACI 318-08 and Commentary ACI 318R-08. Farmington Hills, Michigan, 2008.
2. International Code Council. "International Building Code 2006." Country Club Hills, Illinois.
3. S. K. Ghosh, and A. David, Fanella "Seismic and Wind Design of Concrete Buildings" 2000 IBC, ASCE 7-98, ACI 318-99. Country Club Hills, Illiois, 2003 International Code Council.
4. S. K., Ghosh August W. Domel Jr., and A. David Fanella. *Design of Concrete Buildings for Earthquake and Wind Forces*, 2d ed., Skokie, Illinois, Portland Cement Association, 1995.
5. Portland Cement Association. "Notes on ACI 318-02 Building Code Requirements for Structural Concrete with Design Applications." Skokie, Illinois, 2002.
6. ASCE 7-05, "Minimum Design Loads for Buildings and other Structures." American Society of Civil Engineering, 2005.

PROBLEMS

20.1 Determine seismic design category for a five-story building in the area of northern California if the soil is hard rock.

20.2 Determine base shear for a two-story building located in the area of high seismic risk where $S_S = 1.3\,\text{g}$ and $S_1 = 0.6\,\text{g}$, on soil class B. Assume that the idealized weight of the first floor is 50 kip and of the second floor is 60 kip.

20.3 Determine lateral seismic forces for the five-story building assuming that the idealized mass of each floor is 1000 kip. Consider the structure a building occupancy category III, site class C.

20.4 Design the longitudinal reinforcement for the beam on the second floor of a special-moment frame four-story building assuming the clear span of a beam is 24 feet. Each story height is 12 feet. Beam dimensions are 20 × 24 in., and the column is 24 × 24 in. Bending moments acting on the beam are given in the following table.

Load	Location	Bending Moment (kip-ft)
Dead	Support	−70
	Midspan	45
Live	Support	25
	Midspan	18
Earthquake	Support	±180
	Midspan	0

20.5 Design the transverse reinforcement for the beam of special moment-resisting frame. The beam is reinforced with five no. 8 bars and is 24 × 30 in. Load acting on the beam is W_D = 3.0 kip/ft, W_L = 1.5 kip/ft, and clear span is 24 ft.

20.6 Design the reinforcement for a column on the first floor of four-story building following the provisions for special moment-resisting frame reinforcement. The column is 30 × 30 in. and 12 ft high. Nominal flexural strength of the beam framing into the column M_n = 650 k · ft. Axial load acting on the on the second-floor column is P_u = 1920 kip, axial load acting on the first-floor is P_u = 2000 kip, and minimum axial load in load combination is 1010 kip. The shear force is V_u = 120 kip. Draw the detail of reinforcement.

20.7 Design the reinforcement for a wall having a total height of 28 ft and span of 35 ft. Total gravity load acting on the wall is 5200 kip, factored moment (M_u) = 50,000 kip-ft, and base shear is V = 1000 kip. Wall thickness is 20 in. and boundary elements are 25 × 25 in.

CHAPTER 21

BEAMS CURVED IN PLAN



Curved beams in an office building.

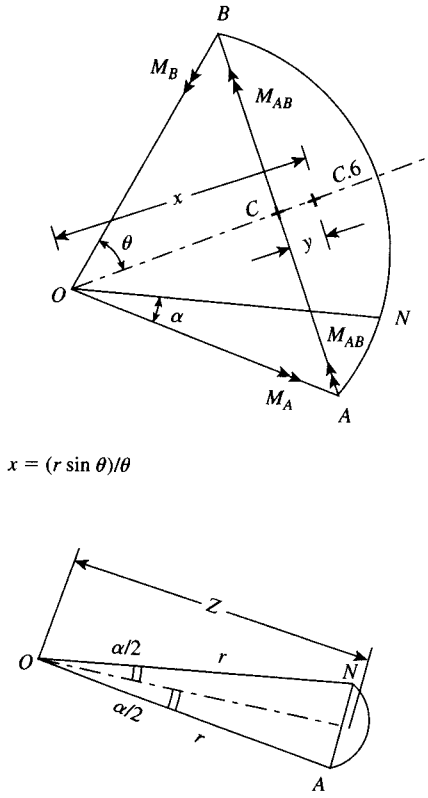
21.1 INTRODUCTION

Beams curved in plan are used to support curved floors in buildings, balconies, curved ramps and halls, circular reservoirs, and similar structures. In a curved beam, the center of gravity of the loads acting normal to the plane of curvature lies outside the line joining its supports. This situation develops torsional moments in the beam, in addition to bending moments and shearing forces. To maintain the stability of the beam against overturning, the supports must be fixed or continuous. In this chapter, the design of curved beams subjected to loads normal to the plane of curvature is presented. Analysis of curved beams subjected to loads in the plane of curvature is usually discussed in books dealing with mechanics of solids.

Analysis of beams curved in plan was discussed by Wilson [1]. He introduced formulas and coefficients to compute stresses in curved flexural members. Timoshinko [2], [3] also introduced several expressions for calculating bending stresses in square and rectangular sections. Tables and formulas for the calculation of bending and torsional moments, shear, and deflections for different cases of loadings on curved beams and rings are presented by Roark and Young [4].

21.2 UNIFORMLY LOADED CIRCULAR BEAMS

The first case to be considered here is that of a circular beam supported on columns placed at equal distances along the circumference of the beam and subjected to normal loads. Due to



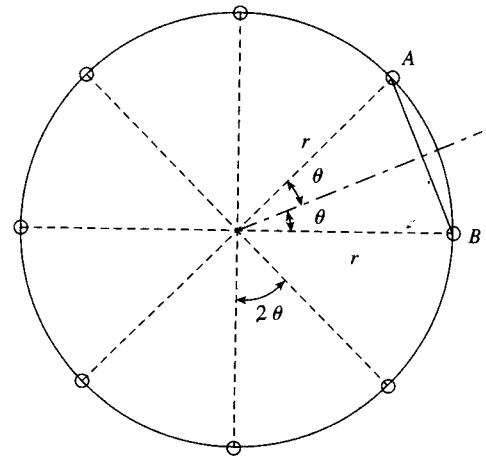


Figure 21.1 Circular Beam.

symmetry, the column reactions will be equal, and each reaction will be equal to the total load on the beam divided by the number of columns. Referring to Fig. 21.1, consider the part AB between two consecutive columns of the ring beam. The length of the curve AB is $r(2\theta)$, and the total load on each column is $P_u = w_u r(2\theta)$, where r is the radius of the ring beam and w_u is the factored load on the beam per unit length. The center of gravity of the load on AB lies at a distance

$$x = \left(\frac{r \sin\theta}{\theta}\right)$$

from the center O. The moment of the load P_u about AB is

$$M_{AB} = P_u \times y = P_u(x - r \cos\theta) = w_u r(2\theta)\left(\frac{r \sin\theta}{\theta} - r \cos\theta\right)$$

Consequently, the two reaction moments, M_A and M_B, are developed at supports A and B, respectively. The component of the moment at support A about AB is $M_A \sin\theta = M_B \sin\theta$. Equating the applied moment, M_{AB}, to the reaction moments components at A and B,

$$2M_A \sin\theta = M_{AB} = w_u r(2\theta)\left(\frac{r \sin\theta}{\theta} - r \cos\theta\right) \tag{21.1}$$

$$M_A = M_B = w_u r^2(1 - \theta \cot\theta)$$

The shearing force at support A is

$$V_A = \frac{P_u}{2} = w_u r\theta \tag{21.2}$$

The shearing force at any point N, V_N, is $V_A - w_u$ (rα), or

$$V_N = w_u r(\theta - a) \tag{21.3}$$

The load on AN is w_u (rα) and acts at a distance equal to

$$Z = \frac{r \sin\alpha/2}{\alpha/2}$$

from the center 0. The bending moment at point N on curve AB is equal to the moment of all forces on one side of o about the radial axis ON.

$$M_N = V_A(r \sin\alpha) - M_A \cos\alpha - (\text{load onthe curve } AN)\left(Z \sin\frac{\alpha}{2}\right)$$

$$M_N = w_u r\theta(r \sin\alpha) - w_u r^2(1 - \theta \cot\theta)\cos\alpha$$

$$- (w_u r\alpha)\left(\frac{r \sin\alpha/2}{\alpha/2} \times \sin\frac{\alpha}{2}\right)$$

$$= w_u r^2\left[\theta \sin\alpha - \cos\alpha + (\theta \cot\theta \cos\alpha) - 2 \sin^2\frac{\alpha}{2}\right]$$

$$M_N = w_u r^2[\theta \sin\alpha + (\theta \cot\theta \sin\alpha) - 1] \tag{21.4}$$

(Note that $\cos\alpha = 1 - 2\sin^2\alpha/2$.) The torsional moment at any point N on curve AB is equal to the moment of all forces on one side of N about the tangential axis at N.

$$T_N = M_A \sin\alpha - V_A \times r(1 - \cos\alpha) + w_r r\alpha\left(r - \frac{r \sin\alpha/2}{\alpha/2} \times \cos\alpha/2\right)$$

$$= w_u r^2(1 - \theta \cot\theta)\sin\alpha - w_u r^2\theta(1 - \cos\alpha) + w_u r^2(\alpha - \sin\alpha)$$

$$T_n = w_u r^2(\alpha - \theta + \theta \cos\alpha - \theta \cot\theta \sin\alpha) \tag{21.5}$$

To obtain the maximum value of the torsional moment T_N, differentiate Eq. 21.5 with respect to α and equate it to 0. This step will give the value of α for maximum T_N.

$$\sin\alpha = \frac{1}{\theta}\left[\sin^2\theta \pm \cos\theta\sqrt{\theta^2 - \sin^2\theta}\right] \tag{21.6}$$

The values of the support moment, midspan moment, the torsional moment, and its angle α from the support can be calculated from Eqs. 21.1 through 21.6. Once the number of supports n is chosen, the angle θ is known,

$$2\theta = \frac{2\pi}{n} \quad \text{and} \quad \theta = \frac{\pi}{n}$$

and the moment coefficients can be calculated as shown in Table 21.1. Note that the angle α is half the central angle between two consecutive columns.

$$\text{Load on each column is } P_u = w_u r(2\theta) = w_u r\left(\frac{2\pi}{n}\right)$$

$$\text{Maximum shearing force is } V_u = \frac{P_u}{2}$$

Table 21.1 Force Coefficients of Circular Beams

Number of Supports, n	$\theta = \frac{\pi}{n}$	K_1	K_2	K_3	$\alpha°$ for T_u (max)
4	90	0.215	0.110	0.0330	19.25
5	72	0.136	0.068	0.0176	15.25
6	60	0.093	0.047	0.0094	12.75
8	45	0.052	0.026	0.0040	9.50
9	40	0.042	0.021	0.0029	8.50
10	36	0.034	0.017	0.0019	7.50
12	30	0.024	0.012	0.0012	6.25

$$\text{Negative moment at any support} = K_1 w_u r^2 \tag{21.7}$$

$$\text{Positive moment at midspan} = K_2 w_u r^2 \tag{21.8}$$

$$\text{Maximum torsional moment} = K_3 w_u r^2 \tag{21.9}$$

The variation of the shearing force and bending and torsional moments along a typical curved beam AB are shown in Fig. 21.2.

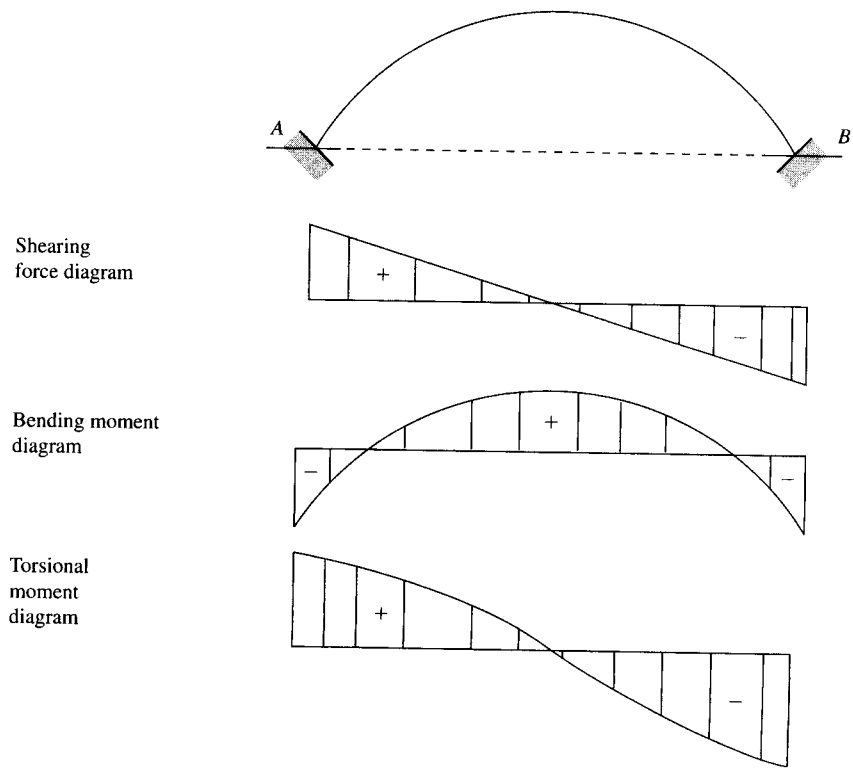


Figure 21.2 Forces in a circular beam.

Example 21.1

Design a circular beam supported on eight equally spaced columns. The centerline of the columns lies on a 40-ft-diameter circle. The beam carries a uniform dead load of 6 K/ft and a live load of 4 K/ft. Use normal-weight concrete with $f'_c = 5$ Ksi, $f_y = 60$ Ksi, and b = 14 in.

Solution

1. Assume a beam size of 14 × 24 in. The weight of the beam is

$$\frac{14 \times 24}{12 \times 12}(0.150) = 0.35 \text{ K/ft}$$

The factored uniform load is $w_u = 1.2(6 + 0.35) + 1.6(5) = 15.7$ K/ft.

2. Because the beam is symmetrically supported on eight columns, the moments can be calculated by using Eqs. 21.7 through 21.9 and Table 21.1 Negative moment at any support is $K_1 \, w_u \, r^2 = 0.052(15.7)(20)^2 = 326.6$ K·ft. The positive moment at midspan is $K_2 \, w_u \, r^2 = 0.216(15.7)(20)^2 = 163.3$ K·ft. The maximum torsional moment is $K_3 \, w_u \, r^2 = 0.004(15.7)(20)^2 = 25.12$ K·ft. Maximum shear is

$$V_u = \frac{P_u}{2} = \frac{w_u r}{2}\left(\frac{2\pi}{n}\right) = (15.7)(20)\left(\frac{\pi}{8}\right) = 123.3 \text{ K}$$

3. For the section at support, $M_u = 326.6$ K ft. Let $d = 21.5$ in.; then

$$R_u = \frac{M_u}{bd^2} = \frac{326.6 \times 12{,}000}{14(21.5)^2} = 605 \text{ psi}$$

For $f'_c = 4$ Ksi and $f_y = 60$ Ksi, $\rho = 0.0126 < \rho_{max} = 0.018$, $\phi = 0.9$

$$A_s = 0.0126 \times 14 \times 21.5 = 3.8 \text{ in.}^2$$

4. For the section at midspan, $M_u = 163.3$ K·ft.

$$R_u = \frac{163.3 \times 12{,}000}{14(21.5)^2} = 303 \text{ psi}$$

$$\rho = 0.006 \quad \text{and} \quad A_s = 0.006 \times 14 \times 21.5 = 1.81 \text{ in.}^2$$

Use two no. 9 bars.

5. Maximum torsional moment is $T_u = 25.12$ K·ft, and it occurs at an angle $\alpha = 9.5°$ from the support (Table 21.1). Shear at the point of maximum torsional moment is equal to the shear at the support minus $w_u \, r\alpha$.

$$V_u = 123.3 - 15.7(20)\left(\frac{9.5}{180} \times \pi\right) = 71.24 \text{ K}$$

The procedure for calculation of the shear and torsional reinforcement for $T_u = 25.12$ K·ft and $V_u = 71.24$ K is similar to Example 15.2

a. Shear reinforcement is required when $V_u > \phi \, V_c/2$.

$$\phi V_c = 2\phi\lambda\sqrt{f_c}bd = 2(0.75)(1.0)\sqrt{4000}(14 \times 21.5) = 28.6 \text{ K}$$

since $\phi V_c/2 = 14.3 \text{ K} < V_u = 71.24 \text{ K}$

Shear reinforcement is required.

b. Torsional reinforcement is required when

$$T_u > T_a = \phi\lambda\sqrt{f'_c}\left(\frac{A_{cP}^2}{P_{cp}}\right)$$

$$A_{cp} = x_0 y_0 = 14 \times 24 = 336 \text{ in.}^2$$

$$P_{cp} = 2(x_0 + y_0) = 2(14 + 24) = 76 \text{ in.}$$

$$T_a = 0.75 \times 1 \times \sqrt{4000}\left(\frac{336^2}{76}\right) = 70.5 \text{ K·in.}$$

$$\text{since } T_u = 25.12 \text{ K·ft} = 301.4 \text{ K·in.} > T_a$$

Therefore torsional reinforcement is required.

c. Design for shear:

i. $V_u = \phi V_c + \phi V_s$ and $\phi V_c = 28.6\,\text{K}.71.24 = 28.6 + 0.75V_s$, so $V_s = 56.8\,\text{K}$

ii. Maximum $V_s = 8\sqrt{f'_c}bd = 8\sqrt{4000}(14 \times 21.5) = 152.3 \text{ K} > V_u$

iii. $A_v/S = V_s/f_y d = 56.8/(60 \times 21.5) = 0.044 \text{ in.}^2/\text{in.}$ (2 legs)
$A_v/2S = 0.022 \text{ in.}^2/\text{in.}$(one leg)

d. Design for torsion:

i. Choose no. 4 stirrups and a 1.5-in. concrete cover:

$$x_1 = 14 - 3.5 = 10.5 \text{ in.}, \quad y_1 = 24 - 3.5 = 20.5 \text{ in.}$$

$$A_{oh} = x_1 y_1 = 10.5(20.5) = 215.25 \text{ in.}^2$$

$$A_o = 0.85A_{oh} = 183 \text{ in.}^2$$

$$p_h = 2(x_1 + y_1) = 2(10.5 + 20.5) = 62 \text{ in.}$$

For $\theta = 45°$, $\cot\theta = 1.0$.

ii. Check the adequacy of the size of the section using Eq. 15.21:

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u p_h}{1.7A_{oh}^2}\right)^2} \le \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f'_c}\right)$$

$$\phi V_c = 28.6\text{K}, \qquad V_c = 38.12 \text{ K}$$

$$\text{Left-hand side} = \sqrt{\left(\frac{71{,}240}{14 \times 21.5}\right)^2 + \left[\frac{301{,}400 \times 62}{1.7(215.25)^2}\right]^2} = 335 \text{ psi}$$

$$\text{Right-hand side} = 0.75\left(\frac{38{,}120}{14 \times 21.5} + 8\sqrt{4000}\right) = 558 \text{ psi} > 335 \text{ psi}$$

The section is adequate.

iii. Determine the required closed stirrups due to T_u from:

$$\frac{A_t}{S} = \frac{T_n}{2A_o f_y \cot\theta}, \; T_n = T_n/\phi, \; \phi = 0.75, \; \cot\theta = 1.0$$

$$= \frac{301.4}{0.75 \times 2 \times 183 \times 60} = 0.0183 \text{ in.}^2/\text{in.} \quad \text{(one leg)}$$

iv. The total area of one leg stirrup is $0.022 + 0.0183 = 0.04\,\text{in.}^2/\text{in.}$ For no. 4 stirrups, area of one leg $= 0.2\,\text{in.}^2$ Spacing of closed stirrups is $0.2/0.04 = 5.0\,\text{in.}$, say, 5.5 in.

$$\text{Minimum } S = \frac{p_h}{8} = \frac{62}{8} = 7.75 \text{ in.} > 5.0 \text{ in.}$$

$$\text{Minimum } \frac{A_{vt}}{S} = \frac{50 b_w}{f_y} = \frac{50(14)}{60{,}000} = 0.0117 \text{ in.}^2/\text{in.}$$

This is less than the A_t/s provided. Use no. 4 closed stirrups spaced at 5.5 in.

e. Longitudinal bars A_l equal $(A_t/s)\ p_h\ (f_{yv}/f_{yl})\ \cot^2\theta$ (Eq. 15.27).

$$A_l = 0.018(62)\left(\frac{60}{60}\right) = 1.13 \text{ in.}^2$$

$$\text{Min. } A_l = \frac{5\sqrt{f'_c}A_{cp}}{f_{yl}} - \left(\frac{A_t}{S}\right) p_h \left(\frac{f_{yv}}{f_{yl}}\right)$$

$$= \frac{(5\sqrt{4000})(336)}{60{,}000} - 0.018(62)\left(\frac{60}{60}\right) = 0.64 \text{ in.}^2 < 1.0$$

Use $A_l = 1.13\,\text{in.}^2$, with one-third at the top, one-third at middepth, and one-third at the bottom, or $0.33\,\text{in.}^2$ in each location. For the section at the support, $A_s = 3.8\,\text{in.}^2 + 0.38 = 4.18\,\text{in.}^2$ Choose two no. 10 and two no. 9 bars ($A_s = 4.53\,\text{in.}^2$) as top bars. At middepth, use two no. 4 bars ($A_s = 0.4\,\text{in.}^2$). Extend two no. 9 bars of the midspan section to the support. At middepth use two no. 4 bars ($A = 0.4\,\text{in.}^2$). Details of the section are shown in Fig. 21.3.



Circular beams in an office building.

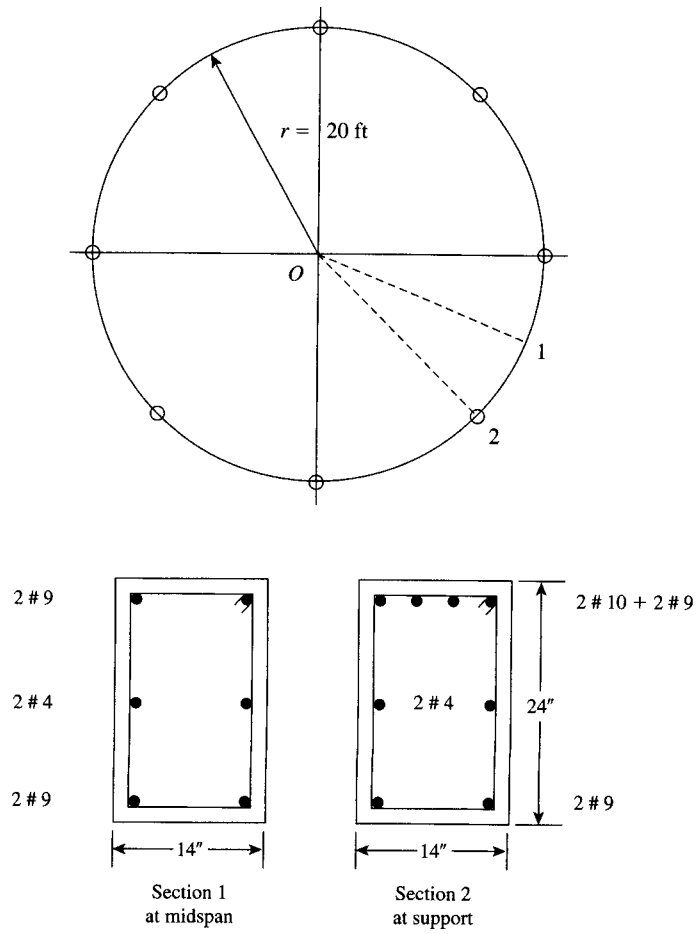


Figure 21.3 Example 21.1.

21.3 SEMICIRCULAR BEAM FIXED AT END SUPPORTS

If a semicircular beam supports a concrete slab, as shown in Fig. 21.4, the ratio of the length to the width of the slab is $2r/r = 2$, and the slab is considered a one-way slab. The beam will be subjected to a distributed load, which causes torsional moments in addition to the bending moments and shearing forces. The structural analysis of the curved beam can be performed in steps as follows.

1. Load on beam: The load on the curved beam will be proportional to its distance from the support AB. If the uniform load on the slab equals w psf, the load on the curved beam at any section N is equal to half the load on the area $NCDE$ (Fig. 21.4). The lengths are $CN = r \sin\theta$, $OC = r \cos\theta$, and $CD = (d/d\theta)(r \cos\theta) = (r \sin\theta\, d\theta)$, and the arc NE is $r\, d\theta$. The load on the curved beam per unit length is equal to

$$w' = \frac{w(r \sin\theta) r \sin\theta\, d\theta}{2(r\, d\theta)} = \frac{wr \sin^2\theta}{2} \tag{21.10}$$

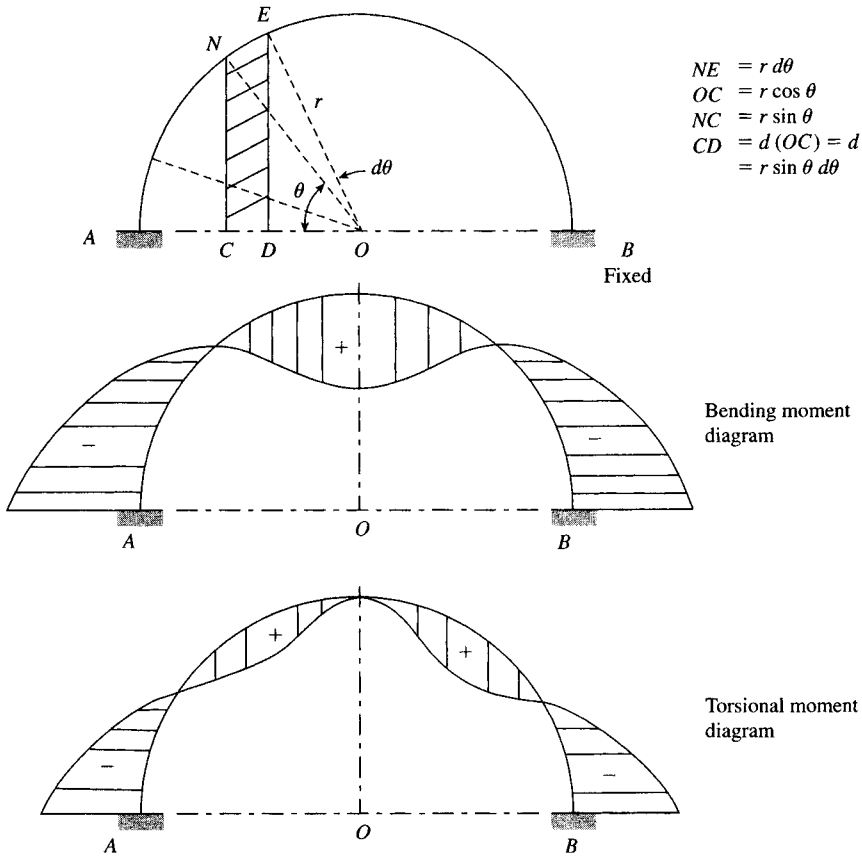


Figure 21.4 Semicircular beam fixed at the supports.

2. Shearing force at A: For a uniform symmetrical load on the slab, the shearing force at A is equal to

$$V_A = V_B = \int_0^{\pi/2} \left(\frac{wr}{2}\sin^2\theta\right)(r\,d\theta) = \frac{wr^2}{2}\left[\frac{\theta}{2} - \frac{1}{4}\sin 2\theta\right] \tag{21.11}$$
$$= \left(\frac{\pi}{8}\right)wr^2 = 0.39wr^2$$

3. Bending moment at A: Taking moments about the line AB, the bending moment at A is equal to

$$M_A = M_B = \int_0^{\pi/2} w'(r\,d\theta) \times (r\,\sin\theta) \tag{21.12}$$
$$= \int_0^{\pi/2} \left(\frac{wr}{2}\sin^2\theta\right)(r\,\sin\theta)(r\,d\theta) = -\frac{wr^3}{3}$$

4. Torsional moment at support A: T_A can be obtained by differentiating the strain energy of the beam with respect to T_A and equating it to 0. Considering that T_A is acting clockwise

at A, then the bending moment at any section N is calculated as follows:

$$M_N = V_A(r\ \sin\theta) - M_A\ \cos\theta + T_A\ \sin\theta - \int_0^{\theta} \left(\frac{wr}{2}\sin^2\theta\right)(r\,d\alpha) \times r\ \sin(\theta - \alpha)$$

$$M_N = wr^3\left[\frac{\pi}{8}\sin\theta - \left(\frac{1}{6}\right)(1 + \cos^2\theta)\right] + T_A\ \sin\theta \tag{21.13}$$

The torsional moment at any station N on the curved beam is equal to

$$T_n = -V_A r(1 - \cos\theta) + M_A\ \sin\theta + T_A\ \cos\theta + \int_0^{\pi/2} \left(\frac{wr}{2}\sin^2\alpha\right)(r\,d\alpha)$$
$$\times\ r[1 - \cos(\theta - \mathrm{a})]$$

$$T_N = wr^3\left[\frac{\pi}{8}(\cos\theta - 1) + \frac{\theta}{4} + \frac{1}{24}\sin 2\theta\right] + T_A\ \cos\theta \tag{21.14}$$

The strain energy is

$$U = \int \frac{M_N^2\,ds}{2\mathrm{EI}} + \int \frac{T_N^2\,ds}{2\mathrm{GJ}} \tag{21.15}$$

where

$ds = r\ d\theta$
G = modulus of rigidity
E = modulus of elasticity
I = moment of inertia of the section
J = rotational constant of the section
= polar moment of inertia

To obtain T_A, differentiate U with respect to T_A

$$\frac{\delta_U}{\delta T_A} = \int \frac{M_N}{EI} \times \frac{dM_N}{dT_A}(r\,d\theta) + \int \frac{T_N}{GJ} \times \frac{dT_N}{dT_A} \times (r\,d\theta) = 0$$

$$\frac{dM_N}{dT_A} = \sin\theta \quad \text{and} \quad \frac{dT_N}{dT_A} = \cos\theta$$

Therefore,

$$\frac{\delta_U}{\delta T_A} = \frac{r}{EI}\int_0^{\pi/2} \sin\theta \left\{wr^2\left[\frac{\pi}{8}\sin\theta - \frac{1}{6}(1 + \cos^2\theta)\right] + T_A\ \sin\theta\right\} d\theta$$
$$+ \frac{r}{GJ}\int_0^{\pi/2} \left\{wr^3\left[\frac{\pi}{8}(\cos\theta - 1) + \frac{\theta}{4} + \frac{1}{24}\sin 2\theta\right] + T_A\ \cos\theta\right\}\cos\theta \times d\theta = 0$$

$$\frac{r}{EI}\left[wr^3\left(\frac{\pi^2}{32} - \frac{2}{9}\right) + T_A\left(\frac{\pi}{4}\right)\right] + \frac{r}{GJ}\left[wr^3\left(\frac{\pi^2}{32} - \frac{2}{9}\right) + T_A\left(\frac{\pi}{4}\right)\right] = 0$$

Let $EI/GJ = \lambda$; then

$$T_A\left(\frac{\pi}{4}\right)(1+\lambda) = wr^3\left[\left(\frac{2}{9}-\frac{\pi^2}{32}\right)+\lambda\left(\frac{2}{9}-\frac{\pi^2}{32}\right)\right]$$

$$= wr^3\left(\frac{2}{9}-\frac{\pi^2}{32}\right)(1+\lambda) = -0.0862wr^3(1+\lambda)$$

Therefore,

$$T_A = -0.11wr^3 \tag{21.16}$$

Substituting the value of T_A in Eq. 21.13, the bending moment at any point N is equal to

$$M_N = wr^3\left[\frac{\pi}{8}\sin\theta - \frac{1}{6}(1+\cos^2\theta) - 0.11\ \sin\theta\right] \tag{21.17}$$

Substituting the value of T_A in Eq. 21.14,

$$T_N = wr^3\left[\frac{\pi}{8}(\cos\theta - 1) + \frac{\theta}{4} + \frac{1}{24}\sin 2\theta - 0.11\ \cos\theta\right] \tag{21.18}$$

5. The value of G/E for concrete may be assumed to be equal to 0.43. The value of J for a circular section is $(\pi/2)r^4$, whereas J for a square section of side x is equal to $0.141x^4$. For a rectangular section with short and long sides x and y, respectively, J can be calculated as follows:

$$J = K' \times y^3 \tag{21.19}$$

The values of K' are calculated as follows:

$$K' = \frac{1}{16}\left[\frac{16}{3} - 3.36\frac{x}{y}\left(1 - \frac{x^4}{12y^4}\right)\right] \tag{21.20}$$

whereas

$$\lambda = \frac{EI}{GJ} = \left(\frac{1}{0.43}\right)\left(\frac{xy^3}{12}\right)\left(\frac{1}{K'yx^3}\right) = \frac{1}{5.16K'}\left(\frac{y}{x}\right)^2$$

Values of K' and λ are both shown in Table 21.2.

Example 21.2

Determine the factored bending and torsional moments in sections C and D of the 10-ft-radius semicircular beam $ADCB$ shown in Fig. 21.5. The beam is part of a floor slab that carries a uniform factored load of 304 psf (including self-weight).

Table 21.2 Values of K' and λ for Different Values of y/x

y/x	0.5	1.0	1.1	1.2	1.25	1.3	1.4	1.5	1.6
K'	0.473	0.141	0.154	0.166	0.172	0.177	0.187	0.196	0.204
λ	0.102	1.37	1.52	1.68	1.76	1.85	2.03	2.22	2.43
y/x	1.7	1.75	2.0	2.5	3.0	4.0	5.0	6.0	10
K'	0.211	0.214	0.229	0.249	0.263	0.281	0.291	0.300	0.312
λ	2.65	2.77	3.39	4.86	6.63	11.03	16.5	23.3	62.1

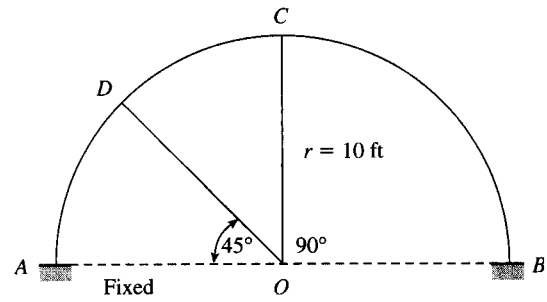


Figure 21.5 Example 21.2.

Solution

1. Factored load $w_u = 304$ psf.
2. For the section at C, $\theta = \pi/2$ and $w_u r^3 = 0.304(10)^3 = 304$. From Eq. 21.17,

$$M_c = 304\left[\frac{\pi}{8}\sin\frac{\pi}{2} - \frac{1}{6}\left(1+\cos^2\frac{\pi}{2}\right) - 0.11\sin\frac{\pi}{2}\right] = 35.3 \text{ K·ft}$$

From Eq. 21.18,

$$T_c = 304\left[\frac{\pi}{8}\left(\cos\frac{\pi}{2} - 1\right) + \frac{\pi}{8} + \frac{1}{24}\sin\pi - 0.11\cos\frac{\pi}{2}\right] = 0$$

3. For the section at D, $\theta = \pi/4$.

$$M_D = 304\left[\frac{\pi}{8}\sin\frac{\pi}{4} - \frac{1}{6}\left(1+\cos^2\frac{\pi}{4}\right) - 0.11\sin\frac{\pi}{4}\right] = -15.2 \text{ K·ft}$$

$$T_D = 304\left[\frac{\pi}{8}\left(\cos\frac{\pi}{4} - 1\right) + \frac{\pi}{16} + \frac{1}{24}\sin\frac{\pi}{2} - 0.11\cos\frac{\pi}{4}\right] = 13.7 \text{ K·ft}$$

4. Maximum shearing force occurs at the supports.

$$V_A = 0.39 w_u r^2 = 0.39(0.304)(100) = 11.9 \text{ K}$$

Maximum positive moment occurs at C, whereas the maximum negative moment occurs at the supports.

$$M_A = -\frac{w_u r^3}{3} = -\frac{304}{3} = 101.3 \text{ K·ft}$$

5. Design the critical sections for shear, bending, and torsional moments, as explained in Example 21.1.

21.4 FIXED-END SEMICIRCULAR BEAM UNDER UNIFORM LOADING

The previous section dealt with a semicircular beam fixed at both ends and subjected to a variable distributed load. If the load is uniform, then the beam will be subjected to a uniformly distributed load w K/ft, as shown in Fig. 21.6. The forces in the curved beam can be determined as follows:

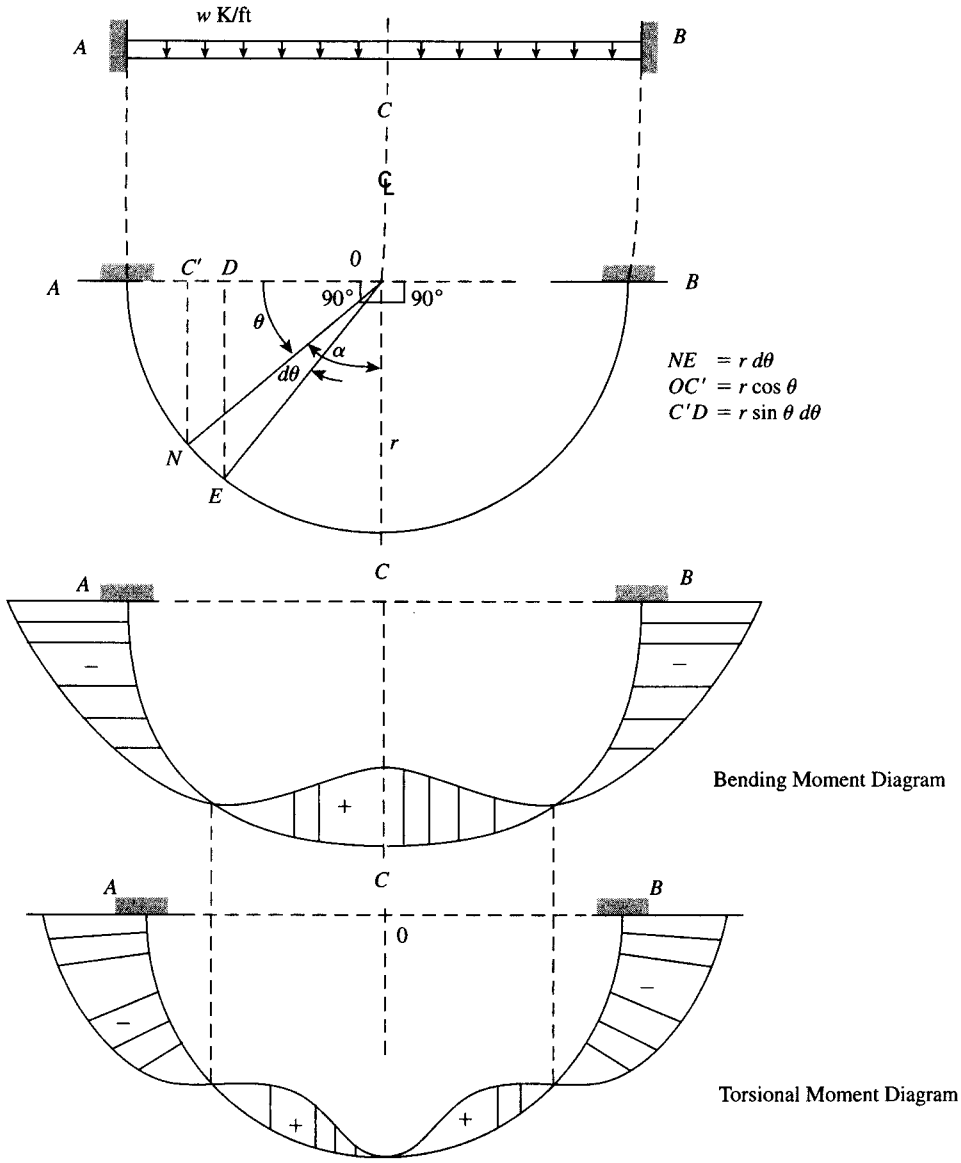


Figure 21.6 Semicircular beam under uniform load.

1. Shearing force at A:

$$V_A = V_B = \int_0^{\pi/2} wr\, d\theta = wr\frac{\pi}{2} = 1.57wr \tag{21.21}$$

2. Bending moment at A:

$$M_A = M_B = \int_0^{\pi/2} w(r\, d\theta) \times (r\, \sin\theta) = wr^2 \tag{21.22}$$

3. Bending moment at any section N on the curved beam when the torsional moment at A (T_A) acts clockwise:

$$M_N = V_A\ (r\ \sin\theta) - M_A\ \cos\theta + T_A\ \sin\theta - \int_0^\theta (wr\,d\alpha)[r\ \sin(\theta - a)]$$

$$= \frac{\pi}{2} wr^2 \sin\theta - wr^2 \cos\theta + T_A\ \sin\theta - [wr^2 - wr^2 \cos\theta]$$

$$M_N = wr^2\left[\frac{\pi}{2}\sin\theta - 1\right] + T_A\ \sin\theta \tag{21.23}$$

4. Torsional moment at any section N:

$$T_N = -V_A\ r(1 - \cos\theta) + M_A\ \sin\theta + T_A\ \cos\theta + \int_0^\theta (wr\,d\alpha)r[1 - \cos(\theta - \alpha)]$$

$$= -\frac{\pi}{2} wr^2 + \frac{\pi}{2} wr^2 \cos\theta + T_A\ \cos\theta + M_A\ \sin\theta + wr^2\theta - wr^2 \sin\theta$$

Substitute $M_A = wr^2$:

$$T_N = wr^2\left[\frac{\pi}{2}\cos\theta - \frac{\pi}{2} + \theta\right] + T_A\ \cos\theta \tag{21.24}$$

5. The strain energy expression was given in the previous section:

$$U = \int \frac{M_N^2\ ds}{2EI} + \int \frac{T_N^2\ ds}{2GJ} \tag{21.25}$$

To obtain T_A, differentiate U with respect to T_A.

$$\frac{\delta U}{\delta T_A} = \int \frac{M_N}{EI} \times \frac{dM_N}{dT_A}(r\,d\theta) + \int \frac{T_N}{GJ} \times \frac{dT_N}{dT_A} \times (r\,d\theta) = 0$$

$$\frac{dM_N}{dT_A} = \sin\theta \quad \text{and} \quad \frac{dT_N}{dT_A} = \cos\theta \quad \text{(from the preceding equations)}$$

$$\frac{\delta U}{\delta T_A} = \frac{r}{EI}\int_0^{\pi/2}\left[wr^2\left(\frac{\pi}{2}\sin - 1\right) + T_A\ \sin\theta\right]\sin\theta\ d\theta$$

$$+ \frac{r}{GJ}\int_0^{\pi/2}\left[wr^2\left(\frac{\pi}{2}\cos\theta - \frac{\pi}{2} + \theta\right) + T_A\ \cos\theta\right]\cos\theta\ d\theta = 0$$

The integration of the preceding equation produces the following:

$$\frac{\delta U}{\delta T_A} = \frac{r}{EI}\left[wr^2\left(\frac{\pi^2}{8} - 1\right) + \frac{\pi}{4}T_A\right] + \frac{r}{GJ}\left[wr^2\left(\frac{\pi^2}{8} - 1\right) + \frac{\pi}{4}T_A\right] = 0$$

$$r\left[wr^2\left(\frac{\pi^2}{8} - 1\right) + \frac{\pi}{4}T_A\right]\left(\frac{EI}{GJ} + 1\right) = 0$$

Because EI/GJ is not equal to zero,

$$wr^2\left(\frac{\pi^2}{8} - 1\right) + \frac{\pi}{4}T_A = 0$$

$$T_A = -wr^2\left(\frac{4}{\pi}\right)\left(\frac{\pi^2}{8} - 1\right) = -0.3wr^2 \tag{21.26}$$

6. Substitute T_A in Eq. 21.23:

$$M_N = wr^2\left[\left(\frac{\pi}{2}\sin\theta - 1\right) - \left(\frac{\pi}{2} - \frac{4}{\pi}\right)\sin\theta\right]$$

$$= wr^2\left(\frac{4}{\pi}\sin\theta - 1\right) \tag{21.27}$$

$$T_N = wr^2\left[\left(\frac{\pi}{2}\cos\theta + \theta - \frac{\pi}{2}\right) - \left(\frac{\pi}{2} - \frac{4}{\pi}\right)\cos\theta\right]$$

$$= wr^2\left(\theta - \frac{\pi}{2} + \frac{4}{\pi}\cos\theta\right) \tag{21.28}$$

The values of the bending and torsional moments at any section N are independent of λ $(1 = EI/GJ)$.

7. Bending and torsional moments at midspan, section C, can be found by substituting $\theta = \pi/2$ in Eqs. 21.27 and 21.28:

$$M_c = wr^2\left(\frac{4}{\pi} - 1\right) = 0.273wr^2 \tag{21.29}$$

$$T_c = wr^2\left(\frac{\pi}{2} - \frac{\pi}{2} + 0\right) = 0 \tag{21.30}$$

21.5 CIRCULAR BEAM SUBJECTED TO UNIFORM LOADING

The previous section dealt with a semicircular beam subjected to a uniformly distributed load. The forces acting on the beam at any section vary with the intensity of load, the span (or the radius of the circular beam), and the angle α measured from the centerline axis of the beam.

Considering the general case of a circular beam fixed at both ends and subjected to a uniform load w (K/ft), as shown in Fig. 21.7, the bending and torsional moments can be calculated from the following expressions:

1. The moment at the centerline of the beam, M_c, can be derived using the strain energy expression, equation 21.25, and can be expressed as follows:

$$M_c = \frac{wr^2}{K_4}[\lambda(K_1 + K_2 - K_3) + (K_1 - K_2)] \tag{21.31}$$

where

$$\lambda = \frac{EI}{GJ}$$

$K_1 = 2(2\sin\theta - \theta)$

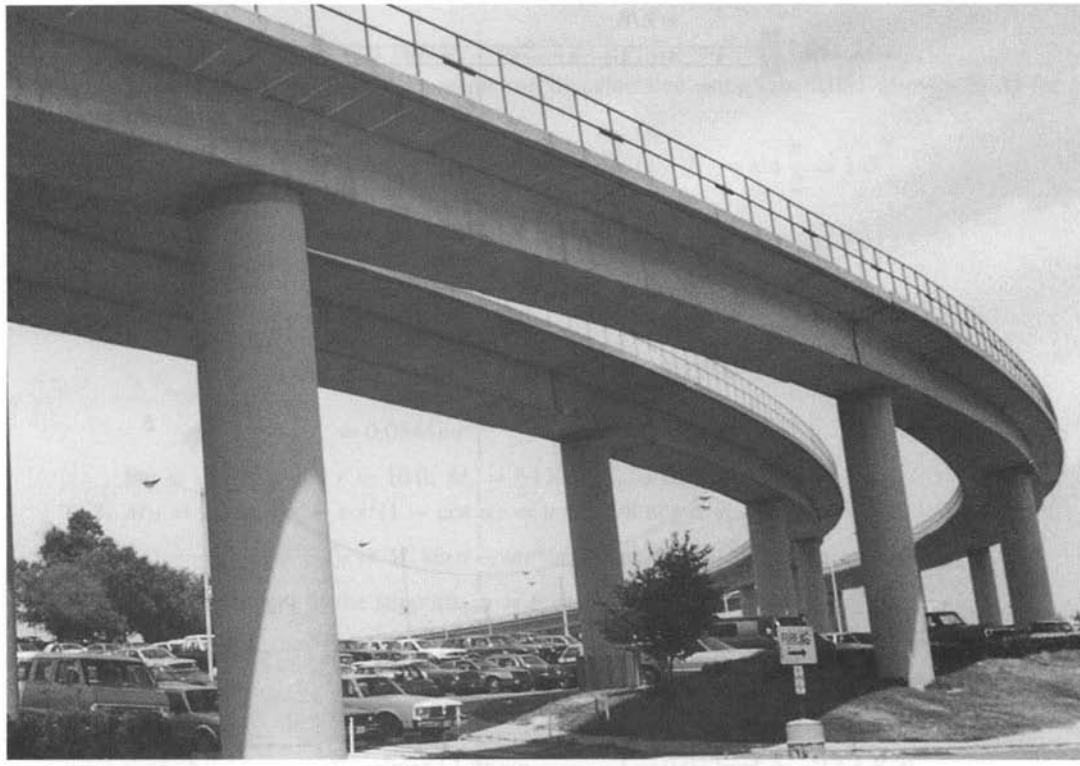
$K_2 = 2\sin\theta\cos\theta = \sin 2\theta$

$K_3 = 4\theta\cos\theta$

$K_4 = 2\theta(\lambda + 1) - (\lambda - 1)\sin 2\theta$

2θ = total central angle of the ends of the beam, angle AOB (Fig. 19.7)

The torsional moment at the centerline section, T_c, is 0.



Curved-beam bridge, Washington, D.C.

2. The moment at any section N on the curved beam where ON makes an angle α with the centerline axis (Fig. 21.7) is

$$M_N = M_c \cos\alpha - wr^2(1 - \cos\alpha) \tag{21.32}$$

3. The torsional moment at any section N on the curved beam as a function of the angle α was derived earlier:

$$T_N = M_c \sin\alpha - wr^2(\alpha - \sin\alpha) \tag{21.33}$$

4. To compute the bending moment and torsional moment at the supports, substitute θ for α in the preceding equations:

$$M_A = M_c \cos\theta - wr^2(1 - \cos\theta) \tag{21.34}$$

$$T_A = M_c \sin\theta - wr^2(\theta - \sin\theta) \tag{21.35}$$

Example 21.3

A curved beam has a quarter-circle shape in plan with a 10-ft radius. The beam has a rectangular section with the ratio of the long to the short side of 2.0 and is subjected to a factored load of 8 K/ft. Determine the bending and torsional moments at the centerline of the beam, supports, and maximum values.

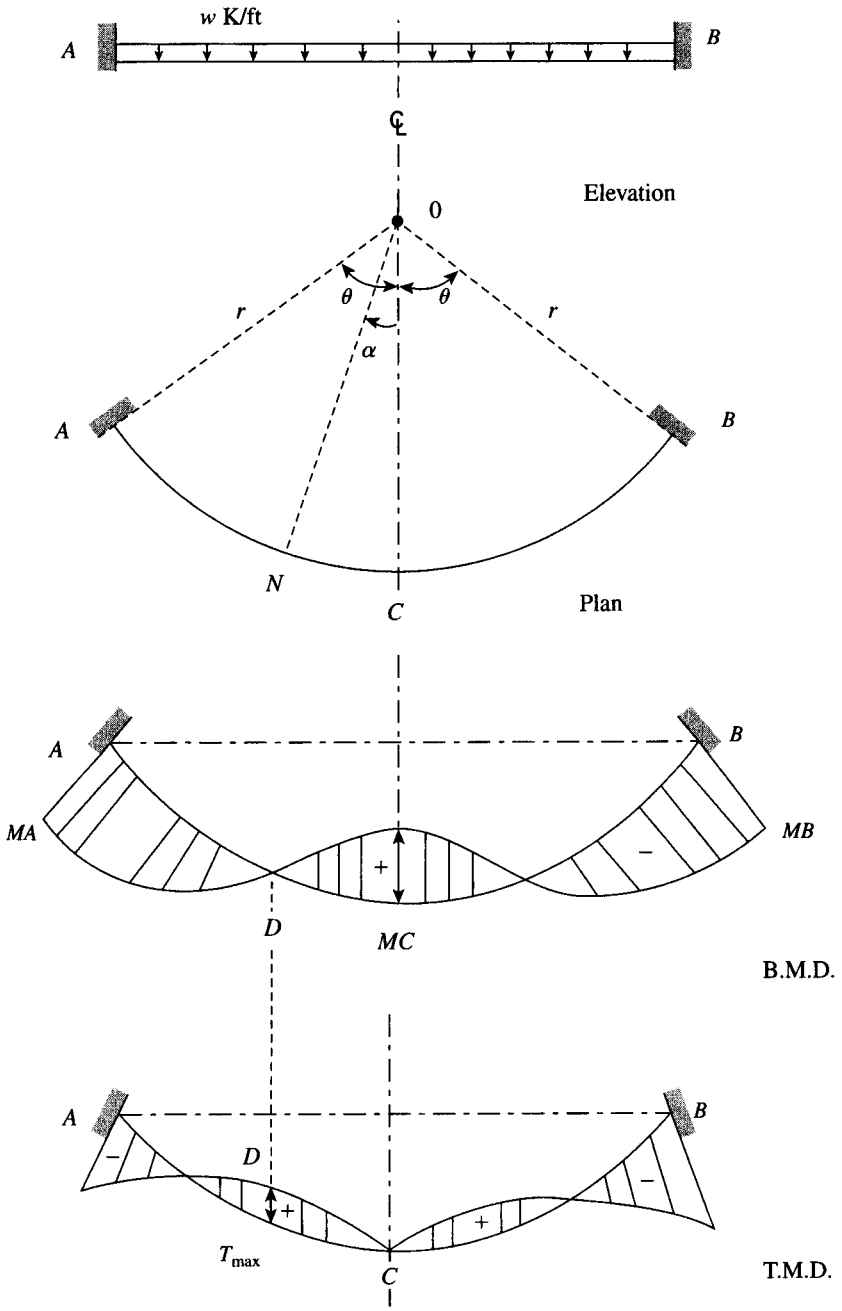


Figure 21.7 Circular beam subjected to uniform load, showing the bending moment diagram (BMD) and the torsional moment diagram (TMD).

Solution

1. For a rectangular section with $y/x = 2$, $\lambda = EI/GJ = 3.39$ (Table 21.2).
2. The bending and torsional moments can be calculated using Eqs. 21.31 through 21.35 for $\theta = \pi/4$. From Eq. 21.31,

$$K_1 = 2\left(2\sin\frac{\pi}{4} - \frac{\pi}{4}\right) = 1.2576, \quad K_2 = \sin\frac{\pi}{2} = 1.0$$

$$K_3 = 4\left(\frac{\pi}{4}\right)\cos\frac{\pi}{4} = 2.2214$$

$$K_4 = 2\left(\frac{\pi}{4}\right)(3.39 + 1) - (3.39 - 1)\sin\frac{\pi}{2} = 4.506$$

$$M_c = \frac{wr^2}{4.506}[3.39(1.2576 + 1.0 - 2.2214) + (1.2576 - 1.0)]$$

$$= 0.0844wr^2$$

For $w = 8$ K·ft and $r = 10$ ft, $M_c = 64$ K·ft; $T_c = 0$.

3. $M_N = M_c \cos\alpha - wr^2(1 - \cos\alpha) = wr^2(1.08\cos\alpha - 1)$

$$T_N = M_c \sin\alpha - wr^2(\alpha - \sin\alpha) = wr^2(1.08\ \sin\alpha - \alpha)$$

For the moments at the supports, $\alpha = \theta = \pi/4$.

$$M_A = wr^2\left(1.08\cos\frac{\pi}{4} - 1\right) = -0.236wr^2$$

$$= -0.236 \times 8 \times (10)^2 = -189 \text{ K·ft}$$

$$T_A = wr^2\left(1.08\sin\frac{\pi}{4} - \frac{\pi}{4}\right) = 0.022wr^2 = -17.4 \text{ K·ft}$$

For $M_N = 0$, $1.08\cos\alpha - 1 = 0$, or $\cos\alpha = 0.926$ and $\alpha = 22.2^\circ = 0.387$ rad. To calculate $T_{N\max}$, let $dT_N/d\alpha = 0$, or $(1.08\cos\alpha - 1) = 0$. Then $\cos\alpha = 0.926$ and $\alpha = 22.2^\circ$.

$$T_N(\max) = wr^2(1.08\ \sin 22.2 - 0.387) = 0.0211wr^2$$

$$T_{N\max} = 0.0211 - 800 = 16.85 \text{ K·ft}$$

21.6 CIRCULAR BEAM SUBJECTED TO A CONCENTRATED LOAD AT MIDSPAN

If a concentrated load is applied at the midspan of a circular beam, the resulting moments vary with the magnitude of the load, the span, and the coefficient $\lambda = EI/GJ$. Considering the general case of a circular beam fixed at both ends and subjected to a concentrated load P at midspan (Fig. 21.8), the bending and torsional moments can be calculated from the following expressions:

1. The moment at the centerline of the beam, section C, can be expressed as follows:

$$M_c = \frac{\lambda(2 - 2\ \cos\theta - \sin^2\theta) + \sin^2\theta}{2\theta(\lambda + 1) - (\lambda - 1)\sin 2\theta}(Pr) \tag{21.36}$$

$$M_c = \frac{Pr}{K_3}(\lambda K_1 + K_2)$$

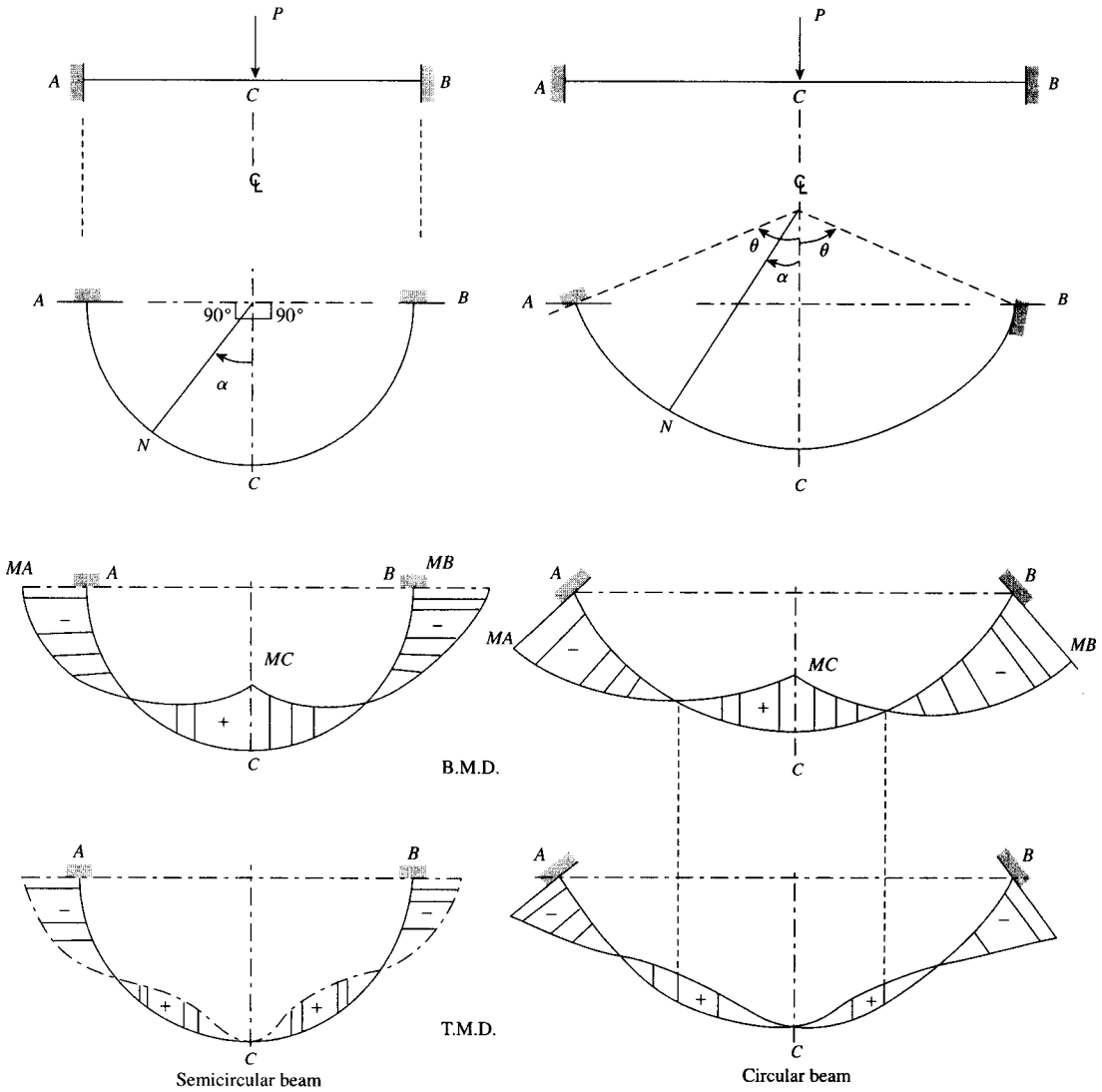


Figure 21.8 Circular beam subjected to a concentrated load at midspan, showing the bending moment diagram (BMD) and the torsional moment diagram (TMD).

where

$$\lambda = \frac{EI}{GJ}$$

$$K_1 = (2 - 2\cos\theta - \sin^2\theta)$$

$$K_2 = \sin^2\theta$$

$$K_3 = 2\theta\,(\lambda + 1) - (\lambda - 1)\sin^2\theta$$

The torsional moment at the centerline is $T_c = 0$.

2. The bending and torsional moments at any section N on the curved beam where ON makes an angle α with the centerline axis are calculated as follows:

$$M_N = M_c \cos\alpha - \left(\frac{P}{2}r\right)\sin\alpha \tag{21.37}$$

$$T_N = M_c \sin\alpha - \left(\frac{P}{2}r\right)(1 - \cos\alpha) \tag{21.38}$$

3. To compute the bending and torsional moments at the supports, substitute θ for α.

$$M_A = M_c \cos\theta - \left(\frac{P}{2}r\right)\sin\theta \tag{21.39}$$

$$T_A = M_c \sin\theta - \left(\frac{P}{2}r\right)(1 - \cos\theta) \tag{21.40}$$

Example 21.4

Determine the bending and torsional moments of the quarter-circle beam of Example 21.3 if $\lambda = 1.0$ and the beam is subjected to a concentrated load at midspan of $P = 20\,\text{K}$.

Solution

1. Given: $\lambda = 1.0$ and $\theta = \pi/4$. Therefore,

$$M_c = \left(\frac{Pr}{2}\right)\left(\frac{1 - \cos\theta}{\theta}\right)$$

(Eq. 21.36) and $T_c = 0$. For $\theta = \pi/4$,

$$M_c = 0.187Pr = 0.187(20 \times 10) = 37.4\ \text{K·ft}$$

2. From Eqs. 21.39 and 21.40,

$$M_A = 0.187Pr\cos\frac{\pi}{4} - \frac{Pr}{2}\sin\frac{\pi}{4} = -0.22\ Pr$$

$$= -0.22 \times (200) = -44\ \text{K·ft}$$

$$T_A = 0.187Pr\sin\frac{\pi}{4} - 0.5Pr\left(1 - \cos\frac{\pi}{4}\right) = -0.0142Pr$$

$$= -0.0142 \times 200 = -2.84\ \text{K·ft}$$

3. $M_N = 0$ when

$$M_c\cos\alpha - \frac{Pr}{2}\sin\alpha = 0 \qquad \text{(Eq. 21.37)}$$

$$0.187Pr\cos\alpha - 0.5Pr\sin\alpha = 0$$

$$\tan\alpha = 0.374 \quad \text{and} \quad \alpha = 20.5^\circ$$

$T_n = 0$ when $M_c \sin\alpha - (P/2)\,r(1 - \cos\alpha) = 0$ (Eq. 21.38), from which $\alpha = 37.7^\circ$.

4. To compute $T_{\max}$, let $dT_N/d\alpha = 0$ (equation (21.38)).

$$0.187Pr\cos\alpha - 0.5Pr\sin\alpha = 0,\ \tan\alpha = 0.374$$

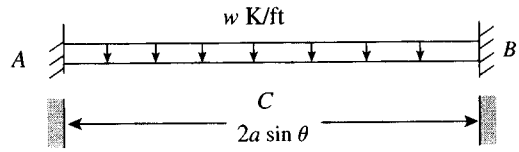
and $\alpha = 20.5^\circ$. Substitute $\alpha = 20.5^\circ$ in Eq. (21.38) to get $T_{\max} = 0.035Pr = 7\,\text{K·ft}$.

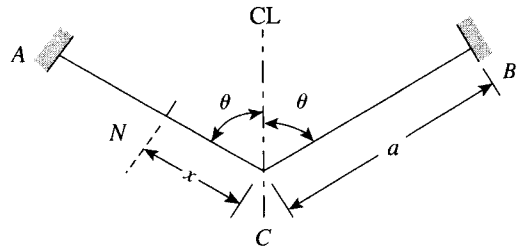
21.7 V-SHAPE BEAMS SUBJECTED TO UNIFORM LOADING

Beams that have a V shape in plan and are subjected to loads normal to the plane of the beam may be analyzed using the *strain-energy principles*. Fig. 21.9 shows typical bending moment diagram for a V-shape beam subjected to a uniform load w. Considering the general case of a V-shape beam fixed at both ends and subjected to a uniform load w (K/ft), the bending and torsional moments can be calculated from the following expressions:

1. The moment at the centerline of the beam, section C, is calculated as follows:

$$M_c = (wa^2)\left[\frac{\sin^2\theta}{6(\sin^2\theta + \lambda\ \cos^2\theta)}\right] \tag{21.41}$$





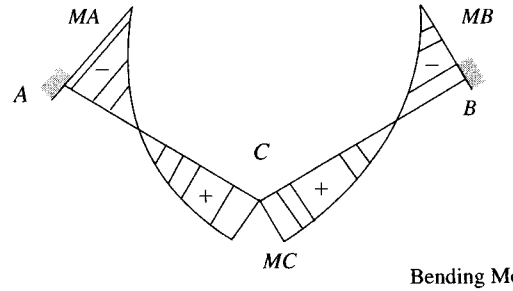


Figure 21.9 V-shape beam under uniform load.



90° V-shape beams, London, Ontario, Canada.

where

$\lambda = \dfrac{EI}{GJ}$

a = half the total length of the beam (length AC)

θ = half the angle between the two sides of the V-shape beam.

The torsional moment at the centerline section is

$$T_c = \frac{M_c}{\sin\theta} \times \cos\theta = M_c \cot\theta \tag{21.42}$$

2. The bending and torsional moments at any section N along half the beam AC or BC at a distance x measured from section C are calculated as follows:

$$M_N = M_c - w\frac{x^2}{2} \tag{21.43}$$

$$T_N = T_c = \frac{M_c}{\sin\theta} \times \cos\theta = M_c \cot\theta \tag{21.44}$$

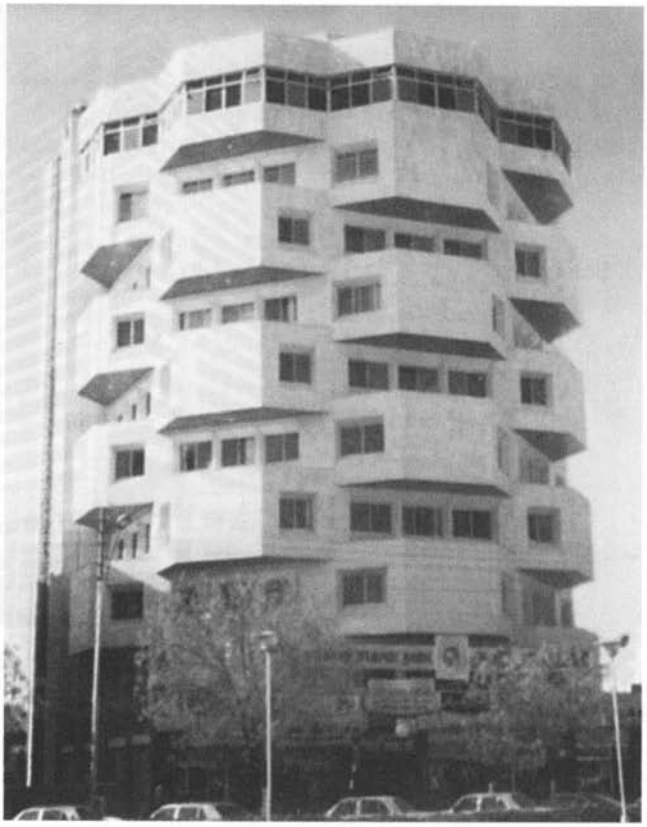
To compute the moments at the supports, let $x = a$. Then

$$M_A = M_c - w\frac{a^2}{2}$$

$$T_A = T_c = M_c \cot\theta$$

Example 21.5

Determine the bending and torsional moments in a V-shape beam subjected to a uniform load of 6 K/ft. The length of half the beam is $a = 10$ ft and the angle between the V-shape members is $2\theta = \pi/2$. The beam section is rectangular with a ratio of long side to short side of 2.



Apartment building.

Solution

1. For a rectangular section with the sides ratio, $y/x = 2$, $\lambda = 3.39$. For this beam $\theta = \pi/4$.

2. $M_c = \dfrac{wa^2}{6}\left[\dfrac{\sin^2\theta}{(\sin^2\theta + \lambda\cos^2\theta)}\right]$ (Eq. 21.41)

$$M_c = \frac{wa^2}{6}\left(\frac{0.5}{(0.5 + 3.39 \times 0.5)}\right) = 0.038wa^2$$

$$= 0.038 \times 6(10)^2 = 22.8 \text{ K·ft}$$

$$M_A = M_c - w\frac{a^2}{2} = 0.038wa^2 - 0.5wa^2 = -0.462wa^2$$

$$= -277.2 \text{ K·ft}$$

$M_N = 0$ when $M_c - w\dfrac{x^2}{2} = 0$

or $0.038wa^2 - 0.5wx^2 = 0$, so $x = 0.276a = 2.76$ ft measured from c.

3. $T_A = T_C = M_C \cot\theta$

$$= 0.038wa^2 = 0.038 \times 600 = 22.8 \text{ K·ft}$$

21.8 V-SHAPE BEAMS SUBJECTED TO A CONCENTRATED LOAD AT THE CENTERLINE OF THE BEAM

The general equations for computing the bending and torsional moments in a V-shape beam fixed at both ends and subjected to a concentrated load P at the centerline of the beam (Fig. 21.10) are as follows:

1. The moment at the centerline of the beam, section C, for any value of λ, is

$$M_c = \left(\frac{Pa}{4}\right)\left(\frac{\sin^2\theta}{(\sin^2\theta + \lambda\cos^2\theta)}\right) \tag{21.45}$$

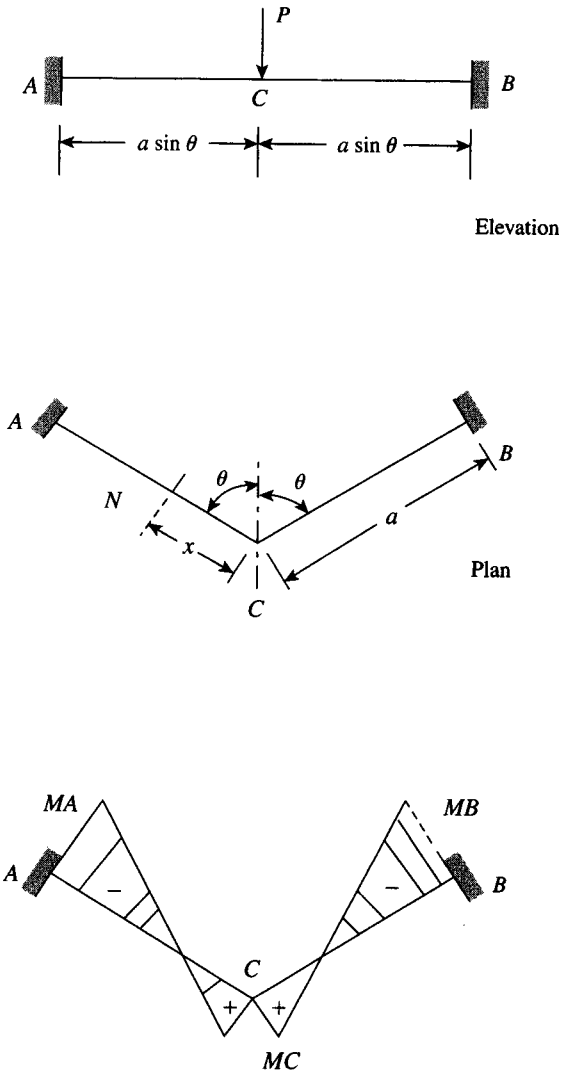


Figure 21.10 V-shape beam under concentrated load.

where

$$\lambda = \frac{EI}{GJ}$$

a = half the total length of the beam (part AB or BC)

θ = half the angle between the two sides of the V-shape beam

The torsional moment at the centerline section is

$$T_c = \frac{M_c}{\sin\theta}\cos\theta = M_c \cot\theta \tag{21.46}$$

2. The bending and torsional moments at any section N along half the beam AC or BC at a distance x measured from C are calculated as follows:

$$M_N = M_c - \frac{Px}{2} \tag{21.47}$$

$$T_N = T_c = M_c \cot\theta \tag{21.48}$$

The moments at the supports are determined by assuming $x = a$:

$$M_A = M_c - \frac{Pa}{2} \tag{21.49}$$

$$T_A = T_c = M_c \cot\theta \tag{21.50}$$

Example 21.6

Determine the bending and torsional moments in a V-shape beam subjected to a concentrated load $P = 30\,\text{K}$ acting at the centerline of the beam. Given: $\theta = \pi/4$, $y/x = 2.0$, and a = 12 ft.

Solution

1. For a rectangular section with $y/x = 2.0$, $\lambda = 3.39$.

2. $$M_c = \frac{Pa}{4}\left(\frac{\sin^2 \pi/4}{\sin^2 \pi/4 + 3.39\cos^2 \pi/4}\right) = 0.057(Pa)$$

$$= 0.057 \times 30 \times 12 = 20.5\,\text{K·ft}$$

$$M_A = M_c - \frac{Pa}{2} = (0.057 - 0.5)Pa = -0.443(Pa)$$

$$= -0.0443 \times 360 = -159.5\,\text{K·ft}$$

$M_N = 0$ when $M_c - \frac{Px}{2} = 0$

Hence $0.057Pa - 0.5\,Px = 0$ and $x = 0.114a = 0.114 \times 12 = 1.37$ ft measured from c.

3. $T_A = T_c = T_N = M_c \cot\frac{\pi}{4} = 0.057(Pa) = 20.5\,\text{K·ft}$

Example 21.7

Determine the bending and torsional moments in the beam of Example 21.6 if the angle θ is $\pi/2$ (a straight beam fixed at both ends).

Solution

Given $\theta = \pi/2$ and the span $L = 2a$ = the distance between the two supports. The bending moment at the centerline is

$$M_c = \frac{Pa}{4}\left(\frac{1}{1}\right) = \frac{Pa}{4} = \frac{PL}{8} = +90 \text{ K·ft}$$

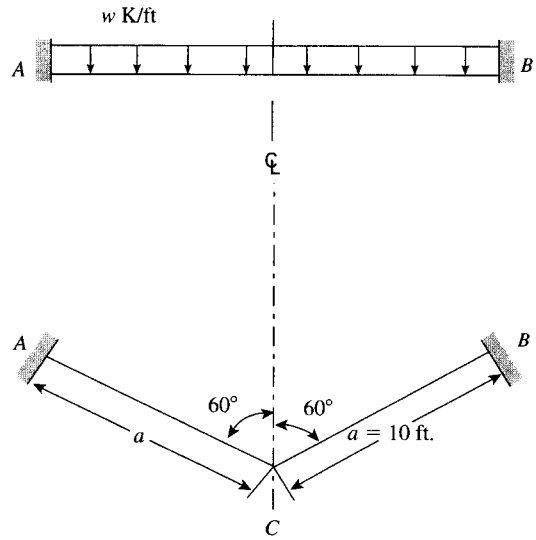
$$M_A = M_c - \frac{Pa}{2} = \frac{PL}{8} - \frac{P}{2}\left(\frac{L}{2}\right) = -\frac{PL}{8} = -90 \text{ K·ft}$$

$$T_A = T_c = 0$$

These values are similar to those obtained from the structural analysis of the fixed-end beam subjected to a concentrated load at midspan.

Example 21.8

The beam shown in Fig. 21.11 has a V shape in plan and carries a uniform dead load of 3.5 K/ft and a live load of 3 K/ft. The inclined length of half the beam is $a = 10$ ft and $\theta = 60°$. Design the beam for shear, bending, and torsional moments using $f'_c = 4$ Ksi and $f_y = 60$ Ksi.



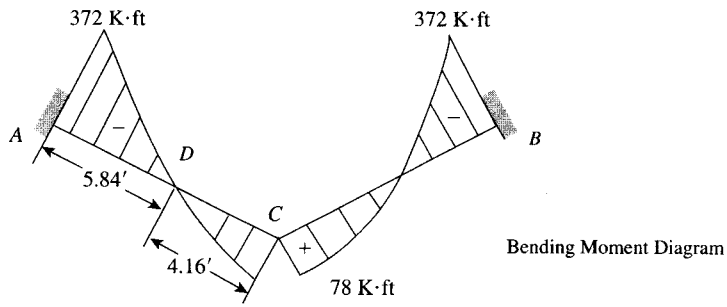


Figure 21.11 Example 21.8.

Solution

1. $w_u = 1.2D + 1.6L = 1.2 \times 3.5 + 1.6 \times 3 = 9.0$ K/ft.
2. Assuming a rectangular section with a ratio of long to short side of $y/x = 1.75$, the value of λ is 2.77 (from Table 21.2). For $\theta = 60^\circ = \pi/3$,

$$M_c = \frac{w_u a^2 \sin^2\theta}{6(\sin^2\theta + \lambda\cos^2\theta)} = \frac{9(100)(0.75)}{6(0.75 + 2.77 \times 0.25)} = +\ 78\ \text{K·ft}$$

$$M_A = M_c - w_u\frac{a^2}{2} = 78 - 9\left(\frac{100}{2}\right) = -372\ \text{K·ft}$$

$$T_A = M_c \cot\theta = 78 \times 0.577 = 45\ \text{K·ft} = 540\ \text{K·in.}$$

$$T_c\ (\text{at } x = 0) = M_c \cot\theta = 45\ \text{K·ft} = 540\ \text{K·in.}$$

$$V_A = 9 \times 10 = 90\ \text{K}$$

The bending moment is zero at $M_N = 0 = M_c - w_u\, x^2/2$. Hence, $78 - \frac{9}{2}x^2 = 0$ and x = 4.16 ft measured from c. The bending moment diagram is shown in Fig. 21.11.

3. Design for a bending moment, M_u, equal to −372 K·ft.
   a. For $f'_c = 4$ Ksi, $f_y = 60$ Ksi, $\rho_{\max} = 0.0018$, choose $\rho = 0.015$, $R_u = 702$ psi and $\phi = 0.9$. (Appendix A)

$$bd^2 = \frac{M_u}{R_u} = \frac{372 \times 12}{0.705} = 6332\ \text{in.}^3$$

   For a ratio,

$$\frac{y}{x} = \frac{(d+3)}{b} = 1.75$$

   as assumed, then $d = 21.4$ in. and $b = 13.8$ in. Use a section 14 × 24 in.

$$A_s = \rho_{\max} bd = 0.015(14 \times 21.4) = 4.5\ \text{in.}^2$$

   b. For the section at midspan, $M_u = 78$ K·ft and actual $d = 21.5$ in.

$$R_u = \frac{M_u}{bd^2} = \frac{78{,}000 \times 12}{14 \times (21.5)} = 145\ \text{psi}$$

$$\rho < \rho_{\min} = 0.0033$$

   Use $A_s = 0.0033 \times 14 \times 21.5 = 1.0\ \text{in.}^2$

   c. Design for torsional moment and shear: $T_u = 45$ K·ft for all sections.

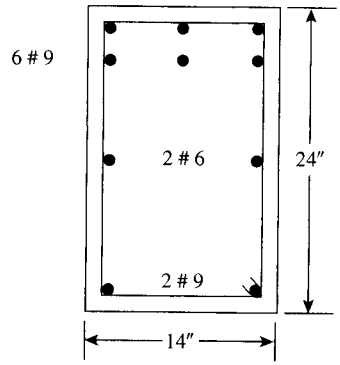
$$V_u(\text{at distance } d) = 90 - \frac{21.5}{12} \times 9 = 74.0\ \text{K}$$

   The design procedure will be similar to that of Example 21.1. Details of the final section are shown in Fig. 21.12.

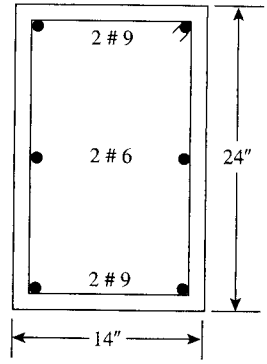
SUMMARY

Sections 21.1– 21.5

In a curved beam in plan, the center of gravity of normal loads lies outside the line joining the supports developing torsional moments. The analysis of uniformly loaded circular beams is presented in these sections.



Section at A



Section at C

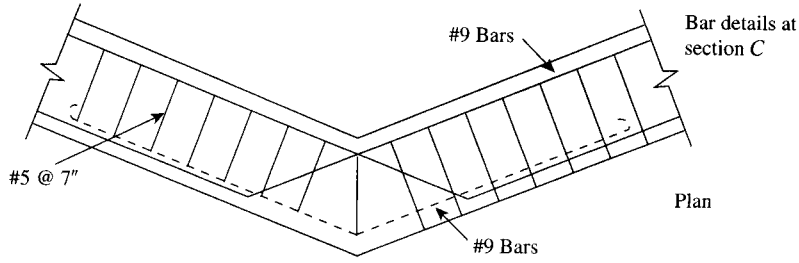


Figure 21.12 Example 21.8.

Section 21.6

The analysis of circular beams subjected to concentrated loads is presented in this section.

Section 21.7

V-shape beams subjected to gravity loads may be analyzed using the strain energy principles. Equations to calculate the torsional moments of these types of beams are presented.

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PROBLEMS

21.1 A circular beam is supported on six equally spaced columns, and its centerline lies on a circle 20 ft in diameter. The beam carries a uniform dead load of 9.8 K/ft and a live load of 5 K/ft. Design the beam using $f'_c = 4$Ksi, $f_y = 60$ Ksi, and $b = 14$ in.

21.2 Design a semicircular beam fixed on both ends. The center of columns lies on a circle 12 ft in diameter. The beam carries uniform dead and live loads of 4.9 K/ft and 3 K/ft, respectively. Use $f'_c = 4$ Ksi, $f_y = 60$ Ksi, and $b = 20$ in.

21.3 Determine the factored bending and torsional moment at sections C and D of the fixed-end beam shown in Fig. 21.5 if the diameter of the circle is 30 ft. The beam is part of a floor slab that carries a uniform dead load (including its own weight) of 126 psf and a live load of 120 psf.

21.4 A quarter-circle cantilever beam has a radius of 8 ft and carries a uniform dead load of 6.4 K/ft and a concentrated live load of 4.25 K at its free end. Design the beam using $f'_c = 4$ Ksi, $f_y = 60$ Ksi, and $b = 14$ in.

21.5 Design the beam shown in Fig. 21.11 if the inclined length of half the beam is $a = 8$ ft. The beam has a 60° V shape in plan and carries uniform dead and live loads of 3.8 K/ft and 4 K/ft. Assume the ratio of the long to the short side of the rectangular section is 2. Use $f'_c = 4$ Ksi, and $f_y = 60$ Ksi.

APPENDIX A

DESIGN TABLES (U.S. CUSTOMARY UNITS)

Table A.1 Values of R_u and $\frac{a}{d}$ for $f'_c = 3000$ psi

Table A.2 Values of R_u and $\frac{a}{d}$ for $f'_c = 4000$ psi

Table A.3 Values of R_u and $\frac{a}{d}$ for $f'_c = 5000$ psi

Table A.4 Values of ρ_{max}, $R_{u\,max}$, ρ_b, and ρ_{min}

Table A.5 Suggested Design Steel Ratios, ρ_s, and Comparison with Other Values

Table A.6 Minimum Thickness of Beams and One-Way Slabs

Table A.7 Minimum Beam Width (in.) (Using Stirrups)

Table A.8 Values of bd^2 (in.^{3})

Table A.9 Rectangular Sections with Compression Steel

Table A.10 Values of Modulus of Elasticity, E_c (ksi)

Table A.11 Development Length

Table A.12 Designation, Areas, Perimeter, and Weights of Standard U.S. Bars

Table A.13 Areas of Groups of Standard U.S. Bars in Square Inches

Table A.14 Areas of Bars in Slabs (Square Inches per Foot)

Table A.15 Common Styles of Welded Wire Fabric

Table A.16 Size and Pitch of Spirals

Table A.1 Values of R_u and a/d for $f'_c = 3000$ psi ($\varepsilon_t \geq 0.005$, $\phi = 0.9$ and $d = d_t$)

100	$f_y = 40$ ksi		$f_y = 50$ ksi		$f_y = 60$ ksi		$f_y = 75$ ksi	
ρ	R_u	a/d	R_u	a/d	R_u	a/d	R_u	a/d
0.2	71	0.031	88	0.039	106	0.047	131	0.059
0.3	105	0.047	131	0.059	156	0.071	192	0.089
0.4	140	0.062	173	0.078	206	0.094	254	0.118
0.5	173	0.078	214	0.098	254	0.118	310	0.148
0.6	206	0.094	254	0.118	301	0.141	368	0.177
0.7	238	0.110	293	0.138	347	0.165	421	0.207
0.8	270	0.126	332	0.157	391	0.189	475	0.236
0.9	301	0.142	369	0.177	434	0.213	524	0.266
1.0	332	0.157	406	0.196	476	0.238	572	0.295
1.1	362	0.173	441	0.216	517	0.260	620	0.325
1.2	390	0.188	476	0.235	556	0.282	615	0.319
1.3	420	0.204	510	0.255	594	0.306	($\rho_{max} = 1.082$)	
1.4	450	0.220	543	0.274	631	0.330		
1.5	476	0.236	575	0.294	667	0.353		
1.6	504	0.252	607	0.314	700	0.376		
1.7	530	0.267			615	0.319		
1.8	556	0.282			($\rho_{max} = 1.356$)			
1.9	582	0.298						
2.0	607	0.314						
2.1	630	0.330						
			615	0.319				
			($\rho_{max} = 1.624$)					
	615	0.319						
	($\rho_{max} = 2.031$)							

Note: Last values are the maximum for $\varepsilon_t = 0.005$.

Table A.2 Values of R_u and a/d for $f'_c = 4000$ psi ($\varepsilon_t \geq 0.005$, $\phi = 0.9$ and $d = d_t$)

100	$f_y = 40$ ksi		$f_y = 50$ ksi		$f_y = 60$ ksi		$f_y = 75$ ksi	
ρ	R_u	a/d	R_u	a/d	R_u	a/d	R_u	a/d
0.2	71	0.024	89	0.029	106	0.035	132	0.044
0.3	106	0.036	132	0.044	158	0.053	194	0.066
0.4	140	0.047	175	0.059	208	0.071	257	0.088
0.5	175	0.059	217	0.074	258	0.089	317	0.110
0.6	208	0.071	260	0.088	307	0.106	378	0.132
0.7	242	0.083	300	0.103	355	0.123	434	0.154
0.8	274	0.094	340	0.118	400	0.141	490	0.176
0.9	307	0.106	378	0.132	447	0.158	545	0.198
1.0	340	0.118	419	0.147	492	0.176	600	0.220
1.1	370	0.130	455	0.161	536	0.194	650	0.242
1.2	400	0.141	492	0.176	580	0.212	702	0.264
1.3	432	0.153	530	0.191	620	0.230	752	0.286
1.4	462	0.165	565	0.206	662	0.247	801	0.308
1.5	492	0.177	600	0.221	700	0.265		
1.6	522	0.188	635	0.236	742	0.282	820	0.319
							($\rho_{max} = 1.445$)	
1.7	550	0.200	670	0.250	780	0.300		
1.8	580	0.212	702	0.265	818	0.318		
1.9	607	0.224	735	0.280				
2.0	635	0.236	768	0.294				
2.1	662	0.248	800	0.309				
2.2	690	0.260			820	0.319		
2.3	717	0.271			($\rho_{max} = 1.806$)			
2.4	742	0.282						
2.5	767	0.294						
2.6	792	0.306						
2.7	817	0.318						
			820	0.319				
			($\rho_{max} = 2.167$)					
	820	0.319						
	($\rho_{max} = 2.715$)							

Note: Last values are the maximum for $\varepsilon_t = 0.005$.

Table A.3 Values of R_u and a/d for $f'_c = 5000$ psi ($\varepsilon_t \geq 0.005$, $\phi = 0.9$ and $d = d_t$)

100	$f_y = 40$ ksi		$f_y = 50$ ksi		$f_y = 60$ ksi		$f_y = 75$ ksi	
ρ	R_u	a/d	R_u	a/d	R_u	a/d	R_u	a/d
0.2	71	0.019	89	0.024	106	0.028	132	0.035
0.3	106	0.029	133	0.036	159	0.042	196	0.052
0.4	141	0.038	176	0.047	210	0.056	260	0.070
0.5	176	0.047	218	0.060	260	0.070	322	0.088
0.6	210	0.056	260	0.071	310	0.085	384	0.106
0.7	244	0.066	302	0.083	360	0.100	442	0.123
0.8	277	0.075	343	0.094	408	0.113	500	0.141
0.9	310	0.085	383	0.106	455	0.127	556	0.159
1.0	343	0.094	424	0.118	502	0.141	612	0.177
1.1	375	0.104	463	0.130	550	0.155	667	0.195
1.2	408	0.113	500	0.141	593	0.169	722	0.212
1.3	440	0.123	540	0.153	637	0.183	776	0.230
1.4	470	0.132	578	0.165	681	0.198	830	0.247
1.5	502	0.141	615	0.177	724	0.212	875	0.265
1.6	532	0.150	652	0.188	766	0.226	920	0.282
1.7	563	0.160	688	0.200	808	0.240	970	0.300
1.8	593	0.169	724	0.212	848	0.254		
1.9	623	0.179	760	0.224	890	0.268	975	0.300
2.0	652	0.188	794	0.235	927	0.282	($\rho_{max} = 1.704$)	
2.1	681	0.198	830	0.247	965	0.292		
2.2	710	0.207	862	0.259	1003	0.311		
2.3	738	0.217	894	0.271				
2.4	766	0.226	927	0.282				
2.5	794	0.235	958	0.294				
2.6	821	0.244	990	0.306	975	0.300		
					($\rho_{max} = 2.123$)			
2.7	848	0.254						
2.8	875	0.263						
2.9	900	0.272						
3.0	127	0.282						
3.1	952	0.292						
			975	0.300				
			($\rho_{max} = 2.551$)					
	975	0.300						
	($\rho_{max} = 3.18$)							

Note: Last values are the maximum for $\varepsilon_t = 0.005$.

Table A.4 Values of ρ_{max}, $R_{u\,max}$, ρ_b, ρ_{min}

$$\rho_b = 0.85\beta_1(f'_c/f_y)[87/(87+f_y)] \quad \rho_{max} = (0.003 + f_y/E_s)\rho_b/0.008 \quad R_u = \phi\rho f_y[1 - \rho f_y/1.7f'_c]$$

	f_y = 40 ksi				f_y = 50 ksi			
f'_c psi	100ρ_{max}	$R_{u\,max}$ psi	100 ρ_b	100 ρ_{min}	100 ρ_{max}	$R_{u\,max}$ psi	100 ρ_b	100 ρ_{min}
3000	2.031	615	3.71	0.50	1.624	615	2.75	0.40
4000	2.715	820	4.96	0.50	2.167	820	3.67	0.40
5000	3.180	975	5.81	0.53	2.551	975	4.32	0.42
6000	3.575	1108	6.53	0.58	2.864	1108	4.85	0.47
	f_y = 60 ksi				f_y = 75 ksi			
f'_c psi	100 ρ_{max}	$R_{u\,max}$ psi	100 ρ_b	100 ρ_{min}	100 ρ_{max}	$R_{u\,max}$ psi	100 ρ_b	100 ρ_{min}
3000	1.356	615	2.14	0.33	1.082	615	1.55	0.27
4000	1.806	820	2.85	0.33	1.445	820	2.07	0.27
5000	2.123	975	3.35	0.35	1.704	975	2.44	0.28
6000	2.389	1108	3.77	0.39	1.920	1108	2.75	0.31

Note: ρ_{max} values are for $\varepsilon_t = 0.005$ and $\phi = 0.9$.

Table A.5 Suggested Design Steel Ratios, ρ_s, and Comparison With Other Steel Ratios

f'_c psi	f_y ksi	100 ρ_b	100 ρ_{max}	100 ρ_s	R_u for ρ_s (psi)	Ratio ρ_s/ρ_b	Ratio ρ_s/ρ_{max}	Weight of ρ_s (lb/ft^3 of concrete)
3000	40	3.71	2.031	1.4	450	0.377	0.689	7
	50	2.75	1.624	1.2	476	0.436	0.739	6
	60	2.15	1.356	1.2	556	0.558	0.885	6
4000	40	4.96	2.715	1.4	462	0.282	0.516	7
	50	3.67	2.167	1.4	565	0.381	0.646	7
	60	2.85	1.806	1.4	662	0.491	0.775	7
5000	40	5.81	3.180	1.6	532	0.275	0.503	8
	50	4.32	2.551	1.6	652	0.370	0.627	8
	60	3.35	2.123	1.6	766	0.478	0.754	8

Note: ρ_{max} values are for $\varepsilon_t = 0.005$ and $\phi = 0.9$.

Table A.6 Minimum Thickness of Beams and One-Way Slabs

Member	Yield Strength f_y (ksi)	Simply Supported	One End Continuous	Both Ends Continuous	Cantilevel
Solid one-way slabs	40	$L/25$	$L/30$	$L/35$	$L/12.5$
	50	$L/22$	$L/27$	$L/31$	$L/11$
	60	$L/20$	$L/24$	$L/8$	$L/10$
Beams or ribbed	40	$L/20$	$L/23$	$L/26$	$L/10$
one-way slabs	50	$L/18$	$L/20.5$	$L/23.5$	$L/9$
	60	$L/16$	$L/18.5$	$L/21$	$L/8$

Table A.7 Minimum Beam Width (in.) (Using Stirrups)

Size of Bars	Number of Bars in Single Layer of Reinforcement							Add For Each Added Bar (in.)
	2	3	4	5	6	7	8	
No. 4	6.1	7.6	9.1	10.6	12.1	13.6	15.1	1.50
No. 5	6.3	7.9	9.6	11.2	12.8	14.4	16.1	1.63
No. 6	6.5	8.3	10.0	11.8	13.5	15.3	17.0	1.75
No. 7	6.7	8.6	10.5	12.4	14.2	16.1	18.0	1.88
No. 8	6.9	8.9	10.9	12.9	14.9	16.9	18.9	2.00
No. 9	7.3	9.5	11.8	14.0	16.3	18.6	20.8	2.26
No. 10	7.7	10.2	12.8	15.3	17.8	20.4	22.9	2.54
No. 11	8.0	10.8	13.7	16.5	19.3	22.1	24.9	2.82
No. 14	8.9	12.3	15.6	19.0	22.4	25.8	29.2	3.39
No. 18	10.5	15.0	19.5	24.0	28.6	33.1	37.6	4.51

Table A.8 Values of bd^2 (in.^{3}) $bd^2 = \left[\frac{M_u}{R_u}\left(\frac{\text{lb}\cdot\text{in.}}{\text{psi}}\right)\right]$

d (in.)	Values of b (in.)											
	6	7	8	9	10	11	12	13	14	15	16	20
4	96	112	128	144	160	176	192	208	224	240	256	320
4.5	122	142	162	182	202	223	244	264	284	305	325	405
5	150	175	200	225	250	275	300	325	350	375	400	500
5.5	182	212	242	273	303	333	364	394	424	455	485	605
6	216	252	288	324	360	396	432	468	504	540	576	720
6.5	255	297	340	382	425	467	510	552	595	637	680	850
7	294	343	392	441	490	539	588	637	686	735	784	980
8	384	448	512	576	640	704	768	832	896	960	1024	1280
9	486	567	648	729	810	891	972	1053	1134	1215	1296	1620
10	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	2000
11	726	847	968	1089	1210	1331	1452	1573	1694	1815	1936	2420
12	864	1008	1152	1296	1440	1584	1728	1872	2016	2160	2304	2880
13	1014	1183	1352	1521	1690	1859	2028	2197	2366	2535	2704	3380
14	1176	1372	1568	1764	1960	2156	2352	2548	2744	2940	3136	3920
15	1350	1575	1800	2025	2250	2475	2700	2925	3150	3375	3600	4500
16	1536	1792	2048	2304	2560	2816	3072	3328	3584	3840	4096	5120
17	1734	2023	2312	2601	2890	3179	3468	3757	4046	4335	4624	5780
18	1944	2268	2592	2916	3240	3564	3888	4212	4536	4860	5184	6480
19	2166	2527	2888	3249	3610	3971	4332	4693	5054	5415	5776	7220
20	2400	2800	3200	3600	4000	4400	4800	5200	5000	6000	6400	8000
21	2646	3087	3528	3969	4410	4851	5292	5733	6174	6615	7056	8820
22	2904	3388	3872	4356	4840	5324	5808	6292	6776	7260	7744	9680
23	3174	3703	4232	4761	5290	5819	6348	6877	7406	7935	8464	10,580
24	3456	4032	4608	5184	5760	6336	6912	7488	8064	8640	9216	11,520
28	4704	5488	6272	7056	7840	8624	9408	10,192	10,976	11,760	12,544	15,680
30	5400	6300	7200	8100	9000	9900	10,800	11,700	12,600	13,500	14,400	18,000
34	6936	8092	9248	10,404	11,560	12,716	13,872	15,028	16,184	17,340	18,496	23,120
40	9600	11,200	12,800	14,400	16,000	17,600	19,200	20,800	22,400	24,000	25,600	32,000

Table A.9 Rectangular Sections with Compression Steel Minimum Steel Percentage $100(\rho - \rho')$ for Compression Steel to Yield

$$(\rho - \rho') \geq 0.85\,\beta_1 \frac{f'_c}{f_y} \times \frac{d'}{d} \times \frac{87}{(87 - f_y)} \quad (f_c \text{ and } f_y \text{ in ksi})$$

f'_c (psi)	β_1	d'/d	f_y 40 ksi	f_y 50 ksi	f_y 60 ksi	f_y 75 ksi
3000	0.85	0.10	1.00	1.02	1.16	2.09
4000	0.85	0.10	1.33	1.35	1.55	2.78
5000	0.80	0.10	1.57	1.59	1.81	3.27
6000	0.75	0.10	1.78	1.81	2.06	3.71
3000	0.85	0.12	1.20	1.22	1.39	2.51
4000	0.85	0.12	1.60	1.62	1.86	3.34
5000	0.80	0.12	1.88	1.91	2.17	3.92
6000	0.75	0.12	2.14	2.17	2.47	4.45
3000	0.85	0.15	1.50	1.53	1.74	3.14
4000	0.85	0.15	2.00	2.03	2.33	4.17
5000	0.80	0.15	2.36	2.39	2.72	4.91
6000	0.75	0.15	2.67	2.72	3.09	5.57

Note: Minimum $(\rho - \rho')$ for any value of $d'/d = 10 \times (d'/d) \times$ value shown in table with $d'/d = 0.10$.

Table A.10 Modulus of Elasticity of concrete, E_c (Ksi)

Concrete Cylinder Strength (f'_c)	Unit Weight of Concrete (psi) 90	100	110	125	145
3000	1540	1800	2080	2520	3150
4000	1780	2090	2410	2920	3640
5000	1990	2330	2690	3260	4060
6000	2185	2560	2950	3580	4500
7000	2360	2760	3190	3870	4800
8000	2520	2950	3410	4130	5200

Note: $E_c = 33\ W^{1.5}\sqrt{f'_c}$

$E_c = 57,000\sqrt{f'_c} = W = 145$ psf (normal-weight concrete)

Table A.11(a) Values of ℓ_d/d_b for Various Values of f'_c and f_y (Tension Bars)

	$f_y = 40$ ksi				$f_y = 60$ ksi			
	≤No. 6 Bars		≥No. 7 Bars		≤No. 6 Bars		≥No. 7 Bars	
f'_c (ksi)	Conditions Met	Others	Conditions Met	Others	Conditions Met	Others	Conditions Met	Others
3	29.3	43.9	36.6	54.8	43.9	65.8	54.8	82.2
4	25.3	38.0	31.7	47.5	38.0	57.0	47.5	71.2
5	22.7	34.0	28.3	42.5	34.0	51.0	42.5	63.7
6	20.7	31.0	25.9	38.8	31.0	46.5	38.8	58.1

Table A.11(b) Development Length ℓ_d for Tension Bars and $f_y = 60$ Ksi ($\psi_t = \psi_e = \lambda = 1$)

Bar Number	Bar Diameter (in.)	Development Length ℓ_d (in.) — Tension Bars			
		$f'_c = 3$ ksi		$f'_c = 4$ ksi	
		Conditions Met	Others	Conditions Met	Others
3	0.375	17	25	15	21
4	0.500	22	33	19	29
5	0.625	28	41	24	36
6	0.750	33	50	29	43
7	0.875	48	72	42	63
8	1.000	55	83	48	72
9	1.128	62	93	54	81
10	1.270	70	105	61	92
11	1.410	78	116	68	102

Table A.12 Designations, Areas, Perimeters, and Weights of Standard U.S. Bars

Bar No.	Diameter (in.)	Cross-Sectional Area (in.^{2})	Perimeter (in.)	Unit Weight per Foot (lb)	Diameter (mm)	Area (mm^2)
2	$\frac{1}{4} = 0.250$	0.05	0.79	0.167	6.4	32
3	$\frac{3}{8} = 0.375$	0.11	1.18	0.376	9.5	71
4	$\frac{1}{2} = 0.500$	0.20	1.57	0.668	12.7	129
5	$\frac{5}{8} = 0.625$	0.31	1.96	1.043	15.9	200
6	$\frac{3}{4} = 0.750$	0.44	2.36	1.502	19.1	284
7	$\frac{7}{8} = 0.875$	0.60	2.75	2.044	22.2	387
8	$1 = 1.000$	0.79	3.14	2.670	25.4	510
9	$1\frac{1}{8} = 1.128$	1.00	3.54	3.400	28.7	645
10	$1\frac{1}{4} = 1.270$	1.27	3.99	4.303	32.3	820
11	$1\frac{3}{8} = 1.410$	1.56	4.43	5.313	35.8	1010
14	$1\frac{3}{4} = 1.693$	2.25	5.32	7.650	43.0	1450
18	$2\frac{1}{4} = 2.257$	4.00	7.09	13.600	57.3	2580

Table A.13 Areas of Groups of Standard U.S. Bars in Square Inches

Bar Number	Number of Bars											
	1	2	3	4	5	6	7	8	9	10	11	12
3	0.11	0.22	0.33	0.44	0.55	0.66	0.77	0.88	1.00	1.10	1.21	1.32
4	0.20	0.39	0.58	0.78	0.98	1.18	1.37	1.57	1.77	1.96	2.16	2.36
5	0.31	0.61	0.91	1.23	1.53	1.84	2.15	2.45	2.76	3.07	3.37	3.68
6	0.44	0.88	1.32	1.77	2.21	2.65	3.09	3.53	3.98	4.42	4.84	5.30
7	0.60	1.20	1.80	2.41	3.01	3.61	4.21	4.81	5.41	6.01	6.61	7.22
8	0.79	1.57	2.35	3.14	3.93	4.71	5.50	6.28	7.07	7.85	8.64	9.43
9	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00
10	1.27	2.53	3.79	5.06	6.33	7.59	8.86	10.12	11.39	12.66	13.92	15.19
11	1.56	3.12	4.68	6.25	7.81	9.37	10.94	12.50	14.06	15.62	17.19	18.75
14	2.25	4.50	6.75	9.00	11.25	13.50	15.75	18.00	20.25	22.50	24.75	27.00
18	4.00	8.00	12.00	16.00	20.00	24.00	28.00	32.00	36.00	40.00	44.00	48.00

Table A.14 Areas of Bars in Slabs (square inches per foot)

Spacing (in.)	Bar Number								
	3	4	5	6	7	8	9	10	11
3	0.44	0.78	1.23	1.77	2.40	3.14	4.20	5.06	6.25
$3\frac{1}{2}$	0.38	0.67	1.05	1.51	2.06	2.69	3.43	4.34	5.36
4	0.33	0.59	0.92	1.32	1.80	2.36	3.00	3.80	4.68
$4\frac{1}{2}$	0.29	0.52	0.82	1.18	1.60	2.09	2.67	3.37	4.17
5	0.26	0.47	0.74	1.06	1.44	1.88	2.40	3.04	3.75
$5\frac{1}{2}$	0.24	0.43	0.67	0.96	1.31	1.71	2.18	2.76	3.41
6	0.22	0.39	0.61	0.88	1.20	1.57	2.00	2.53	3.12
$6\frac{1}{2}$	0.20	0.36	0.57	0.82	1.11	1.45	1.85	2.34	2.89
7	0.19	0.34	0.53	0.76	1.03	1.35	1.71	2.17	2.68
$7\frac{1}{2}$	0.18	0.31	0.49	0.71	0.96	1.26	1.60	2.02	2.50
8	0.17	0.29	0.46	0.66	0.90	1.18	1.50	1.89	2.34
9	0.15	0.26	0.41	0.59	0.80	1.05	1.33	1.69	2.08
10	0.13	0.24	0.37	0.53	0.72	0.94	1.20	1.52	1.87
12	0.11	0.20	0.31	0.44	0.60	0.79	1.00	1.27	1.56

Table A.15 Common Styles of Welded Wire Fabric

Style Designation	Steel Area (in.^{2} ft)		Weight Approx. lb/100 ft^2
	Longitudinal	Transverse	
6 × 6—W1.4 × W1.4	0.03	0.03	21
6 × 6—W2 × W2	0.04	0.04	29
6 × 6—W2.9 × W2.9	0.06	0.06	42
6 × 6—W4 × W4	0.08	0.08	58
6 × 6—W5.5 × W5.5	0.11	0.11	80
4 × 4—W1.4 × W1.4	0.04	0.04	31
4 × 4—W2 × W2	0.06	0.06	43
4 × 4—W2.9 × W2.9	0.09	0.09	62
4 × 4—W4 × W4	0.12	0.12	86

Table A.16 Size and Pitch of Spirals

f_y (ksi)	Diameter of Column (in.)	Outside to Outside of Spiral (in.)	f'_c(psi)		
			3000	4000	5000
40	14, 15	11, 12	$\frac{3}{8}-1\frac{3}{4}$	$\frac{1}{2}-2\frac{1}{2}$	$\frac{1}{2}-1\frac{3}{4}$
	16	13	$\frac{3}{8}-1\frac{3}{4}$	$\frac{1}{2}-2\frac{1}{2}$	$\frac{1}{2}-2$
	17–19	14–16	$\frac{3}{8}-1\frac{3}{4}$	$\frac{1}{2}-2\frac{1}{2}$	$\frac{1}{2}-2$
	20–23	17–20	$\frac{3}{8}-1\frac{3}{4}$	$\frac{1}{2}-2\frac{1}{2}$	$\frac{1}{2}-2$
	24–30	21–27	$\frac{3}{8}-2$	$\frac{1}{2}-2\frac{1}{2}$	$\frac{1}{2}-2$
60	14, 15	11, 12	$\frac{3}{8}-2\frac{3}{4}$	$\frac{3}{8}-2$	$\frac{1}{2}-2\frac{3}{4}$
	16–23	13–20	$\frac{3}{8}-2\frac{3}{4}$	$\frac{3}{8}-2$	$\frac{1}{2}-2\frac{3}{4}$
	24–29	21–26	$\frac{3}{8}-3$	$\frac{3}{8}-2\frac{1}{4}$	$\frac{1}{2}-3$
	30	27	$\frac{3}{8}-3$	$\frac{3}{8}-2\frac{1}{4}$	$\frac{1}{2}-3\frac{1}{4}$

APPENDIX B

DESIGN TABLES (SI UNITS)

Table B.1 Values of R_u and $\frac{a}{d}$ for $f'_c = 21$ MPa (R_u in MPa)

Table B.2 Values of R_u and $\frac{a}{d}$ for $f'_c = 28$ MPa (R_u in MPa)

Table B.3 Values of R_u and $\frac{a}{d}$ for $f'_c = 35$ MPa (R_u in MPa)

Table B.4 Values of $\rho_{\max}$, $R_{u\max}$, ρ_b, and $\rho_{\min}$

Table B.5 Suggested Design Steel Ratios, ρ_s, and Comparison with Other Steel Ratios

Table B.6 Minimum Thickness of Beams and One-way Slabs

Table B.7 Rectangular Sections with Compression Steel

Table B.8 Values of Modulus of Elasticity, E_c

Table B.9 Development Length

Table B.10 Designation, Areas, and Mass of Bars

Table B.11 ASTM Standard Metric Reinforcing Bars

Table B.12 Areas of Group of Bars (mm^2)

Table B.1 Values of R_u and a/d for $f'_c = 21$ MPa (R_u in MPa), ($\varepsilon_t \geq 0.005$, $\phi = 0.9$ and $d = d_t$)

100	$f_y = 280$ MPa		$f_y = 350$ MPa		$f_y = 420$ MPa		$f_y = 520$ MPa	
ρ	R_u	a/d	R_u	a/d	R_u	a/d	R_u	a/d
0.2	0.50	0.031	0.62	0.039	0.75	0.047	0.92	0.059
0.3	0.74	0.046	0.92	0.059	1.10	0.071	1.35	0.089
0.4	0.98	0.062	1.22	0.078	1.45	0.094	1.79	0.118
0.5	1.21	0.078	1.50	0.098	1.79	0.118	2.18	0.148
0.6	1.45	0.094	1.79	0.118	2.12	0.141	2.59	0.177
0.7	1.68	0.110	2.06	0.138	2.44	0.165	2.96	0.207
0.8	1.90	0.126	2.33	0.157	2.75	0.189	3.34	0.236
0.9	2.12	0.142	2.59	0.177	3.05	0.213	3.68	0.266
1.0	2.33	0.157	2.84	0.196	3.35	0.238	4.02	0.295
1.1	2.55	0.173	3.10	0.216	3.64	0.260	4.36	0.325
1.2	2.74	0.188	3.35	0.235	3.91	0.280	4.32	0.319
1.3	2.95	0.204	3.59	0.255	4.18	0.306	($\rho_{max} = 1.085$)	
1.4	3.16	0.220	3.82	0.274	4.44	0.330		
1.5	3.35	0.236	4.04	0.294				
1.6	3.54	0.252	4.27	0.314				
1.7	3.73	0.267			4.32	0.319		
1.8	3.91	0.282			($\rho_{max} = 1.37$)			
1.9	4.09	0.298						
2.0	4.27	0.314						
2.1	4.43	0.330	4.32	0.319				
			($\rho_{max} = 1.63$)					
	4.32	0.319						
	($\rho_{max} = 2.04$)							

Note: Last values are the maximum for $\varepsilon_t = 0.005$.

Table B.2 Values of R_u and a/d for $f'_c = 28$ MPa (R_u in MPa), ($\varepsilon_t \geq 0.005$, $\phi = 0.9$ and $d = d_t$)

100	$f_y = 280$ MPa		$f_y = 350$ MPa		$f_y = 420$ MPa		$f_y = 520$ MPa	
ρ	R_u	a/d	R_u	a/d	R_u	a/d	R_u	a/d
0.2	0.50	0.024	0.63	0.029	0.75	0.025	0.93	0.044
0.3	0.74	0.036	0.93	0.044	1.11	0.053	1.36	0.066
0.4	0.98	0.047	1.23	0.059	1.46	0.071	1.81	0.088
0.5	1.23	0.059	1.53	0.074	1.81	0.089	2.23	0.110
0.6	1.46	0.071	1.83	0.088	2.16	0.106	2.66	0.132
0.7	1.70	0.083	2.11	0.103	2.50	0.123	3.05	0.154
0.8	1.93	0.094	2.39	0.118	2.81	0.141	3.45	0.176
0.9	2.16	0.106	2.66	0.132	2.14	0.158	3.83	0.198
1.0	2.39	0.118	2.95	0.147	3.46	0.176	4.22	0.220
1.1	2.60	0.130	3.20	0.161	3.77	0.194	4.57	0.242
1.2	2.81	0.141	3.46	0.176	4.08	0.212	4.94	0.264
1.3	3.04	0.153	3.73	0.191	4.36	0.230	5.29	0.286
1.4	3.25	0.165	3.97	0.206	4.65	0.247		
1.5	3.46	0.177	4.22	0.221	4.92	0.265		
1.6	3.67	0.188	4.46	0.236	5.22	0.282	5.77	0.319
							($\rho_{max} = 1.45$)	
1.7	3.87	0.200	4.71	0.250	5.48	0.300		
1.8	4.08	0.212	4.94	0.265	5.75	0.318		
1.9	4.27	0.224	5.17	0.280				
2.0	4.46	0.236	5.40	0.294				
2.1	4.65	0.248	5.62	0.309				
2.2	4.85	0.260			5.77	0.319		
2.3	5.04	0.271			($\rho_{max} = 1.82$)			
2.4	5.22	0.282						
2.5	5.39	0.294						
2.6	5.57	0.306						
2.7	5.74	0.318						
2.8	5.92	0.330	5.77	0.319				
			($\rho_{max} = 2.18$)					
	5.77	0.319						
	($\rho_{max} = 2.73$)							

Note: Last values are the maximum for $\varepsilon_t = 0.005$.

Table B.3 Values of R_u and a/d for $f'_c = 35$ MPa (R_u in MPa), ($\varepsilon_t \geq 0.005$, $\phi = 0.9$ and $d = d_t$)

100	$f_y = 350$ MPa		$f_y = 420$ MPa		$f_y = 520$ MPa	
ρ	R_u	a/d	R_u	a/d	R_u	a/d
0.2	0.63	0.024	0.75	0.028	0.93	0.035
0.3	0.93	0.036	1.12	0.042	1.38	0.052
0.4	1.24	0.047	1.48	0.056	1.83	0.070
0.5	1.53	0.060	1.83	0.070	2.26	0.088
0.6	1.83	0.071	2.18	0.085	2.70	0.106
0.7	2.12	0.083	2.53	0.100	3.11	0.123
0.8	2.41	0.094	2.87	0.113	3.52	0.141
0.9	2.69	0.106	3.20	0.127	3.91	0.159
1.0	2.98	0.118	3.53	0.141	4.30	0.177
1.1	3.26	0.130	3.87	0.155	4.69	0.195
1.2	3.52	0.141	4.17	0.169	5.08	0.212
1.3	3.80	0.153	4.48	0.183	5.46	0.230
1.4	4.06	0.165	4.79	0.198	5.84	0.247
1.5	4.32	0.177	5.09	0.212	6.15	0.265
1.6	4.58	0.188	5.39	0.226	6.47	0.282
1.7	4.84	0.200	5.68	0.240	6.82	0.300
1.8	5.09	0.212	5.96	0.254		
1.9	5.34	0.224	6.26	0.268	6.85	0.3192
2.0	5.58	0.235	6.52	0.282	($\rho_{max} = 1.71$)	
2.1	5.84	0.247	6.78	0.296		
2.2	6.06	0.259				
2.3	6.29	0.271				
2.4	6.52	0.282				
2.5	6.74	0.294				
			6.85	0.319		
			($\rho_{max} = 2.16$)			
	6.85	0.319				
	($\rho_{max} = 2.57$)					

Note: Last values are the maximum for $\varepsilon_t = 0.005$.

Table B.4 Values of ρ_{max}, $R_{u\,max}$, ρ_b, ρ_{min}

$$\rho_b = 0.85\beta_1(f'_c/f_y)[87/(87+f_y)] \quad \rho_{max} = (0.003 + f_y/E_s)\rho_b/0.008 \quad R_u = \phi\rho f_y[1 - \rho f_y/1.7f'_c]$$

	f_y = 280 MPa				f_y = 350 MPa			
f'_c MPa	100 ρ_{max}	$R_{u\,max}$ MPa	100 ρ_b	100 ρ_{min}	100 ρ_{max}	$R_{u\,max}$ MPa	100 ρ_b	100 ρ_{min}
21	2.031	4.32	3.71	0.50	1.624	4.32	2.75	0.40
28	2.715	5.77	4.96	0.50	2.167	5.77	3.67	0.40
35	3.180	6.85	5.81	0.53	2.551	6.85	4.32	0.42
42	3.575	7.78	6.53	0.58	2.864	7.78	4.85	0.47
	f_y = 420 MPa				f_y = 525 MPa			
f'_c MPa	100 ρ_{max}	$R_{u\,max}$ MPa	100 ρ_b	100 ρ_{min}	100 ρ_{max}	$R_{u\,max}$ MPa	100 ρ_b	100 ρ_{min}
21	1.356	4.32	2.14	0.33	1.082	4.32	1.55	0.27
28	1.806	5.77	2.85	0.33	1.445	5.77	2.07	0.27
35	2.123	6.85	3.35	0.35	1.704	6.85	2.44	0.28
42	2.389	7.78	3.77	0.39	1.920	7.78	2.75	0.31

Note: ρ_{max} values are for $\varepsilon_t = 0.005$ and $\phi = 0.9$.

Table B.5 Suggested Design Steel Ratios, ρ_s, and Comparison with Other Steel Ratios

f'_c MPa	f_y MPa	100 ρ_b	100 ρ_{max}	100 ρ_s	R_u for ρ_s (MPa)	Ratio ρ_s/ρ_b	Ratio ρ_s/ρ_{max}	Weight of ρ_s (kg/m^3 of concrete)
21	280	3.71	2.04	1.4	3.16	0.377	0.689	112
	350	2.75	1.63	1.2	3.35	0.436	0.739	96
	420	2.15	1.37	1.2	3.91	0.558	0.885	96
28	280	4.96	2.73	1.4	3.25	0.282	0.516	112
	350	3.67	2.18	1.4	3.97	0.381	0.646	112
	420	2.85	1.81	1.4	4.65	0.491	0.775	112
35	280	5.81	3.20	1.6	3.72	0.275	0.503	128
	350	4.32	2.57	1.6	4.58	0.370	0.627	128
	420	3.35	2.16	1.6	5.39	0.478	0.754	128

Note: ρ_{max} values are for $\varepsilon_t = 0.005$ and $\phi = 0.9$.

Table B.6 Minimum Thickness of Beams and One-Way Slabs

Member	Yield Strength f_y (MPa)	Simply Supported	One End Continuous	Both Ends Continuous	Cantilever
Solid one-way slabs	280	$L/25$	$L/30$	$L/35$	$L/12.5$
	350	$L/22$	$L/27$	$L/31$	$L/11$
	420	$L/20$	$L/24$	$L/28$	$L/10$
Beams or ribbed one-way slabs	280	$L/20$	$L/23$	$L/26$	$L/10$
	350	$L/18$	$L/20.5$	$L/23.5$	$L/9$
	420	$L/16$	$L/18.5$	$L/21$	$L/8$

Table B.7 Rectangular Sections with Compression Steel. Minimum Steel Percentage $100(\rho - \rho')$ for Compression Steel to Yield

$$(\rho - \rho') \geq 0.85\beta_1\left(\frac{f'_c}{f_y}\right) \times \left(\frac{d'}{d}\right) \times \frac{600}{(600 - f_y)}, (f_y, f'_c \text{ in MPa})$$

f'_c MPa	β_1	d'/d	f_y 300 MPa	f_y 400 MPa	f_y 500 MPa
21	0.85	0.10	1.20	1.35	2.16
28	0.85	0.10	1.45	1.63	2.60
35	0.80	0.10	1.59	1.80	2.85
42	0.75	0.10	1.70	1.91	3.06
21	0.85	0.12	1.45	1.63	2.60
28	0.85	0.12	1.73	1.95	3.12
35	0.80	0.12	2.02	2.27	3.64
42	0.75	0.12	2.04	2.29	3.67
21	0.85	0.15	1.81	2.03	3.25
28	0.85	0.15	2.17	2.44	3.90
35	0.80	0.15	2.38	2.68	4.28
42	0.75	0.15	2.55	2.87	4.59

Note: Minimum $(\rho - \rho')$ for any value of $d'/d = 10 \times (d'/d) \times$ value shown in table with $d'/d = 0.10$.

Table B.8 Modulus of Elasticity of Normal-Weight Concrete

General: $E_c = 0.043W^{1.5}\sqrt{f'_c}$ MPa
For Normal-Weight Concrete, $W = 2350$ kg/m^3:
$E_c = 4730\sqrt{f'_c}$ MPa

f'_c MPa	E_c (kN/mm^2)
17.5	20.0
21.0	22.5
28.0	25.0
35.0	29.0
42.0	32.0
49.0	33.5
56.0	36.5

Table B.9(a) Values of ℓ_d/d_b for Various Values of f'_c and f_y (Tension Bars)

	$f_y = 300$ MPa				$f_y = 400$ MPa			
	≤20 M Bars		≥25 M Bars		≤20 M Bars		≥25 M Bars	
f'_c MPa	Conditions Met	Others	Conditions Met	Others	Conditions Met	Others	Conditions Met	Others
20	34.0	50.5	42.0	63.0	45.0	67.0	56.0	84.0
30	27.5	41.5	34.5	51.5	36.5	55.0	46.0	68.5
35	25.5	38.5	32.0	47.5	34.0	51.0	42.5	63.5
40	23.5	35.5	29.5	44.5	31.5	47.5	39.5	59.5

Table B.9(b) Development Length ℓ_d/d_b for Tension Bars and $f_y = 400$ MPa ($\alpha = \beta = \lambda = 1.0$)

		Development Length ℓ_d/d_d (mm) — Tension Bars			
		$f'_c = 20$ MPa		$f'_c = 30$ MPa	
Bar Number	Bar Diameter (mm)	Conditions Met	Others	Conditions Met	Others
10M	11.3	Ê510	Ê765	Ê415	Ê620
15M	16.0	Ê720	1080	Ê585	Ê875
20M	19.5	Ê880	1320	Ê710	1070
25M	25.2	1410	2120	1160	1740
30M	29.9	1675	2510	1375	2065
35M	35.7	2000	3000	1640	2465

Table B.10 Designations, Areas, and Mass of Bars

	Nominal Dimensions		
Bar No.	Diameter (mm)	Area (mm^2)	Mass (kg/m)
#10	9.5	71	0.560
#13	12.7	129	0.994
#16	15.9	199	1.552
#19	19.1	284	2.235
#22	22.2	387	3.042
#25	25.4	510	3.973
#29	28.7	645	5.060
#32	32.3	819	6.404
#36	35.8	1006	7.907
#43	43.0	1452	11.38
#57	57.3	2581	20.24

ASTM A615 M Grade 300 is limited to sizes #10 through #19; otherwise, grades are 400 or 500 MPa. (These bars are soft conversion of #3 to #18 in U.S. customary units.)

Table B.11 ASTM Standard Metric Reinforcing Bars

	Nominal Dimensions		
Bar-size Designation (number)	Diameter (mm)	Area (mm^2)	Mass (kg/m)
10M	11.3	100	0.785
15M	16.0	200	1.570
20M	19.5	300	2.355
25M	25.2	500	3.925
30M	29.9	700	5.495
35M	35.7	1000	7.850
45M	43.7	1500	11.775
55M	56.4	2500	19.625

ASTM A615M grade 300 is limited to size 10 M through 20 M; otherwise, grades are 400 or 500 MPa.

Table B.12 Areas of Group of Bars (mm^2) — Metric

Bar No. Metric	Number of Bars									
	1	2	3	4	5	6	7	8	9	10
#10	71	142	213	384	355	426	497	568	639	710
#13	129	258	387	516	645	774	903	1032	1161	1290
#16	199	398	597	796	995	1194	1393	1592	1791	1990
#19	284	568	852	1136	1420	1704	1988	2272	2556	2840
#22	387	774	1161	1548	1935	2322	2709	3096	3483	3870
#25	510	1020	1530	2040	2550	3060	3570	4080	4590	5100
#29	645	1290	1935	2580	3225	3870	4515	5160	5805	6450
#32	819	1638	2457	3276	4095	4914	5733	6552	7371	8190
#36	1006	2012	3018	4024	5030	6036	7042	8048	9054	10060

APPENDIX C

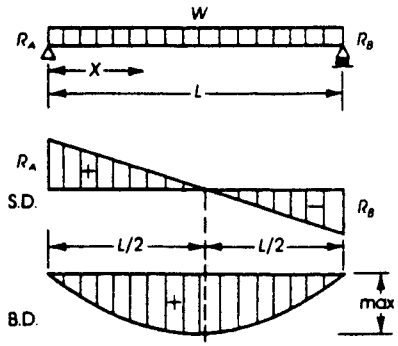
STRUCTURAL AIDS

Table C.1 Simple Beams (Cases 1–20)
Table C.2 Cantilever Beams (Cases 21–24)
Table C.3 Propped Beams (Cases 25–32)
Table C.4 Fixed End Beams (Cases 33–40)
Table C.5 Moments in Two Unequal Spans and Values of the Coefficient K (Cases 1–3)
Table C.6 Moments in Three Unequal Spans and Values of the Coefficient K (Cases 4–6)
Table C.7 Maximum and Minimum Moments in Equal Spans Continuous Beams (Cases 7–8)
Table C.8 Moments in Unequal Spans Continuous Beams Subjected to Unequal Loads (Case 9)

Note: S.S. stands for shearing force diagram. B.D. stands for bending moment diagram.
Bending moments are drawn on the tension sides of beams.
Moments, shearing forces and deflections, for any combination of loadings, are obtained by superposition.

Table C.1 Simple Beams (w = Load/Unit Length)

1. Uniform load



W = total load = wL

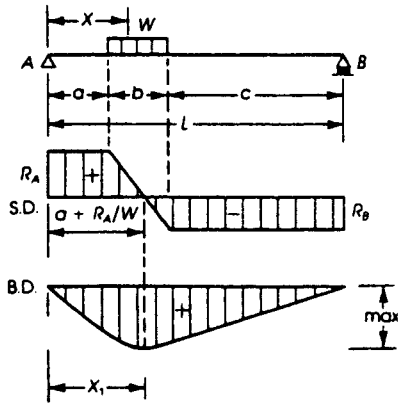
$$R_A = R_B = V_A = V_B = \frac{W}{2}$$

$$M_x = \frac{Wx}{2}\left(1 - \frac{x}{L}\right)$$

$$M_{\max} = \frac{WL}{8} \quad \text{(at center)}$$

$$\Delta_{\max} = \frac{5}{384} \times \frac{WL^3}{EI} \quad \text{(at center)}$$

2. Uniform partial load



W = total load = wb

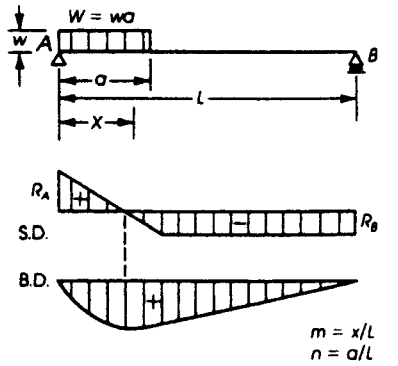
$$R_A = V_A = \frac{W}{L}\left(\frac{b}{2} + c\right)$$

$$R_B = V_B = \frac{W}{L}\left(\frac{b}{2} + a\right)$$

$$M_{\max} = \frac{W}{2b}(x^2 - a^2) \quad \text{when } x = a + \frac{R_A b}{W}$$

$$\Delta_{\max} = \frac{W}{384EI}(8L^3 - 4Lb^2 + b^3) \quad \text{when } a = c$$

3. Uniform partial load at one end



W = total load = wa

$$R_A = V_A = W\left(1 - \frac{a}{2L}\right)$$

$$R_B = V_B = \frac{Wa}{2L}$$

$$M_{\max} = \frac{Wa}{2}\left(1 - \frac{a}{2L}\right)^2 \quad \text{when } x = a\left(1 - \frac{a}{2L}\right)$$

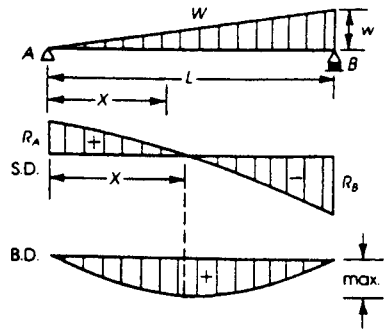
$$\Delta = \frac{WL^4}{24aEI}\, n^2[2m^3 - 6m^2 + m(4 + n^2) - n^2]$$

when $x \geq a$

$$\Delta = \frac{WL^4 m}{24aEI}\,[n^2(2 - n)^2 - 2nm^2(2 - n) + m^3] \quad \text{when } x < a$$

Table C.1 *(continued)*

4. Triangular load on span with maximum value at one end



$$W = \text{total load} = \frac{wL}{2}$$

$$R_A = V_A = \frac{W}{3}$$

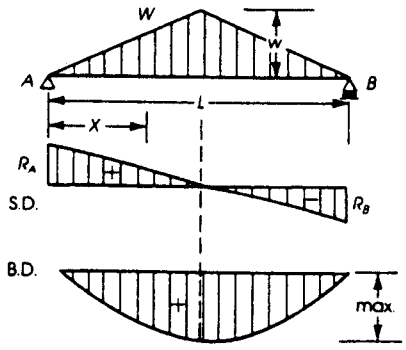
$$R_B = V_B = \frac{2W}{3}$$

$$M_x = \frac{Wx}{3}\left(1 - \frac{x^2}{L^2}\right)$$

$$M_{\max} = 0.128WL \quad \text{when } x = 0.5774L$$

$$\Delta_{\max} = \frac{0.01304WL^3}{EI} \quad \text{when } x = 0.5193L$$

5. Triangular load with maximum value at midspan



$$W = \text{total load} = \frac{wL}{2}$$

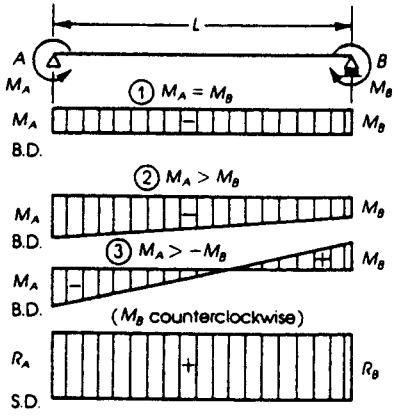
$$R_A = R_B = V_A = V_B = \frac{W}{2}$$

$$M_x = Wx\left(\frac{1}{2} - \frac{2x^2}{3L^2}\right)$$

$$M_{\max} = \frac{WL}{6} \quad \text{(at midspan)}$$

$$\Delta_{\max} = \frac{WL^3}{60EI} \quad \text{(at midspan)}$$

6. Moments at ends



$$R_A = R_B = V_A = V_B = \frac{M_A - M_B}{L}$$

$$\Delta_{\max} \text{ (at midspan)} = \frac{ML^2}{8EI} \quad \text{when } M_A = M_B$$

$$\Delta \text{ (at midspan)} = \frac{M_A L^2}{16EI} \quad \text{when } M_B = 0$$

$$\Delta \text{ (at midspan)} = \frac{M_B L^2}{16EI} \quad \text{when } M_A = 0$$

(continued)

Table C.1 *(continued)*

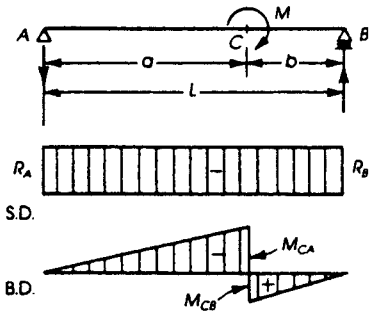
7. External moment at any point

$$R_A = -R_B = V_A = V_B = \frac{M}{L}$$

$$M_{CA} = \frac{Ma}{L}$$

$$M_{CB} = \frac{Mb}{L}$$

$$\Delta_c = \frac{-Mab}{3EIL}\ (a - b)$$

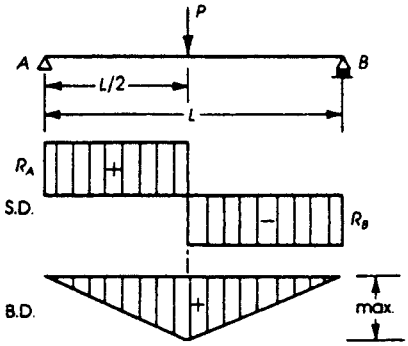


8. Concentrated load at midspan

$$R_A = R_B = V_A = V_B = \frac{P}{2}$$

$$M_{\max} = \frac{PL}{4} \quad \text{(at midspan)}$$

$$\Delta_{\max} = \frac{PL^3}{48EI} \quad \text{(at midspan)}$$



9. Concentrated load at any point

$$R_A = V_A = \frac{Pb}{L}$$

$$R_B = V_B = \frac{Pa}{L}$$

$$M_{\max} = \frac{Pab}{L} \quad \text{(at point load)}$$

$$\Delta_c = \frac{Pa^2b^2}{3EIL} \quad \text{(at point load)}$$

$$\Delta_{\max} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right] \quad \text{(when } a \geq b\text{)}$$

at $x = \sqrt{a(b + L)/3}$

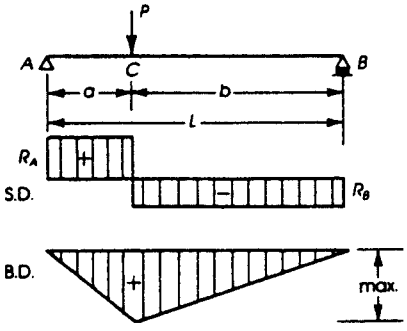


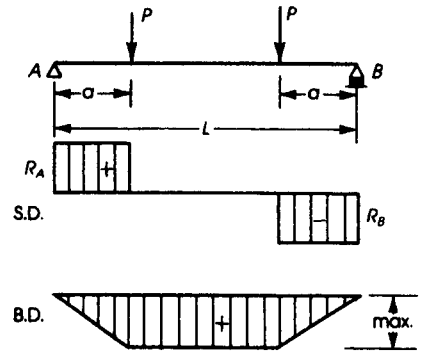
Table C.1 *(continued)*

10. Two symmetrical concentrated loads

$$R_A = R_B = V_A = V_B = P$$

$$M_{\max} = Pa$$

$$\Delta_{\max} = \frac{PL^3}{6EI}\left[\frac{3a}{4L} - \left(\frac{a}{L}\right)^3\right] \quad \text{(at midspan)}$$



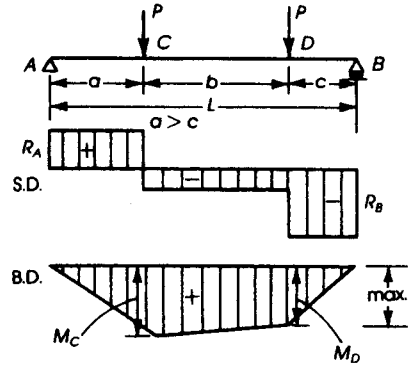
11. Two concentrated loads

$$R_A = V_A = \frac{P(b+2c)}{L}$$

$$R_B = V_B = \frac{P(b+2a)}{L}$$

$$M_C = \frac{Pa(b+2c)}{L}$$

$$M_D = \frac{Pc(b+2a)}{L}$$

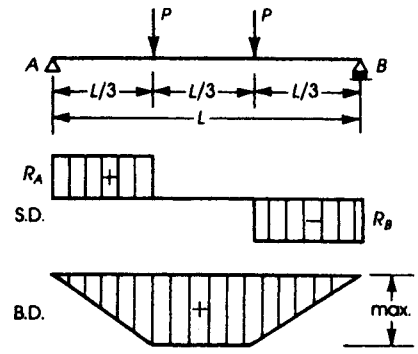


12. Two concentrated loads at one-third points

$$R_A = R_B = V_A = V_B = P$$

$$M_{\max} = \frac{PL}{3}$$

$$\Delta_{\max} = \frac{23PL^3}{648EI} \quad \text{(at midspan)}$$



(continued)

Table C.1 *(continued)*

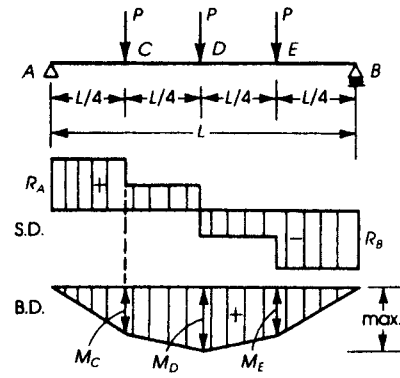
13. Three concentrated loads at one-fourth points

$$R_A = R_B = V_A = V_B = \frac{3P}{2}$$

$$M_C = M_E = \frac{3PL}{8}$$

$$M_D = \frac{PL}{2}$$

$$\Delta_{max} = \frac{19PL^3}{384EI} \quad \text{(at midspan)}$$



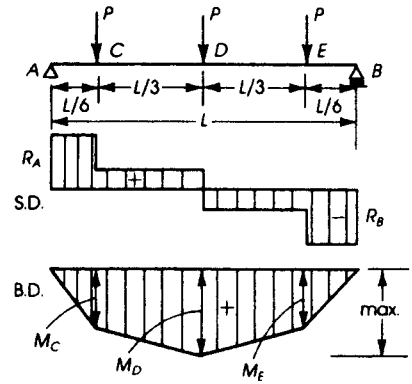
14. Three concentrated loads as shown

$$R_A = R_B = V_A = V_B = \frac{3P}{2}$$

$$M_C = M_E = \frac{PL}{4}$$

$$M_D = \frac{5PL}{12}$$

$$\Delta_{max} = \frac{53PL^3}{1296EI} \quad \text{(at midspan)}$$



15. Uniformly distributed load and variable end moments

W = total load = wL

$$R_A = V_A = \frac{W}{2} + \frac{M_1 - M_2}{L}$$

$$R_B = V_B = \frac{W}{2} - \frac{M_1 - M_2}{L}$$

$$M_3 = \frac{WL}{8} - \frac{M_1 + M_2}{2} + \frac{(M_1 - M_2)^2}{2WL}$$

$$\text{at } x = \frac{L}{2} + \frac{M_1 - M_2}{W}$$

$$\Delta_x = \frac{Wx}{24EIL}\left[x^3 - \left(2L + \frac{4M_1}{W} - \frac{4M_2}{W}\right)x^2 + \frac{12M_1L}{W}x + L^3 - \frac{8M_1L^2}{W} - \frac{4M_2L^2}{W}\right]$$

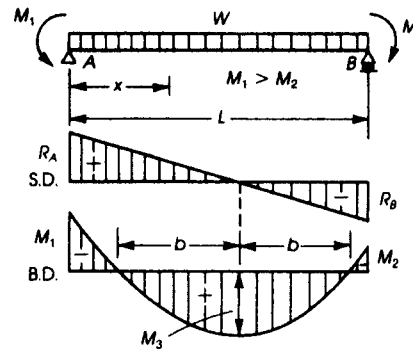
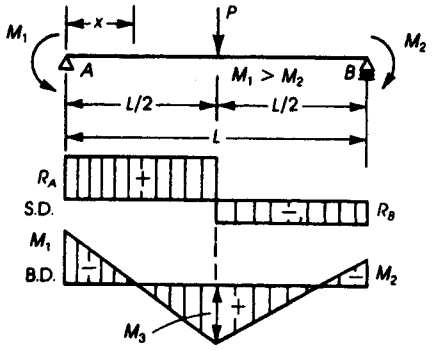


Table C.1 *(continued)*

16. Concentrated load at center and variable end moments



$$R_A = V_A = \frac{P}{2} + \frac{M_1 - M_2}{L}$$

$$R_B = V_B = \frac{P}{2} - \frac{M_1 - M_2}{L}$$

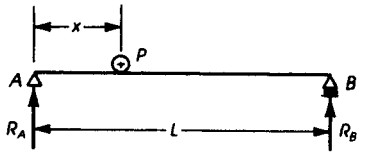
$$M_3 = \frac{PL}{4} - \frac{M_1 + M_2}{2} \quad \text{(at midspan)}$$

$$M_x = \left(\frac{P}{2} + \frac{M_1 - M_2}{L}\right) x - M_1 \quad \text{when } x < \frac{L}{2}$$

$$M_x = \frac{P}{2}(L - x) + \frac{(M_1 - M_2)}{L} x - M_1 \quad \text{when } x > \frac{L}{2}$$

$$\Delta_x = \frac{Px}{48EI}\left[3L^2 - 4x^2 - \frac{8(L-x)}{PL}\{M_1(2L - x) + M_2(L + x)\}\right] \quad \text{when } x < \frac{L}{2}$$

17. One concentrated moving load



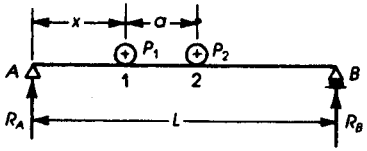
R_A max $= V_A$ max $= P$ at $x = 0$

R_B max $= V_A$ max $= P$ at $x = L$

$$M_{\max} = \frac{PL}{4} \quad \text{at } x = \frac{L}{2}$$

$$M_x = \frac{P}{L}(L - x)x$$

18. Two equal concentrated moving loads



$$R_A \text{ max} = V_A \text{ max} = P\left(2 - \frac{a}{L}\right) \quad \text{at } x = 0$$

$$M_{\max} = \frac{P}{2L}\left(L - \frac{a}{2}\right)^2$$

when $a < 0.586L$ under load 1 at $x = \frac{1}{2}\left(L - \frac{a}{2}\right)$

$$M_{\max} = \frac{PL}{4} \quad \text{when } a > 0.5L \text{ with one load at midspan}$$

(continued)

Table C.1 *(continued)*

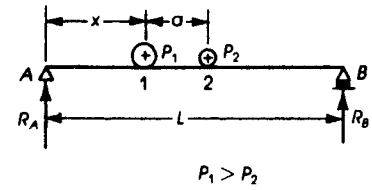
19. Two unequal concentrated moving loads

$$R_A \max = V_A \max = P_1 + P_2\left(\frac{L-a}{L}\right) \quad \text{at } x = 0$$

$$M_{\max} = (P_1 + P_2)\frac{x^2}{L}$$

under load P_1 at $x = \frac{1}{2}\left(L - \frac{P_2 a}{P_1 + P_2}\right)$

$$M_{\max} = \frac{P_1 L}{4}$$ may occur with larger load at center of span and other load off span



20. General rules for simple beams carrying moving concentrated loads $V_{\max}$ occurs at one support and other loads on span (trial method) For $M_{\max}$:place center line of beam midway between center of gravity of loads and nearest concentrated load. $M_{\max}$ occurs under this load (here P_1)

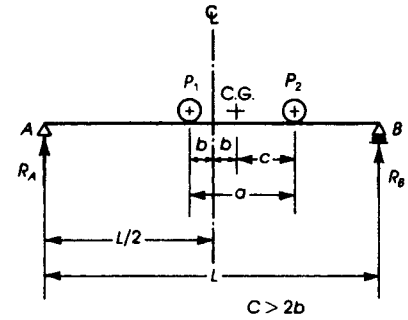


Table C.2 Cantilever Beams

21. Uniform load

W = total load = wL

$$R_A = V_A = W$$

$$M_A = \frac{WL}{2} \quad \text{(at support A)}$$

$$M_x = \frac{Wx^2}{2L}$$

$$\Delta_B \max = \frac{WL^3}{8EI}$$

$$\Delta_x = \frac{W}{24EIL}(x^4 - 4L^3x + 3L^4)$$

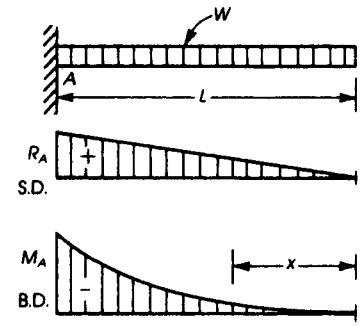


Table C.2 *(continued)*

22. Partial uniform load starting from support

W = total load = wa

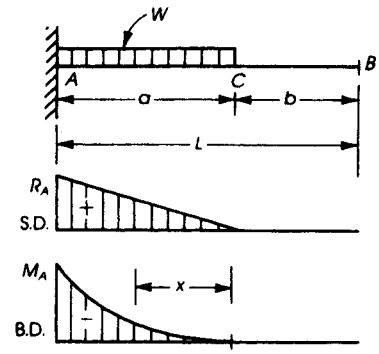
$R_A = V_A = W$

$M_A = \dfrac{Wa}{2}$ (at support A)

$M_x = \dfrac{Wx^2}{2a}$

$\Delta_C = \dfrac{Wa^3}{8EI}$

$\Delta_B \text{max} = \dfrac{Wa^3}{8EI}\left(1 + \dfrac{4b}{3a}\right)$



23. Concentrated load

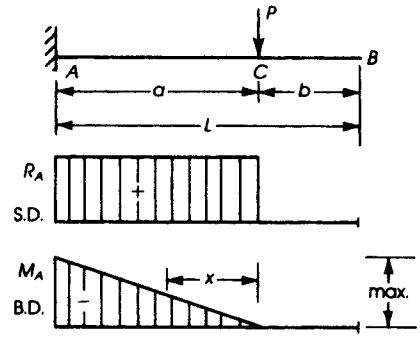
$R_A = V_A = P$

$M_{\text{max}} = Pa$ (at support A)

$M_x = Px$

$\Delta_C = Pa^3/3EI$

$\Delta_B \text{ max} = \dfrac{Pa^3}{3EI}\left(1 + \dfrac{3b}{2a}\right)$ (at free end)



24. Concentrated load at free end

$R_A = V_A = P$

$M_{\text{max}} = PL$ (at A)

$M_x = Px$

$\Delta_B \text{ max} = \dfrac{PL^3}{3EI}$

$\Delta_x = \dfrac{P}{6EI}(2L^3 - 3L^2x + x^3)$

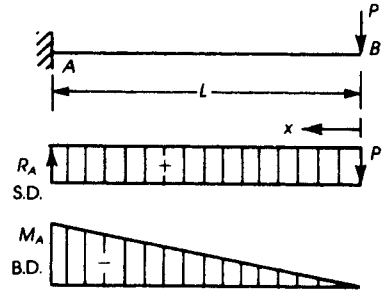
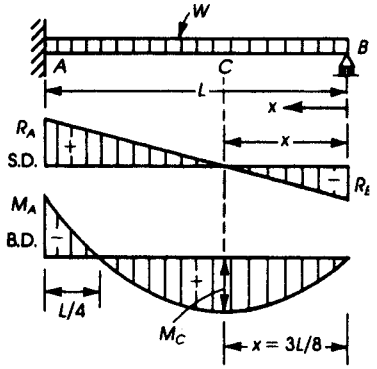


Table C.3 Propped Beams

25. Uniform load



W = total load $= wL$

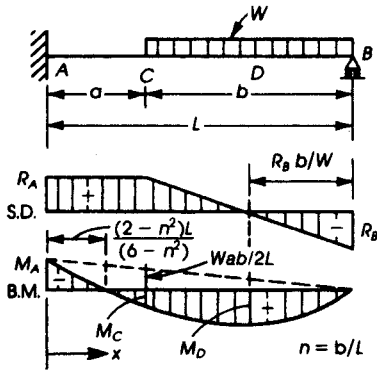
$$R_A = V_A = \frac{5W}{8} \quad R_B = V_B = \frac{3W}{8}$$

$$M_A = -\frac{WL}{8} \quad M_C = \frac{9WL}{128} \left(\text{at} x = \frac{3}{8}L\right)$$

$$\Delta_x = \frac{WL^3}{48EI}(m - 3m^3 + 2m^4) \text{where} m = \frac{x}{L}$$

$$\Delta_{\max} = \frac{WL^3}{185EI} \text{at a distance} x = 0.4215L \text{(from support B)}$$

26. Partial uniform load starting from hinged support



$$W = wb \quad n = \frac{b}{L}$$

$$R_A = V_A = \frac{Wn}{8}(6 - n^2)$$

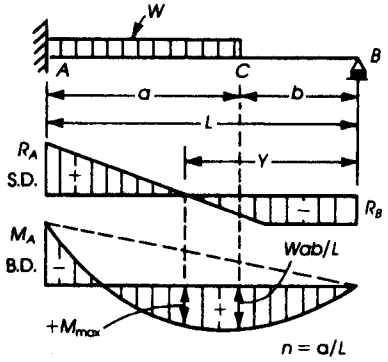
$$R_B = V_B = \frac{W}{8}(n^3 - 6n + 8)$$

$$M_A = -\frac{Wb}{8}(2 - n^2) \quad M_C = \frac{Wb}{8}(6n - n^3 - 4)$$

$$\Delta_x = \frac{WbL^2}{48EI}[(n^2 - 6)m^3 - (3n^2 - 6)m^2] \text{ when } x \le a$$

$$\Delta_x = \frac{WL^4}{48bEI}[2P^4 - p^3n(n^3 - 6n + 8) + Pn^2(3n^2 - 8n + 6)] \text{ when } x \ge a \text{ and } P = \frac{L - x}{L}$$

27. Partial uniform load starting from fixed end



$$W = wa \quad n = \frac{a}{L}$$

$$R_A = V_A = \frac{W}{8}[8 - n^2(4 - n)]$$

$$R_B = V_B = \frac{Wn^2}{8}(4 - n) \quad Y = b + an^2(4 - n)$$

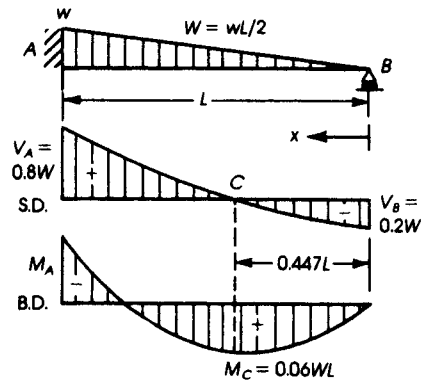
$$M_A = -\frac{W_a}{8}(2 - n)^2$$

$$M_{\max} = \frac{Wa}{8}\left\{-\frac{[8 - n^2(4 - n)]^2}{16} + 4 - n(4 - n)\right\}$$

$$\Delta_C = \frac{Wa^3}{48EI}(6 - 12n + 7n^2 - n^3)$$

Table C.3 *(continued)*

28. Triangular load on all span L



$$W = \text{total load} = \frac{wL}{2}$$

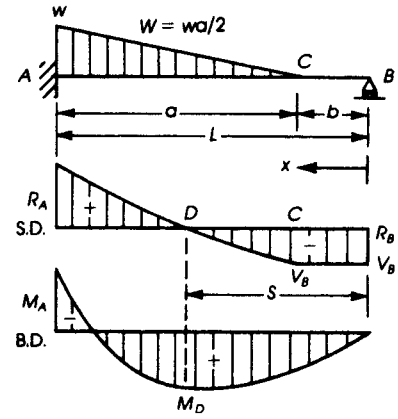
$$R_A = V_A = \frac{4}{5}W \quad R_B = \frac{W}{5} = V_B$$

$$M_A = -\frac{2}{15}WL$$

$$M_C = +\frac{3}{50}WL$$

$$\Delta_{\max} = \frac{WL^3}{212EI} \quad (\text{at } x = 0.447L)$$

29. Triangular load on part of the span



$$W = \frac{wa}{2}$$

$$R_B = V_B = \frac{Wa^2}{20L^3}(5L - a)$$

$$R_A = W - R_B$$

$$M_A = \frac{Wa}{60L^2}(3a^2 - 15aL + 20L^2)$$

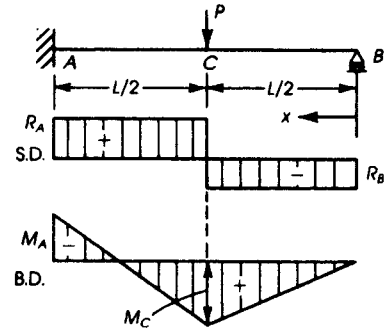
Maximum positive moment at

$$S = b + \frac{a^2}{2L}\sqrt{1 - \frac{a}{5L}}$$

$M_{\max}$ (positive at) D:

$$M_D = R_B S - \frac{WL}{3a^3}(-b + S)^3$$

30. Concentrated load at midspan



$$R_A = V_A = \frac{11P}{16}$$

$$R_B = V_B = \frac{5P}{16}$$

$$M_A = -\frac{3PL}{16}$$

$$M_C = \frac{5PL}{32}$$

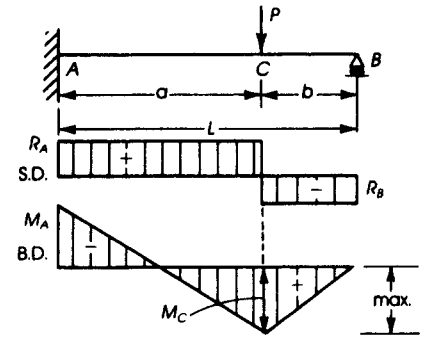
$$\Delta_C = \frac{7PL^3}{768EI}$$

$$\Delta_{\max} = \frac{PL^3}{107EI} \quad (\text{at } x = 0.447L \text{ from B})$$

(continued)

Table C.3 *(continued)*

31. Concentrated load at any point



$$R_A = V_A = P - R_B \quad R_B = V_B = \frac{Pa^2}{2L^3}(b + 2L)$$

$$M_A = -\frac{Pb(L^2 - b^2)}{2L^2}$$

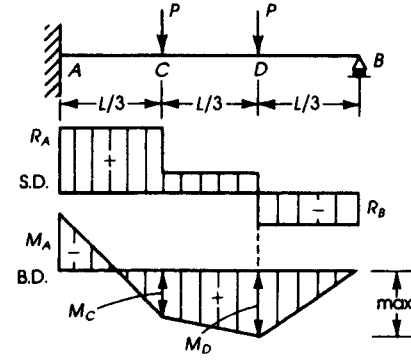
M_A max $= 0.193PL$ when $b = 0.577L$

$$M_C = \frac{Pb}{2}\left(2 - \frac{3b}{L} + \frac{b^3}{L^3}\right)$$

M_C max $= 0.174PL$ when $b = 0.366L$

$$\Delta_C = \frac{Pa^3b^2}{12EIL^3}(4L - a)$$

32. Two concentrated loads at one-third points



$$R_A = V_A = \frac{4P}{3}$$

$$R_B = V_B = \frac{2P}{3}$$

$$M_A = -\frac{PL}{3}$$

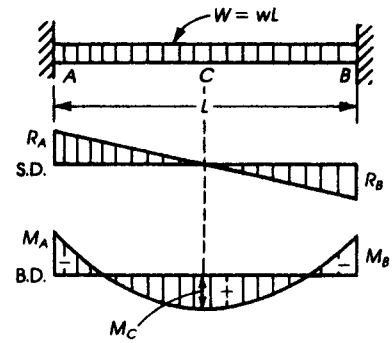
$$M_C = \frac{PL}{9} \quad M_D = \frac{2PL}{9}$$

$$\Delta_{max} = \frac{PL^3}{65.8EI}$$

occurs at point $= 0.423L$ from support B

Table C.4 Fixed End Beams

33. Uniform load



W = total load = wL

$$R_A = V_A = R_B = V_B = \frac{W}{2}$$

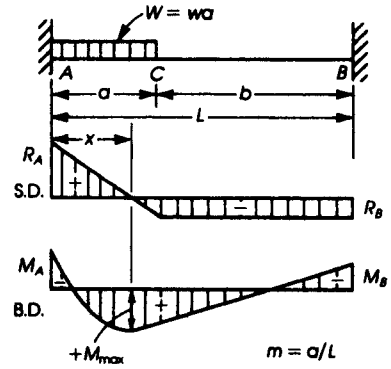
$$M_A = M_B = -\frac{WL}{12} \quad \text{(at support)}$$

$$M_C \text{ max} = \frac{WL}{24} \quad \text{(at midspan)}$$

$$\Delta_{\max} = \frac{WL^3}{384EI} \quad \text{(at midspan)}$$

$$\Delta_x = \frac{Wx^3}{24EIL}(L-x)^2 \quad \text{(from } A \text{ or } B\text{)}$$

34. Uniform partial load at one end



W = total load = wa $\quad m = \dfrac{a}{L}$

$$R_A = V_A = \frac{W(m^3 - 2m^2 + 2)}{2}$$

$$R_B = V_B = \frac{Wm^2(2-m)}{2} = W - R_A$$

$$M_A = \frac{WLm}{12}(3m^2 - 8m + 6)$$

$$M_B = \frac{WLm^2}{12}(4 - 3m)$$

$$M_{\max} = \frac{WLm^2}{12}\left(-\frac{3}{2}m^5 + 6m^4 - 6m^3 - 6m^2 + 15m - 8\right)$$

when $x = \dfrac{a}{2}(m^3 - 2m^2 + 2)$

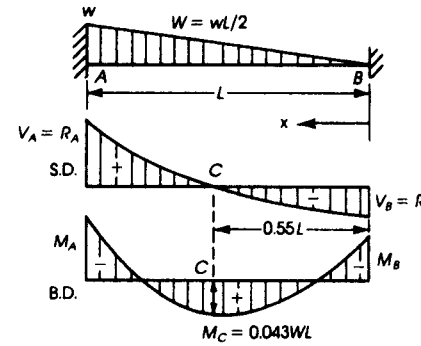
$$\Delta_{\max} = \frac{WL^3}{333EI}$$

$$\Delta_C = \frac{WL^3}{384EI}$$

(*continued*)

Table C.4 *(continued)*

35. Triangular load



$$W = \frac{wL}{2}$$

$$R_A = V_A = 0.7W$$

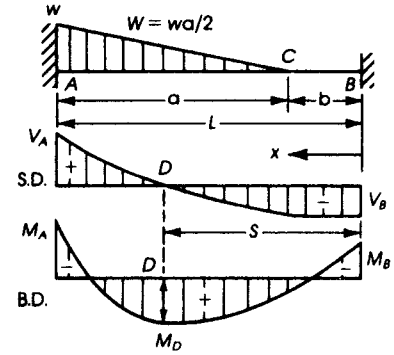
$$R_B = V_B = 0.3W$$

$$M_A = \frac{WL}{10} \quad M_B = \frac{WL}{15}$$

$$\Delta_{\max} = \frac{WL^3}{382EI} \text{(at } x = 0.55L \text{ from B)}$$

$$M_C \text{(maximum positive moment)} = +\frac{WL}{23.3} \text{(at} 0.55L \text{ from B)}$$

36. Triangular load on part of the span



$$W = \frac{wa}{2}$$

$$R_B = V_B = \frac{Wa^2}{10L^3}(5L - 2a)$$

$$R_A = W - R_B$$

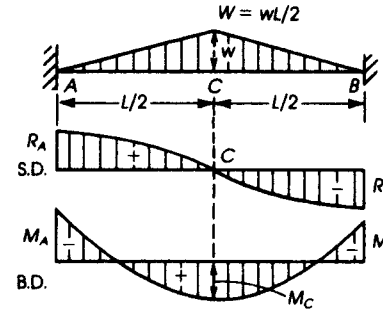
$$M_A = \frac{Wa}{30L^2}(3a^2 + 10bL)$$

$$M_B = \frac{Wa}{30L^2}(-3a^2 + 5aL)$$

$$\text{Maximum positive moment at } S = b + \frac{a^2}{3.16L}\sqrt{5 - \frac{2a}{L}}$$

$$M_D = R_B S - \frac{WL}{3a^3}(a + S - L)^3 - M_B$$

37. Triangular load, maximum intensity at midspan



$$W = \text{total load} = \frac{wL}{2}$$

$$R_A = R_B = \frac{W}{2}$$

$$M_A = M_B = -\frac{5}{48}WL$$

$$M_C \text{(maximum positive)} = \frac{WL}{16}$$

$$\Delta_{\max} = \frac{1.4WL^3}{384EI} \text{(at midspan)}$$

Table C.4 *(continued)*

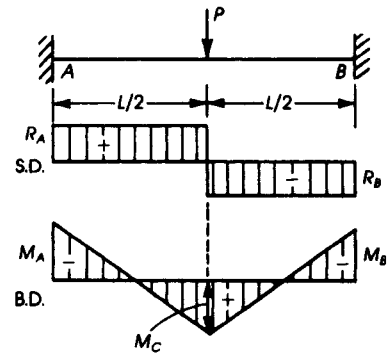
38. Concentrated load at midspan

$$R_A = V_A = R_B = V_B = \frac{P}{2}$$

$$M_A = M_B = M_C = -\frac{PL}{8}$$

$$\Delta_{max} = \frac{PL^3}{192EI} \text{(at midspan)}$$

$$\Delta_x = \frac{Px^2}{48EI}(3L - 4x)\left(x < \frac{L}{2}\right)$$



39. Two symmetrical concentrated loads

$$R_A = V_A = R_B = V_B = P$$

$$M_A = M_B = -\frac{Pa(L-a)}{L}$$

$$M_C = M_D = \frac{Pa^2}{L}$$

$$\Delta_{max} = \frac{PL^3}{6EI}\left[\frac{3a^2}{4L^2} - \left(\frac{a}{L}\right)^3\right] \text{(at midspan)}$$

If $a = \frac{L}{3}$,

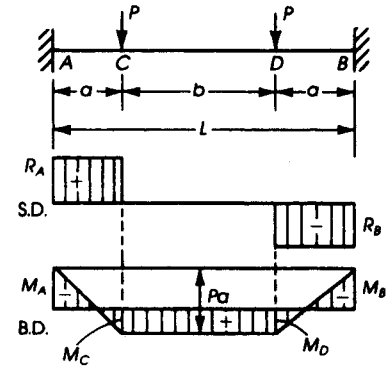
$$M_A = M_B = \frac{2}{9}PL$$

$$\Delta_{max} = \frac{5PL^3}{648EI} \text{(at centerline)}$$

If $a = \frac{L}{4}$,

$$M_A = M_B = \frac{3}{16}PL$$

$$\Delta_{max} = \frac{PL^3}{192EI} \text{(at centerline)}$$



(continued)

Table C.4 *(continued)*

40. Concentrated load at any point

$$R_A = V_A = P\left(\frac{b}{L}\right)^2\left(1+\frac{2a}{L}\right)$$

$$R_B = V_B = P\left(\frac{a}{L}\right)^2\left(1+\frac{2b}{L}\right)$$

$$M_A = -\frac{Pab^2}{L^2} \quad M_B = -\frac{Pba^2}{L^2} \quad M_C = \frac{2Pa^2b^2}{L^3}$$

$$\Delta_C = \frac{Pa^3b^3}{3EIL^3} \quad \text{(at point C)}$$

$$\Delta_{max} = \frac{2Pa^2b^3}{3EI(3L-2a)^2} \quad \text{when } x = \frac{2bL}{3L-2a} \quad \text{and } b > a$$

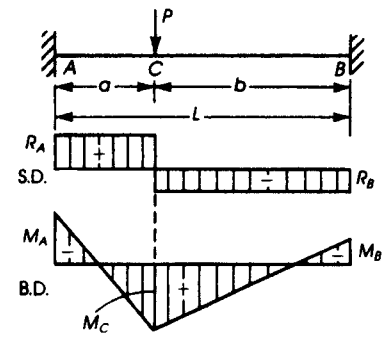
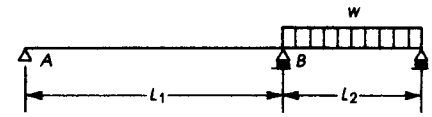


Table C.5 Moments in Two Unequal Spans and Values of the Coefficient K (w = Unit Load/Unit Length)

1. Load on short span

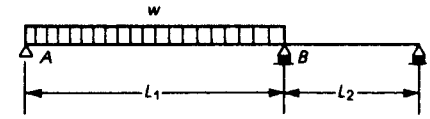
$$M_B = \frac{wL_2^3}{8(L_1+L_2)} = \frac{wL_2^2}{K}$$



L_2/L_1	0.20	0.25	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
K	46.0	40.0	34.7	28.0	24.0	21.4	19.5	18.0	16.9	15.9

2. Load on long span

$$M_B = \frac{wL_1^3}{8(L_1+L_2)} = \frac{wL_1^2}{K}$$



L_2/L_1	0.20	0.25	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
K	9.6	10.0	10.4	11.2	12.0	12.8	13.6	14.4	15.2	15.9

3. Both spans loaded with w_1 on L_1 and w_2 on L_2

$$M_B = \frac{w_1L_1^3 + w_2L_2^3}{8(L_1+L_2)}$$

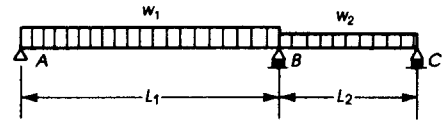
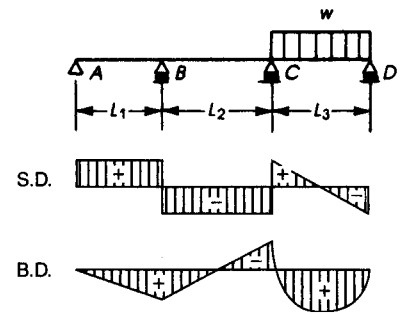


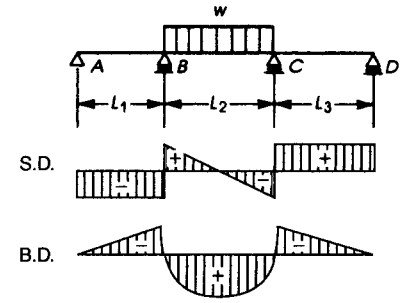
Table C.6 Moments in Three Unequal Spans and Values of the Coefficient K (w = Load/Unit Length)

4. Load on span *CD*



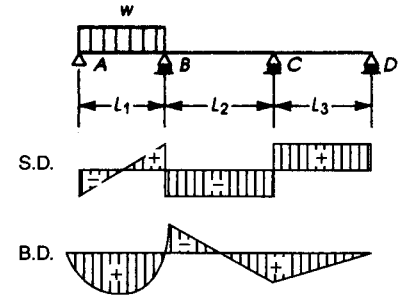
L_2/L_3	$M_B = \frac{wL_3^2}{K}$ (positive)	$M_C = \frac{wL_3^2}{K}$ (negative)
0.25	100.0	9.9
0.30	90.9	10.3
0.40	76.3	11.0
0.50	70.4	11.7
0.60	65.8	12.3
0.70	62.9	13.0
0.80	61.7	13.7
1.00	59.9	14.9

5. Load on middle span



L_2/L_1	$M_B = M_C = \frac{wL_2^2}{K}$ (negative)
0.25	43.5
0.30	38.5
0.40	32.3
0.50	27.8
0.60	25.6
0.70	23.3
0.80	22.2
1.00	20.0

6. Load on span *AB*

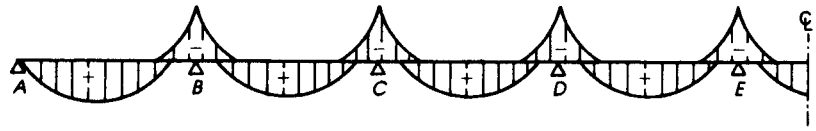


L_2/L_1	$M_B = \frac{wL_1^2}{K}$ (negative)	$M_C = \frac{wL_1^2}{K}$ (positive)
0.25	9.9	100.0
0.30	10.3	90.9
0.40	11.0	76.3
0.50	11.7	70.4
0.60	12.3	65.8
0.70	13.0	62.9
0.80	13.7	61.7
1.00	14.9	59.9

Table C.7 Maximum and Minimum Moments in Equal Spans Continuous Beams

7. Uniform loads

$$M = \frac{wL^2}{K}$$



where w = (D.L. + L.L.) per unit length D.L. = Uniform dead load L.L. = Uniform live load

Values of coefficient K

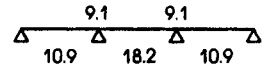
Ratio D.L./w	First span AB (positive moment)				Second support B (negative moment)			
	Number of spans				Number of spans			
	2	3	4	5	2	3	4	5
0.0	10.5	10.0	10.2	10.1	8.0	8.6	8.3	8.3
0.1	10.8	10.2	10.4	10.3	8.0	8.7	8.4	8.5
0.2	11.1	10.4	10.6	10.6	8.0	8.8	8.5	8.6
0.3	11.4	10.6	10.9	10.8	8.0	9.0	8.6	8.7
0.4	11.8	10.9	11.1	11.0	8.0	9.1	8.6	8.8
0.5	12.1	11.1	11.4	11.3	8.0	9.2	8.8	8.9
0.6	12.5	11.4	11.7	11.6	8.0	9.4	8.9	9.0
0.7	12.9	11.6	12.0	11.9	8.0	9.5	9.0	9.1
0.8	13.3	11.9	12.3	12.2	8.0	9.7	9.1	9.2
0.9	12.8	12.2	12.6	12.5	8.0	9.8	9.2	9.4
1.0	14.3	12.5	13.0	12.8	8.0	9.9	9.3	9.5

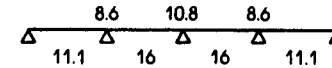
Ratio D.L./w	Second span BC (positive moment)			Third support C (negative moment)		Third span CD (positive moment)	Interior span (positive moment)	Interior support (negative moment)
	Number of spans			Spans		Span		
	3	4	5	4	5	5		
0.0	13.4	12.4	12.7	9.3	9.0	11.7	12.0	8.8
0.1	14.3	13.2	13.5	9.7	9.3	12.3	12.6	9.1
0.2	15.4	14.0	14.3	10.0	9.6	12.9	13.3	9.8
0.3	16.7	14.9	15.3	10.4	9.9	13.6	14.1	9.5
0.4	18.2	16.0	16.5	10.8	10.2	14.3	15.0	9.9
0.5	20.0	17.2	17.9	11.5	10.5	15.2	16.0	10.1
0.6	22.2	18.7	19.5	11.7	10.9	16.2	17.2	10.5
0.7	25.0	20.4	21.4	12.2	11.3	17.3	18.4	10.8
0.8	28.6	22.4	23.8	12.7	11.7	18.5	20.0	11.2
0.9	33.3	24.9	26.6	13.3	12.2	20.0	21.8	11.6
1.0	40.0	28.3	30.0	14.0	12.7	21.7	24.0	12.0

Table C.7 *(continued)*

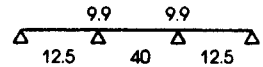
Example: K values

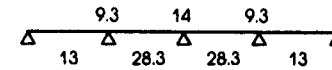
1. $\dfrac{\text{D.L.}}{w} = 0.4$





2. $\dfrac{\text{D.L.}}{w} = 1.0$



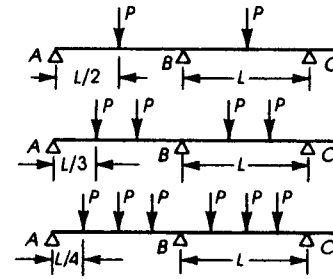


8. Concentrated loads

P' = concentrated dead load

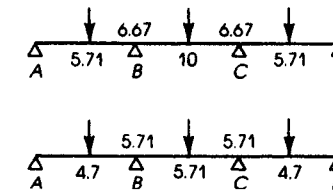
P'' = concentrated live load

$$M = \left(\frac{P'}{K_1} + \frac{P''}{K_2}\right) L$$



Number of Spans	First Span AB						Second Support B					
	K_1 (D.L.)			K_2 (L.L.)			K_1 (D.L.)			K_2 (L.L.)		
	2	3	4	2	3	4	2	3	4	2	3	4
Central load	6.40	5.71	5.89	4.92	4.70	4.76	5.35	6.67	6.22	5.33	5.71	5.53
One-third-point loads	4.50	4.09	4.20	3.60	3.46	3.50	3.00	3.75	3.50	3.00	3.21	3.11
One-fourth-point loads	3.67	3.20	3.34	2.61	2.46	2.50	2.13	2.67	2.49	2.13	2.28	2.21

Number of Spans	Second span BC				Third Support C	
	K_1		K_2		K_1	K_2
	3	4	3	4	4	4
Central load	10.00	8.61	5.71	5.46	9.33	6.22
One-third-point loads	15.00	9.00	5.00	4.50	5.25	3.50
One-fourth-point loads	8.00	6.05	3.20	3.01	3.72	2.49



Example: K values

K_1(dead load) $$M_{AB}(\max) = \left(\frac{P'}{5.71} + \frac{P''}{4.7}\right) L$$

$$-M_B(\max) = \left(\frac{P'}{6.67} + \frac{P''}{5.71}\right) L$$

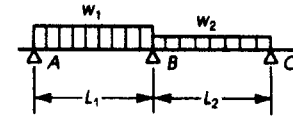
K_2(live load) $$M_{BC}(\max) = \left(\frac{P'}{10} + \frac{P''}{5.71}\right) L$$

Table C.8 Moments in Unequal Spans Continuous Beams Subjected to Unequal Loads

9. Unequal spans and unequal loads. For approximate bending moments in continuous beams, use

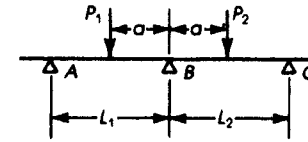
$L' = 0.8L$ for spans continuous at both ends

$L' = L$ for spans continuousatonly one end



1. Uniform loads (load on two adjacent spans):

$$M_B = \frac{w_1 L_1'^3 + w_2 L_2'^3}{8.5(L_1' + L_2')}$$



2. Concentrated loads:

$$M_B = \frac{K P_1 L_1'^2}{L_1' + L_2'} \text{ due to load } P_1$$

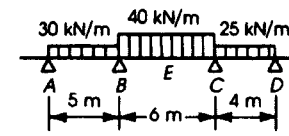
$$M_B = \frac{K P_2 L_2'^2}{L_1' + L_2'} \text{ due to load } P_1$$

a/L	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
K	0.080	0.136	0.168	0.182	0.176	0.158	0.128	0.090	0.050	0.000

3. Moments within span:

- Maximum positive moment is obtained by superposing B.M. due to D.L. + L.L. and the negative moments at supports due to D.L. only.
- Maximum negative moment is obtained by superposing B.M. due to D.L. only and the negative moments at supports due to D.L. + L.L.

Example:



$$M_B = -\frac{30(0.8 \times 5)^3 + 40(0.8 \times 6)^3}{8.5(0.8 \times 5 + 0.8 \times 6)} = -84.8 \text{kN m}$$

$$M_C = -\frac{40(0.8 \times 6)^3 + 25(0.8 \times 6)^3}{8.5(0.8 \times 4 + 0.8 \times 6)} = -77.1 \text{kN m}$$

$$M_E = (\text{at centerline of BC}) = +\frac{wL^2}{8} + \frac{1}{2}(M_B + M_C) = \frac{40 \times 36}{8} + \frac{1}{2}(-84.8 - 77.1) = +99 \text{kN·m}$$

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